CONDITION ASSESSMENT OF (FRP) FIBER REINFORCED POLYMER STRENGTHENING OF BRIDGE COMPONENTS

by

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Abstract

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Fiber Reinforced Polymers (FRP) is a newly developed composite type of material which have many advantages over conventional materials (concrete and steel) like light weight, non-corrosive nature, design flexibility and quick installation. Due to its superior thermo-mechanical properties, it has emerged as one of the best economical method to strengthen and repair reinforced concrete bridges that were damaged each year due to various reasons like structural deterioration over time, fire accidents, corrosion of steel reinforcement and over height vehicle collision. This research aims to evaluate the long term performance of FRP strengthened bridge components with the help of Pull-Off tests. It will also recommend certain guidelines to be followed during the time of repair for the better performance of FRP. In this study a total of eight concrete bridges in the DFW area, that were damaged in the past and strengthened using Carbon Fiber Reinforced Polymer (CFRP) wrap were identified and later Pull-Off tests were performed on selected test locations of the bridge components as per the specifications of ASTM D7522/D7522M (2009) .After evaluation of the Pull-Off test results it was determined that the overall performance of FRP is good.

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Chapter 1

Introduction

1.1 Current State of Bridge Infrastructure in USA

According to the American Society of Civil Engineers (ASCE) 2013 Report card for America's Infrastructure published in the year 2013, one in nine (11%) of the nation's bridges are structurally deficient. Each year, a considerable number of bridges are damaged due to various reasons like vehicle collision, fire, corrosion of steel reinforcement, structural deterioration over time and design flaws. In regard to the total number of bridges to be repaired, the U.S Department of Transportation (USDOT) determined that out of 607,380 bridges, 13.95% (84,748) can be classified as functionally obsolete, and 10.98% (66,749) as structurally deficient. To eliminate the nation's bridge deficient backlog by the year 2028, a total of $20.5 billion is required annually which is $7.7 billion more than the current spending of $12.8 billion(Report card on America's Infrastructure, 2013).

According to the Federal Highway Administration (FHWA) estimation, more than 30% of the bridges in service today have exceeded their 50 year design life. Hence, repair works, maintenance and rehabilitation programs will still require significant investment in the upcoming years (Report card on America's Infrastructure, 2013). As per the report of U.S DOT Federal Highway Administration (2012), the estimated total rehabilitation and replacement costs for all the structurally deficient bridges are $35.2 billion and $51.8 billion, respectively. Table 1-1 presents the information about the total number of structurally deficient bridges and the number of replaced bridges by state in 2012 (U.S. DOT, 2012).

Table 1-1 Structurally Deficient National Highway System (NHS) and Non-National Highway System (NNHS) Bridges by State (USDOT)

State	Total Number of SD NHS and NNHS Bridges	Total Area (M²) of SD NHS and NNHS Bridges	Total Number of SD NHS and NNHS Bridges Replaced in 2012	Total Area (M²) of SD NHS and NNHS Bridges Replaced in 2012
AK	128	68823	3	2937
AL	1448	342546	13	21439
AR	898	348219	15	29488
AZ	247	216443	8	21951
CA	2978	4430018	11	8396
CO	566	268894	11	13460
CT	406	548027	12	8105
DC	30	97552	0	0
DE	53	40448	1	71
FL	262	469030	8	37002
GA	878	301542	21	28591
HI	146	45228	1	1434
IA	5193	934995	45	21815
ID	397	128013	9	10092
IL	2311	1269106	84	50617
IN	2036	767158	22	9351
KS	2658	401519	7	5099
KY	1244	1244	20	19781
LA	1783	1783	51	34960
MA	493	493	14	8824
MD	368	368	8	6516
ME	356	356	7	8736
MI	1354	1354	42	17497
MN	1190	1190	15	6624
MO	3528	3528	97	40560
MS	2417	2417	32	34243
MT	399	399	5	2647
NC	2192	2192	100	71481
ND	746	746	2	860
NE	2779	2779	22	25815
NH	362	362	0	0
NJ	651	651	6	3748

Table 1-1—Continued

State	Total Number of SD NHS and NNHS Bridges	Total Area (M²) of SD NHS and NNHS Bridges	Total Number of SD NHS and NNHS Bridges Replaced in 2012	Total Area (M²) of SD NHS and NNHS Bridges Replaced in 2012
NM	307	307	9	7259
NY	2169	2169	29	13537
OH	2462	2462	47	35337
OK	5382	5382	79	59301
OR	433	433	11	3133
PA	5540	5540	118	76124
PR	282	282	4	2224
RI	156	156	6	2841
SC	1141	1141	8	31878
SD	1208	1208	8	4333
TN	1195	501053	13	29801
TX	1372	755572	120	125887
UT	126	36444	5	1828
VA	1250	566671	19	8502
VT	288	73164	6	6728
WA	366	636350	9	6765
WI	1157	375044	28	12791
WV	952	300865	24	8768
WY	426	195097	3	1300
TOTAL	66749	29456198	1238	990478

Based on the above-mentioned facts, it is clear that there is an urgent requirement to strengthen the structurally deficient bridges to extend the service life and make them safer. Fiber Reinforced Polymers (FRP) is a newly developed composite type of material which have many advantages over conventional materials (concrete and steel) like light weight, non-corrosive nature, design flexibility and quick installation. Under the limited funding scenario, Fiber reinforced polymer (FRP) strengthening method turns out to be the best and most economical bridge retrofitting method for concrete bridges due to its superior thermo-mechanical properties (high strength and stiffness) and cost savings. Structural rehabilitation using FRP not only costs less (usually 1/15th to 1/10th of replacement costs) but also saves a lot of construction time due to its rapid installation, thus avoiding traffic congestion and accidents.

Depending on the severity of damage to the bridge components different types of strengthening methods are available.

Surface patching:

It is a technique in which new and existing concrete are bonded by means of a bonding agent. Surface or shallow patching is used when the damage is between the surface and the first mat of reinforcing steel. Cement-based mortar or concrete, nonshrink quick-setting mortar, and epoxy mortar are used as bonding agents for surface patching. When the deteriorated concrete is too deep for surface patching and extends deeper than the first layer of reinforcing steel, cement-based polymer concrete (mixture of polymers with cement-based mortar) should be used.

Pneumatically Placed Concrete:

It is a technique in which a mixture of sand, cement, water and admixtures is sprayed at high velocity onto concrete surfaces where a bonding agent is applied priorly. This method is used when large sections of bridge components is to be repaired and depth of concrete deterioration is low.

The following repair methods are used for bridge girders with significant concrete deterioration and corroded strands in prestressed concrete bridges.

Post-tensioning:

This method includes the placement of symmetrical jacking corbels on either side of the damaged area, in the sound sections of the beam, and anchoring them to the bottom flange. Later, post-tensioning tendons are passed through the corbels and anchored against the bearing plates. Preloading is done before the concrete is repaired, and when the patched concrete gains sufficient strength, the preloading is removed, and the exterior post-tensioning of the beam is applied, simultaneously at both corbels. Threaded bars are normally placed in plastic conduits, and later pressure grouted.

Metal Sleeve Splice:

This method is used where there are many severed strands, and a large quantity of concrete is deteriorated, to restore the beam to its function. This method includes applying the necessary preloading, replacing the concrete, removing preloading after the concrete has gained sufficient strength and installing the metal sleeve. This method does not normally restore prestress, though partial or full prestress may be restored by preloading.

Bridge strengthening using FRP wrap has many advantages over the above mentioned repair methods like very less weight, quick installation, long durability, fatigue and impact resistance, excellent quality control, less interception to traffic flow and more cost savings.

1.2 Fiber Reinforced Polymers (FRP) as Bridge Strengthening Material

FRP is a composite type of material fabricated in the form of a polymer matrix reinforced with fibers. Usually, the polymer is an epoxy, vinyl ester or polyester and fibers are made of glass, carbon and aramid. Strengthening of structurally deficient Bridge components using FRP has many inherent advantages due to its non-corrosive nature, lightweight, flexibility in design, better aesthetics and superior mechanical properties (strength and stiffness), as compared to conventional materials (Ganga Rao & Vijay, 1998).

FRP is defined as a combination of high-strength, high-stiffness structural fibers with light weight, environmentally resisting polymers producing composite materials with mechanical properties and durability better than either of the constituents alone (Bakis et al, 2002). Carbon fiber reinforced polymer (CFRP) repair of bridges is the most common type of strengthening method, using high strength carbon fibers with a modulus of elasticity ranging from 32,000 ksi to 34,000 ksi. CFRP strengthening increases the axial, flexural, shear and impact resistance of the repaired bridge components.

To strengthen the damaged bridge girders, two types of techniques are available depending on the type of strength (shear or flexural) enhancement desired. In the case of the flexural strengthening, FRP wrap is applied to the bottom face (tension side) of the bridge girder with fibers orienting in the longitudinal axis direction. In the case of the shear strengthening, FRP wrap is applied to the web of the girder (U Straps) with fibers orienting transversely to the longitudinal axis. To strengthen the bridge columns, FRP is wrapped around the perimeter.

1.3 Research Objective

The objective of this study is to evaluate the long-term performance of damaged prestressed concrete bridge components strengthened with CFRP (Carbon Fiber Reinforced Polymer) using Pull-Off tests. While there are several available laboratory tests, due to the difficulties in achieving realistic field conditions in the laboratory through techniques like accelerated aging, exposure to freeze-thaw cycles and exposure to deicing agents, the test results may differ substantially from the actual field results. Therefore a field assessment of the FRP strengthened bridge components through Pull-Off test method and the use of this field data to evaluate the long-term behavior of FRP will be helpful in better understanding FRP as a bridge strengthening material .

1.4 Method and Approach

It has been shown that durability data generated through laboratory experiments can differ substantially from field data (Karbhari et al. 2003). Likewise, accelerated exposure data and real-time performance are unlikely to follow a simple linear relationship and the relationships have yet to be confidently determined (Byars et al. 2003). In the light of the above observations, this study involved the use of field test method (Pull-Off test) to evaluate the performance of FRP strengthened Bridge components in various Bridges that were damaged for various reasons.

As a part of this research, a total of eight bridges that were damaged in the past and strengthened using CFRP wrap were identified for field assessment in the DFW area. After the visual inspection, site inspection log for each bridge was prepared indicating the bridge location, date of inspection, number of components repaired with FRP, total number of spans, lanes to be closed for Pull-Off test and general remarks on the bridge condition. Based on the data from site inspection, the test locations for the Pull-Off test were identified, and lane closures were scheduled based on the location of the FRP strengthened components in relation to the traffic lanes. Thereafter, Pull-Off test was performed according to the specifications of ASTM D7522/D7522M (2009) on selected test locations of girders and columns. Based on the test results, the long term performance of FRP strengthened components was evaluated, and suitable conclusions are drawn accordingly.

Chapter 2

Literature Review

2.1 Strengthening of Reinforced Concrete Bridges using FRP Wraps

Bridge strengthening procedure using CFRP to repair prestressed concrete girders damaged due to over height vehicle collision is explained from Figures 2-1 to 2-13.

Figure 2-1 shows a damaged concrete girder due to over height vehicle collision.

Figure 2-1 Damaged concrete girder (Image Courtesy: TXDOT)

Firstly, the loose and unsound concrete around the damaged section is removed and then the damaged prestressed strands are spliced using mechanical devices (Figure 2-2).

Figure 2-2 Splicing of prestressed strands (Image Courtesy: TXDOT)

Figure 2-3 shows the steel pins attached to hold the spliced strands in position

Figure 2-3 Installed steel pins in repair areas (Image Courtesy: TXDOT)

Then the spliced strands are tensioned using special devices to achieve the desired strength (Figure 2-4).

Figure 2-4 Tensioning of spliced strands (Image Courtesy: TXDOT)

After tensioning the damaged strands, a 40 kip loaded truck is positioned to preload the beam before patching the concrete (Figure 2-5). In this way, when the preload is removed after the curing of patched concrete, it undergoes pre-compression.

Figure 2-5 Preloading of the beam before repair (Image Courtesy: TXDOT)

After loading the beam, sand blasting is used to remove the loose debris from the repair surface and then shotcreting is done (Figures 2-6 and 2-7).

Figure 2-6 Sandblasting to clean the repair areas (Image Courtesy: TXDOT)

Figure 2-7 Shotcreting (Image Courtesy: TXDOT)

After allowing the concrete to cure for a sufficient time, epoxy is used to fill the cracks left over after the concrete repair work (Figure 2-8).

Figure 2-8 Epoxy injection in to the cracks (Image Courtesy: TXDOT)

Before the application of primer and epoxy over the repaired concrete surface, the loaded truck is removed. Figures 2-9 and 2-10 shows the application of primer and epoxy, respectively. After applying the epoxy using nap roller, CFRP wrap is installed covering the entire bottom flange and web.

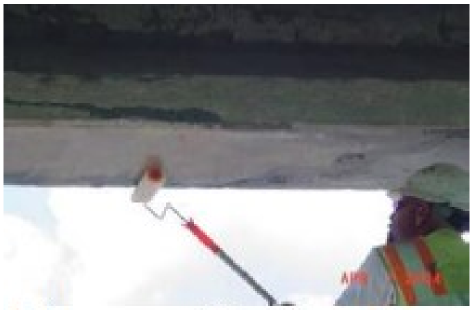


Figure 2-9 Application of primer (Image Courtesy: TXDOT)

Figure 2-10 Application of epoxy (Image Courtesy: TXDOT)

Figures 2-10 and 2-11 shows the application of the first and second layer of FRP strengthening respectively.

Figure 2-11 First Layer of FRP application (Image Courtesy: TXDOT)

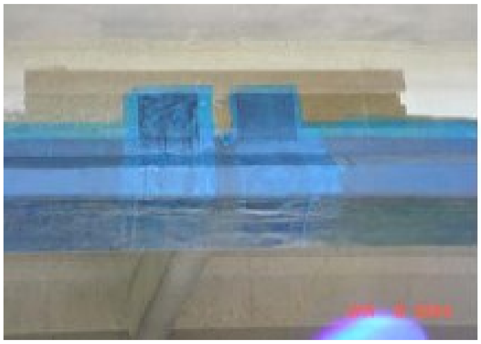


Figure 2-12 Applying second layer of FRP (Image Courtesy: TXDOT)

After allowing the epoxy to cure, a layer of surface coat is applied over the FRP wrap, preventing the damage of FRP due to various environmental factors (Figure 2-13).

Figure 2-13 Top coat application (Image Courtesy: TXDOT)

The damaged girder after strengthening using CFRP wrap is shown in the Figure 2-14.


APR 9 2004

Figure 2-14 Completed CFRP repair (Image Courtesy: TXDOT)

2.2 Advantages and Disadvantages of FRP

The tensile strength of carbon fibers is usually 550 ksi, which is almost ten times greater than the yield strength of mild steel. In addition to the superior thermo-mechanical properties FRP has many advantages over traditional repair materials (GangaRao & Vijay, 1998). Some of its advantages are listed below.

Less Cost: Cost of strengthening of damaged reinforced concrete structures using FRP is reasonable, usually 1/15th to 1/10th of the replacement cost. Cost savings due to lower material transportation costs, quicker construction times, and lower labor can offset the higher material costs of FRP. The service life is extended by additional 20-30 years (GangaRao & Vijay, 1998).

Quick Installation: Rehabilitation of structures using FRP approximately takes 80% less time than other bridge strengthening techniques like post tensioning and metal sleeve slice etc., thereby avoiding huge construction delays (GangaRao & Vijay, 1998).

Superior Engineering Properties: FRP has better engineering properties like strength and stiffness compared to traditional repair materials. In addition, FRP is non-corrosive in nature and less susceptible to moisture and chemical degradation.

Quality Control: Due to the manufacturing of FRP under controlled conditions in the industries it has less design flaws and superior quality.

Light Weight: As FRP is very light in weight, the transportation costs and dead weight added to the structure after repair is very less.

Some of the disadvantages of using FRP are higher material costs , less exposure to the use of FRP as repair material (long-term performance of FRP is not known completely yet), lack of availability of codes and specifications similar to conventional materials. In spite of higher initial costs, FRP strengthening of bridge components is economical due to its cost savings (lower material transportation costs, quicker construction times, and lower labor), long service life and superior engineering properties (Neale et al, 1998).

2.3 Bond between FRP and Concrete substrate

Karbhari and Ghosh (2009) stated that, since the composite element is bonded on to the concrete substrate, the efficacy of the rehabilitation scheme depends on the combined action of the entire system with emphasis on the integrity and durability of the bond between the FRP and concrete. In addition to that, Byars et al (2003) concluded that changes in mechanical properties such as Young’s modulus, tensile and interlaminar shear strengths and bond strength are the best indicators of changes in the performance of FRP. This clearly indicates the requirement of strong bond between FRP and concrete substrate for the success of FRP strengthening. Without the presence of a strong bond, there will be improper transfer of stresses leading to premature debonding and failure of repaired structures when subjected to environmental exposure (Karbhari & Gosh, 2009).

Environmental factors like moisture, chlorides, alkali, stress temperature, UV actions, carbonation and acid attack, affect the durability of FRP (Karbhari et al, 2003).

2.4 Tests on FRP

The behavior of bond between FRP and concrete substrate can be understood by various nondestructive and destructive evaluation methods. Sections 2.4.1 and 2.4.2 describes the different types of testing methods that are available for evaluating the bond quality and strength of FRP.

2.4.1 Nondestructive Evaluation Methods

Acoustic sounding

It is a technique in which the structure to be tested is tapped gently with a hammer or coin and changes in sound are noticed when areas of different bond quality are tapped (Clarke 2002). ASTM D4580 (2003), explains the procedures for performing the tap tests on FRP strengthened bridges. Chain dragging and Rotary Percussion are other types of sounding techniques to evaluate the bond quality of FRP. Acoustic Sounding is a very easy and effective method to detect the voids under the FRP strengthening. This method will not give accurate results when there are too many loud sounds at the time of testing.

Thermalgraphic Imaging

It is a technique in which the voids and defects in the structure to be tested are identified through the imaging of the thermal patterns at the object's surface. Thermalgraphic inspection is safe, nonintrusive and non-contact method using infrared cameras, where the detection of relatively shallow subsurface defects is done in a fast manner under large surfaces. ASTM D4788 (2003), explains the procedures for testing FRP strengthening. Voids do not conduct heat as solid materials, thereby creating a different set of thermal images. Controlled testing conditions are required to achieve accurate results from this test method.

2.4.2 Destructive Evaluation Methods

Differential Scanning Calorimetry (DSC)

The bond between FRP and concrete is essential to transfer loads through the polymer matrix or adhesive. Changes in the mechanical properties of the matrix material at temperatures above glass transition temperature(Tg), have the potential to cause loss of bond at only modestly increased temperatures, resulting in a loss of interaction between FRP and concrete (Bisby et al., 2005). Inconsistent epoxy mixture ratios, presence of moisture, level of saturation in the CFRP fabric of epoxy, and the temperature during the curing process affect the performance of the FRP. DSC is a thermal analysis technique used to determine the liquid-glass transition temperature, commonly referred to as “glass transition temperature” (T_g) of the CFRP and filler resins. Within this temperature range, the material undergoes a change in heat capacity though no phase change occurs. The glass transition temperature is dependent on the degree of cross-linking. With increasing cross-linking, the glass transition shifts to higher temperatures (User Com, 2000). Increasing temperature causes a solid material to transfer from a higher stiffness of a “glass” region to a less stiff “rubber” region. It is important for the glass transition temperature to be above any possible temperatures the composite might encounter during its service life. Because of this, low T_g values or occurrences such as fire or plasticization of the resin can be detrimental to FRP repaired structures (Allen, 2011).

Pull-Off Tests

The FRP pull-off test determines the greatest tension force (applied perpendicular to the surface) that the FRP-epoxy-concrete bond can resist. The method consists of adhesively bonding a metallic circular loading fixture (dolly) normal to the flat testing surface. The dolly contains a threaded hole in the center that allows for attachment of the fixed alignment adhesion testing device (pull-off tester). After attaching the tester, a tension force is applied gradually to the dolly until a partial or full detachment of the dolly is witnessed. The load witnessed at the time of rupture is regarded as maximum bond force.

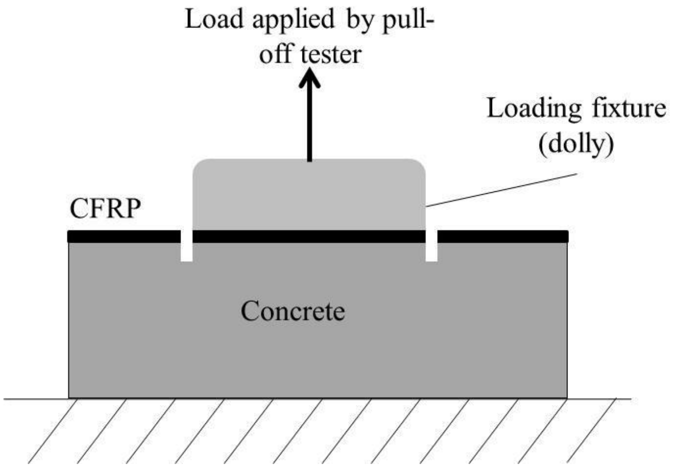


Figure 2-15 Pull off test mechanism

ASTM D7522/D7522M (2009), Standard Test Method for Pull-Off Strength for FRP Bonded to Concrete Substrate specifies the procedure to find the pull-off strength of FRP bonded to concrete. The maximum force recorded during each pull-off test is used to calculate the pull-off bond strength σ_p, as shown in Equation 1, where F_p is the pull-off force, and D is the diameter of the dolly.

$$\sigma_p = \frac{4F_p}{\pi D^2} \qquad (1)$$

The test procedure and failure modes classified by ASTM D7522/D7522M (2009) are discussed in a detailed manner in the following sections.

2.5 Pull-Off Tests

2.5.1 Overview

The main instruments required to perform pull-off tests are shown in Figure 2-2.

- Fixed Alignment Adhesion Test Device. It consists of the following parts.
    - Loading fixture - having a minimum diameter of 2 in. and a flat surface that can be adhered to the FRP surface.
    - Adhesion Tester Grip – having a central grip. It engages the loading fixture in a manner such that the resultant force is normal to the FRP surface.
    - Adhesion Tester Base – to permit uniform bearing against the FRP surface to react the test force.
- Timer – to control the rate of load application
- Force indicator and calibration information
- Loading fixture bonding adhesive (Epoxy)
- Circular hole cutter

Precautions during Testing:

- Misalignment between the circular test fixture and the drilled circle can lead to biased results.
- Testing at non-standard temperatures or relative humidity may affect the test results.
- The rate of stress applied to the FRP-concrete interface should be less than 150 psi/min.
- The depth of drilled core through the bonded FRP and adhesive layers and into the concrete substrate should be between 0.25 in. and 0.5 in.

- For non-horizontal surface, the adhesion tester assembly should be supported so that its weight does not contribute to the force exerted in the test.

Pull-Off Test Procedure (ASTM D7522, 2009)

- Select a circular loading fixture having a diameter not less than 2 in.
- Score through the FRP laminate into the substrate concrete using a core drill apparatus.
- Prepare the FRP surface for bonding the loading fixture. The FRP surface should be cleaned with solvent, sanded with medium-grit sandpaper, rinsed with solvent and allowed to dry.
- Attach the loading fixture with the designated bonding adhesive following the manufacturer's instructions.
- Carefully connect central grip of the adhesion tester to the loading fixture without bumping, bending or otherwise stressing the sample and connect the assembly to its control mechanism.
- Align the device according to the manufacturer's instructions and set the force indicator to zero.
- Apply manual or mechanized loading so that the continuous grip assembly motion results in a rate of applied normal stress at the FRP-concrete interface of less than 150 psi/min until rupture occurs.
- Record pull-off force measurement and nature of the failure plane.
- Record any departures from the procedure such as possible misalignment, hesitations in the force applications, etc.
- Clean the tested section with a solvent, sanded with medium-grit sandpaper, rinse with solvent and allowed to dry

- Fill the tested section (dent) with epoxy and reapply FPR strengthening of sufficient size.

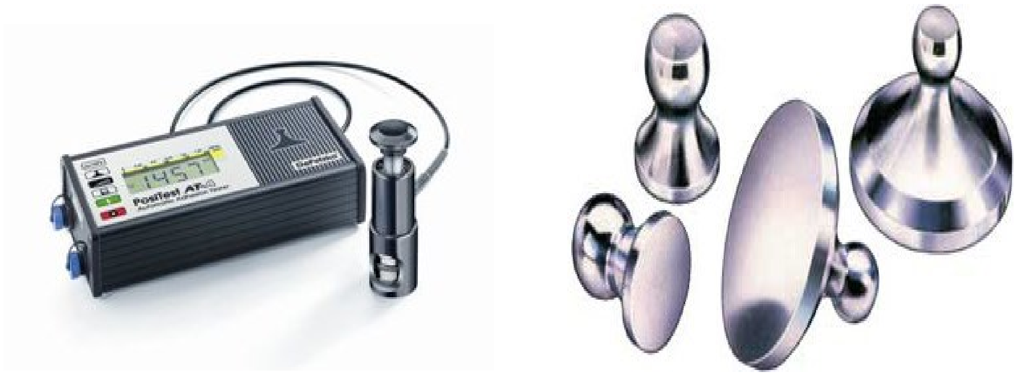
2.5.2 Advantages and Disadvantages of Pull-Off Test

Pull-Off test method has the following advantages:

- This method is quick and economical.
- On-site testing with only minimal damage to the FRP
- Immediate test results

The main disadvantage is that the results to pull off tests can vary drastically for locations within close range. The variability in results may also occur due to the torsional stresses induced at the time of core drilling. Other disadvantages are:

- The test method is not appropriate for determining the pull-off strength of the FRP from the patch material. An additional test method is required to determine the pull-off strength of the patch from the substrate concrete.
- Variations in results may be obtained using different devices.
- Improper surface preparation and improper curing of bonding adhesive will cause premature failure.
- Nonuniform FRP or FRP to substrate adhesive thickness in one specimen can affect an individual test result and lead to non-symmetrical or mixed mode failure pattern.
- Variation in FRP or adhesive thickness between specimens can cause biased or scattered test results. The FRP surface should be cleaned with solvent and sanded with medium-grit sandpaper, rinsed with solvent and allowed to dry. Manufacturer’s recommendations should be checked for the elapsed time between adhesive application and pull-off testing.



(a) Pull Off Tester Device (b) Circular Loading Fixtures

(c) Epoxy for Bonding Loading Fixtures (d) Core Drill

Figure 2-16 FRP Pull-Off Testing Apparatus and Materials

2.5.3 ASTM Standard Pull-Off Test Failure Modes

According to ASTM D7522/D7522 (2009) standard, seven types of failure modes are possible, labeled from Mode A through Mode G, as shown in Figure 2-4. Explanation of each mode is presented in Table 2-1. Various types of ASTM failure modes are further illustrated in the Figure 2-5.

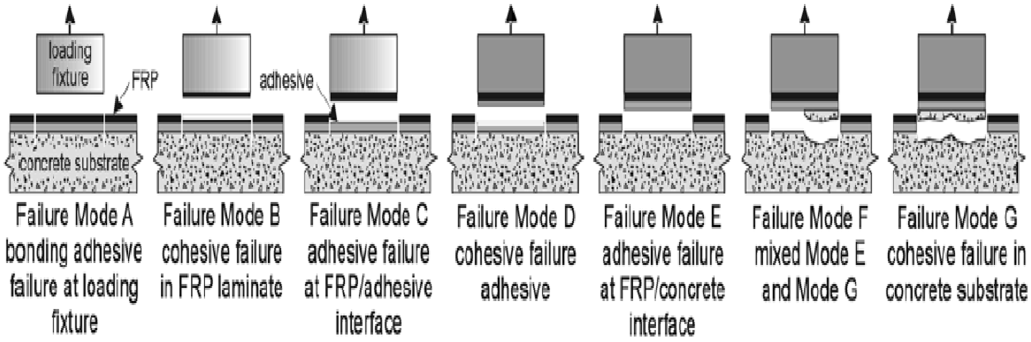


Figure 2-17 Failure modes for pull-off test (ASTM D7522/D7522M-09)

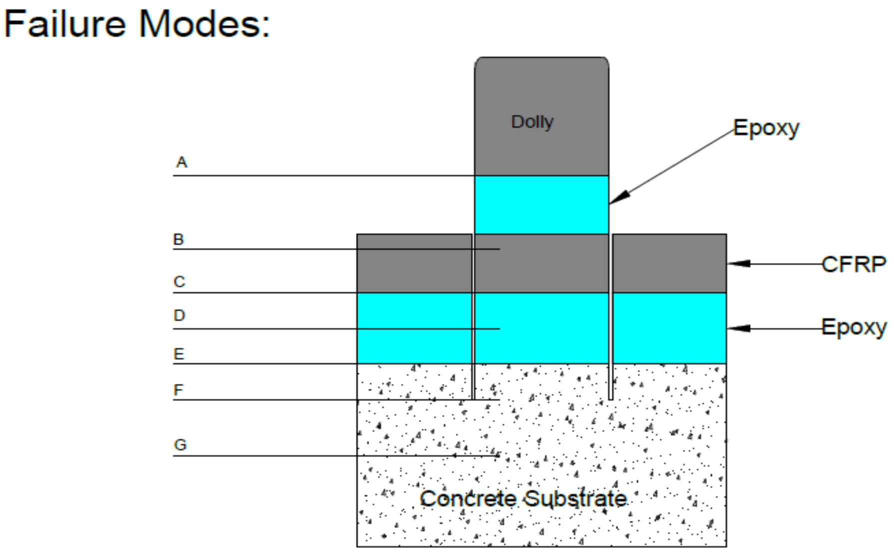


Figure 2-18 Illustration of pull off test failure modes

Table 2-1 Pull-off test failure modes (ASTM D7522/D7522M, 2009)

Failure Mode	Failure Type	Causes of Failure
A	Bonding epoxy failure at loading fixture.	Use of an inappropriate bonding epoxy system for affixing the loading fixture.
B	Cohesive failure in FRP laminate.	Incomplete wet-out of the fibers or plies comprising the laminate or may also result from environmental degradation of the FRP material itself.
C	Epoxy failure at FRP/epoxy interface.	Improper selection of epoxy for adherent materials, contamination of epoxy, improper or incomplete cure of epoxy, contamination or improper preparation or cleaning of adherent surfaces or environmental degradation.
D	Cohesive failure in epoxy.	Contamination of epoxy, incomplete cure, environmental damage of material.
E	Epoxy failure at FRP/concrete interface.	Improper selection of epoxy for adherent materials, contamination of epoxy, improper or incomplete cure of epoxy, contamination or improper preparation or cleaning of adherent surfaces or environmental degradation.
F	Mixed cohesive failure in substrate and epoxy failure at the epoxy/substrate interface	Inconsistent FRP-concrete adhesion. Failure is partly epoxy and partly on substrate
G	Cohesive failure in concrete substrate	Proper adhesion of FRP-concrete. Desirable failure mode

Chapter 3

Pull-Off Test Literature Review

3.1 Previous Research on FRP

The impact of freeze-thaw exposure on FRP strengthening is an important factor to be considered, as a significant number of bridges are located in regions where freeze-thaw cycling is very common. Since both the FRP and concrete have different coefficients of thermal expansion, large changes of temperature can create excessive stresses in the FRP-concrete interface causing premature de-bonding (Yun and Wu, 2010).

Green et al, (2000) conducted a study to evaluate the durability of FRP bonded to concrete by subjecting it to freeze–thaw cycles (16 hours of freezing and 8 hours of thawing in water, a total of 50, 150, and 300 freeze-thaw cycles). Temperatures were varied from 0°F in the freezing stage to 59°F in the thawing stage. Single shear tests were performed on 12 samples of size 6 in x 6 in x 16 in. In addition, nine small beams of size 4 in x 6 in x 48 in were tested under bending after achieving 28 day concrete strength of 4.5 ksi. CFRP wrap reinforcement of size 2 in x 12 in was used for this study, one strip placed in the center of the blocks for shear testing and two strips placed on the ends of the beams. Surprisingly, direct shear test results in this study indicated an increase in the ultimate load and maximum strain with increasing number of freeze-thaw cycles instead of causing degradation of the specimen. This may have been due to the enhanced curing process of the specimens that were subjected to freeze-thaw exposures in water bath for the entire cycling period, as opposed to the control specimens which were left dry and in room temperature. Also, with an increase in the number of cycles the failure plane was transferred into the FRP-epoxy layer. Control specimens showed a full failure in the substrate, whereas specimens undergoing conditioning experienced a more adhesive failure. This study concluded 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 (Green et al, 2000).

In addition to the freeze-thaw effect, careful attention should be paid to the exposure of CFRP and concrete to moisture and other kinds of solutions such as salt water or deicing agents. Malvar et al, 2003 conducted a study to evaluate the effects of moisture and chloride content on the CFRP bond to concrete by using Pull-Off tests. This study involved the testing of square concrete pile exposed to saltwater and marine conditions for 48 months. Later the pile was strengthened with CFRP. Pull-Off tests were split into two methods. In case of method-1, a primer and hydro blasting was used on the surface and where as in the second case neither the primer nor hydro blasting was used. Pull-Off test results indicated 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 (Malvar et al, 2003).

Pan et al (2010) studied the effect of chloride content on the behavior of bond between concrete and FRP. A total of 14 concrete specimens with dimensions of 6 in x 6 in 6 in were cast using two different concrete strengths. Ten blocks with average compressive strength 2.50 ksi, and four blocks with a strength of 2.33 ksi. The specimens were strengthened with two layers of FRP wrap and immersed in chloride solution. Four solutions with varying levels of sodium chloride concentration 3%, 6%, 10%, and 15% and exposure times of 0, 15, 30, 60, 90, and 120 days were used. Direct shear tests were conducted on these specimens to determine the possible bond deterioration over time. All specimens exhibited a de-bonding type of failure and the ultimate load decreased in the specimens that were immersed for 0 to 30 days, due to the deterioration of the adhesive when exposed to the chloride solution (Pan et al, 2010). Also, the load decreased in those specimens that were immersed in chloride solution for more than 90 days due to degradation of both the concrete and the adhesive (Pan et al, 2010). On the other hand, the specimens immersed between 30 and 60 days showed an increase in the ultimate load, which might be caused because of the increase in concrete strength resulting from further hydration (Pan et al , 2010).

Dai et al (2010) conducted a study performing Pull-Off and flexural tests on FRP strengthened concrete specimens after exposing them to 8, 14, and 24 months of wet-dry cycles(four days of immersion in sea water followed by a three-day drying period at room temperature). The study 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 (Dai et al , 2010).

Allen (2011), conducted a field assessment on the Castlewood Canyon Bridge located in Fortcolins, Colorado, which was strengthened with FRP in the year 2003. The arches were strengthened in shear, flexure, and axially using CFRP wet layup process. Pull-off tests, tensile tests, and differential scanning calorimetry (DSC) were chosen to evaluate the durability of the FRP application. A total of 42 pull-off tests were conducted in 2003 as per ASTM specifications. The summary of 2003 Pull-Off tests is tabulated in Table 3-1 below.

Table 3-1 Summary of Failure Modes for Pull-Off Tests (Allen, 2011)

42 Tests	Failure Modes of Pull-off tests in 2003							
	A	B	C	D	E	F	G	NA
Quantity	9	0	0	0	2	3	25	3
Percentage	21.4	0	0	0	4.8	7.1	59.5	7.1

After eight years, the condition of the CFRP material and its bond were evaluated once again in the year 2011. A total of 27 pull-off tests were conducted on selected test locations of the bridge in 2011. The summary of 2011 Pull-Off tests is tabulated in Table 3-2 below.

Table 3-2 Summary of Failure Modes for Pull-Off Tests (Allen, 2011)

27 Tests	Failure Modes of 2011 Pull-off tests in 2011							
	A	B	C	D	E	F	G	NA
Quantity	2	2	0	0	7	8	8	2
Percentage	7.8	7.8	0	0	25.9	29.6	29.6	7.4

Some important observations made by Allen (2011), are presented below. “Increase in percentage of failure modes B, E, and F indicates that other interfaces other than within the substrate are weaker and controlling”(Allen, 2011). “Tensile strength of concrete could have improved marginally since 2003 due to continued curing especially if the substrate was shotcrete rather than the original concrete”, to judge this, Allen (2011) compared bond strengths of the Pull-Off tests in 2003 and 2011 considering only samples failing in mode G. Below is Table 3-3 with strengths of failure mode G for comparison.

Table 3-3 Pull-off Test Results of Failure Mode G Tests (Allen, 2011)

Year	Average	Maximum	Minimum	Sample Size
	psi	psi	psi	
2003	423	597	217	25
2011	300	553	19	8

Other important observations made by Allen (2011), are the number of voids increased from 3% to 28% over 8 years of service. Voids found previously had an average increase in size of approximately 400%. Pull-off test failure modes were distributed differently than 2003 results with more failures occurring in the FRP layer Pull-off tests results of 2011 had a lower mean and higher standard deviation than the 2003 results.33% (9 of 27) of pull-off tests in 2011 were below the minimum 200 psi compared to 2.4% (1 of 42) in 2003. The tensile strength of the substrate decreased, along with average, maximum, and minimum strength values. Of the 27 pull-off tests conducted in 2011, nine tests, two of which were failure mode G, failed to meet the 200 psi minimum requirement. The increase in number of voids and size of existing voids, change in distribution of failure modes, decrease in average bond strength with more inadequate strength values, and increase in variance of bond strengths indicate deterioration of the bond between the CFRP composite and the concrete arch over time (Allen, 2011). Finally Allen, 2011, concluded that it would be prudent to monitor the durability and performance of the bond closely and consistently to try and accurately quantify the development of the degradation.

Hag-Elsafi et al (2004) evaluated the performance of the Wynantskill Creek Bridge in New York that was strengthened with FRP in 1999 to increase the shear and flexural capacities. Prior to the application of FRP and immediately after the FRP repair, strain gauges were installed, and load tests were conducted using the NYSDOT dump trucks to find the performance of the repaired bridge. An additional load test conducted in November, 2001 showed no signs of deterioration of the FRP strengthening. From the test results, Hag-Elsafi et al, (2004) concluded that there was no signs of deterioration in the retrofit system after two years of service. This study confirms that the FRP strengthening is durable and satisfactory to the conditions between November, 1999 and November, 2001.

Mata Carrillo (2012), studied the behavior of the bond between the concrete and the CFRP when subjected to various environmental conditions like freeze-thaw cycles, wet-dry cycles and immersion in deicing agents over two testing stages(6 and 12 months). Pull-Off tests and three-point bending tests were used in this study. Concrete blocks and beams with CFRP were placed face down in a 0.25 in - 0.50 in depth of deicing solution (Magnesium Chloride solution) for 4 days and were removed from the containers and allowed to dry for 3 days. These cycles were repeated for 6 and 12 months. Stage 1 was the testing stage after 6 months of environmental exposure in which a total of 15 blocks and 13 beams were tested. In Stage 2 testing after 12 months of exposure, 15 blocks and 13 beams were tested. A total of 45 concrete blocks were casted for conducting the pull-off strength tests and were reinforced with13 in x 5 in FRP wrap. Three pull-off tests were performed in each block of size 14 in x 6 in x 3.5 in (135 pull-off tests in total) for the entire study. Stage zero test results are summarized in Table 3-4.

Table 3-4 Stage Zero Pull-Off Test Results (Matacarrillo, 2012)

Test Label	Pull-off Strength (psi)	Failure Mode (ASTM D7522)
1	374	A
2	442	F
3	391	F
4	444	A
5	270	A
6	266	A
7	405	A
8	401	A
9	433	F
10	439	F

Majority of the Pull-Off test results after stage zero showed Mode A type of failure. Matacarrillo (2012) predicted the following possible reasons for this type of behavior, twisting of the dolly during adhesion causing minor air voids in the dolly-CFRP interface there by decreasing the bond strength and in also the experimental errors during testing.

Forty-five pull-off tests were performed during first stage of this research after 6 months. The samples were subjected to water immersion, wet-dry 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). After allowing the specimens to dry in room temperature for five days, the Pull-Off tests are performed later.

Stage one test results are presented in Table 3-5. There are large variances in the results from the pull-off tests. Control specimens showed the lowest strengths in relation to the other groups and also large variations in magnitude among specimens that underwent similar conditioning are observed.(Matacarrillo, 2012)

Matacarrillo(2012) listed several potential reasons for such discrepancies in the results as inconsistencies in the depth of the core drilling, improper mixing of epoxy and varying volumes of epoxy used for each dolly, twisting of the dollies when adhering to the FRP surface.

As part of stage two of this research, a total of forty-five pull-off tests were performed on the samples that were subjected to the same environmental factors as in stage one. The results are presented in Table 3-6. A total of thirty-two (71%) samples out of forty five exhibited Mode-F type of failure.

After the analysis of Pull-Off test results of all the three stages, Matacarrillo, 2012 concluded that, 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.

Table 3-5 Stage one Pull-Off Test Results (Matacarrillo, 2012)

Sample	Exposure	Pull-Off Strength(psi)	Failure Mode
1	Dry	226	F
2		152	F
3		139	A
4	Immersion in water	528	A
5		513	A
6		408	A
7		308	A
8		304	A
9		173	F
10	Wet-Dry in chloride deicer	89	F
11		132	F
12		133	F
13		291	F
14		579	F
15		530	F
16	Non chloride deicer	179	F
17		458	F
18		432	F
19		579	F
20		475	F
21		575	F
22		101	F
23		403	G
24		294	F
25		82	F
26		142	F
27		224	F
28	Chloride deicer	408	F
29		405	F
30		528	A
31		375	A
32		355	F
33		237	F
34		467	F
35		627	G
36		522	F
37		389	F
38		80	F
39		-	-
40	Freeze thaw in chloride deicer	313	F
41		296	A
42		287	A
43		422	F
44		351	F
45		291	A

Table 3-6 Stage 2 Pull-Off Test Results (Matacarrillo, 2012)

Sample	Exposure	Pull-Off Strength(psi)	Failure Mode
1	Dry	245	F
2		249	F
3		365	F
4	Immersion in water	382	A
5		353	A
6		367	A
7		209	F
8		399	F
9		425	F
10	Wet-Dry in chloride deicer	192	F
11		152	F
12		334	F
13		275	F
14		313	F
15		66	F
16	Non- chloride deicer	340	A
17		308	A
18		311	F
19		372	F
20		425	G
21		386	F
22		330	F
23		310	F
24		306	A
25		131	F
26		239	F
27		241	F
28	Chloride Deicer	95	F
29		154	F
30		329	F
31		289	A
32		163	F
33		190	F
34		139	F
35		329	F
36		270	F
37		281	F
38		273	A
39		251	F
40	Freeze thaw in chloride deicer	330	F
41		283	F
42		348	A
43		344	A
44		311	A
45		494	A

Chapter 4

Field Assessment Using Pull-Off Tests

4.1 Bridge Selection Procedure for Pull-Off Tests

To achieve the objectives of this research, a list of bridges that were damaged in the past and strengthened using CFRP wrap were identified for field assessment. The Bridge Division of the TxDOT Dallas District was contacted in order to get an inventory of FRP strengthened concrete bridges in the DFW area. The list contained the bridge site location, County, District, NBI number, type of FRP repair, and the date of repair. After getting the information of all the FRP strengthened bridges, it was determined that a preliminary site visit to all the bridges would be beneficial in the selection of bridges for the Pull-Off test.

<u>Bridge Inspection Procedure</u>:

1. Eight bridges that were to be inspected and tested were identified from the list of FRP strengthened bridges.
2. Time Schedule was prepared for visual inspection of each bridge based on their location and proximity.
3. Site inspection log was prepared for each bridge indicating the bridge location, date of inspection, number of components repaired with FRP, total number of spans, lanes to be closed for pull out test and general remarks on the bridge condition.
4. On the day of inspection, the bridge components that were repaired with FRP were identified and images of the FRP strengthened components (column or girder), and concrete deterioration were captured using a camera.
5. After identification of the FRP strengthened components, each was visually inspected for air pockets, delamination, debonding and FRP degradation.

6. During the time of inspection the present condition of the FRP repaired components of various bridges The bridge components which were damaged for various reasons (after the last FRP retrofitting) and needed repair future repair work were identified.
7. The location of the FRP strengthened components in relation to the traffic lanes was identified and recorded. This information would be useful for scheduling traffic lane closures that would be needed for performing Pull-Off tests.

Based on the data gathered during the site visit, a total of eight bridges were selected for evaluation. The selected bridge information is presented in Table 4-1, and the bridge locations are shown in the Figure 4-1.

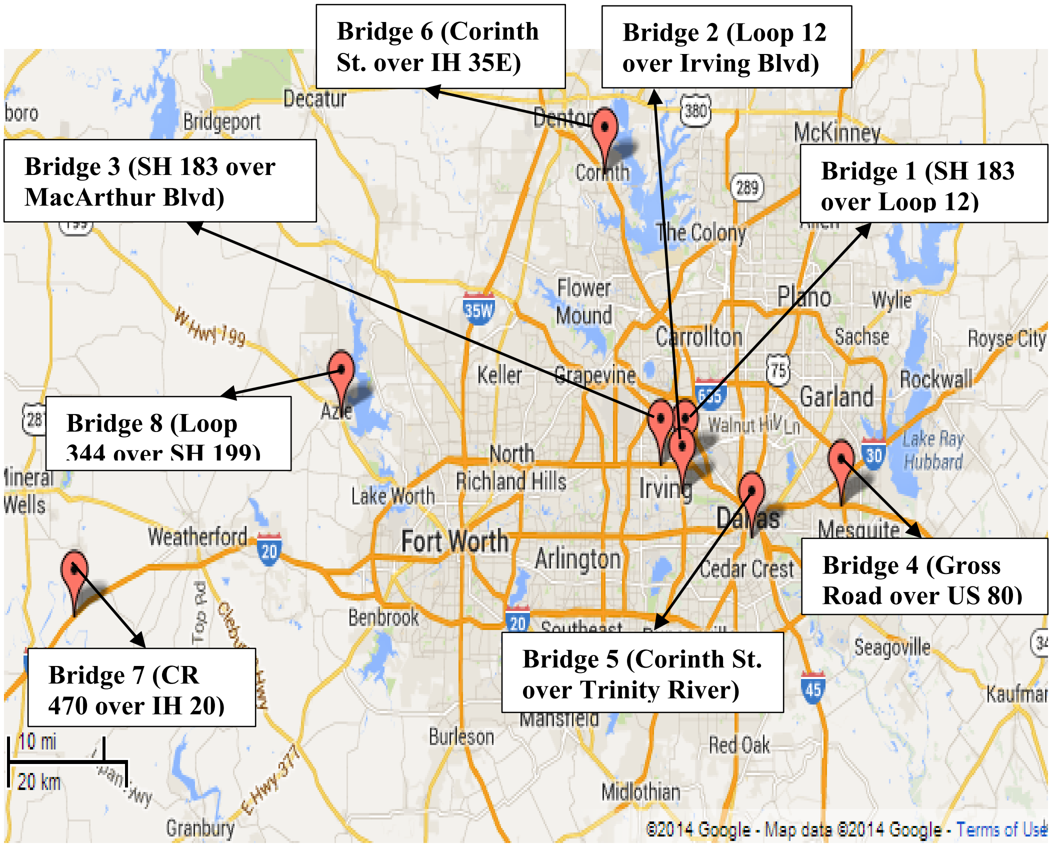


Figure 4-1 Location map of selected bridges with FRP strengthening

Table 4-1 List of selected bridges for pull off strength test

Bridge No.	Roadway	County	District	Location	Component Strengthened	Date of Strengthening	Date of Inspection	Pull off Test Date
1	LP 12	Dallas	Dallas	SH 183 over Loop 12	Girder	11/07/2006	05/28/2013	09/15/2013
2	LP 12	Dallas	Dallas	LP 12 over Irving Blvd.	Girder	07/14/2011	05/28/2013	09/15/2013
3	SH 183	Dallas	Dallas	SH 183 over MacArthur Blvd.	Column , Girder	12/21/2005	05/28/2013	09/15/2013
4	US 80	Dallas	Dallas	Gross Road over U.S. 80	Girder	03/04/2011	05/28/2013	10/27/2013
5	Corinth Street	Dallas	Dallas	Corinth St. over Trinity River	Pier Bent	02/09/2009	05/28/2013	10/27/2013
6	IH 35E	Denton	Dallas	Corinth Street over IH 35E	Girder	03/08/2007	10/02/2013	10/27/2013
7	IH 20	Parker	Fort Worth	CR 470 over IH 20	Column	09/01/2007	07/06/2013	12/19/2013
8	SH 199	Tarrant	Fort Worth	Loop 344 over SH 199	Girder	10/01/2008	07/06/2013	12/19/2013

Bridge Inspection Results:

1. In general, the FRP strengthening itself of the repaired bridges appeared to be in good shape.
2. Some delamination, degradation and air pockets underneath the FRP wrapping were observed.
3. In a couple of cases, improper application of the FRP wrapping was detected.
4. Most of the inspected bridges had damaged girders due to oversized vehicle collision. In some cases, the previous FRP wrapping was damaged due to the impact. In several bridges, concrete spalling and subsequent prestressing strand damage were observed. Repair and FRP strengthening of the damaged elements are highly recommended.
5. Indications of previous FRP pull out testing in some girders and columns were detected, most likely undertaken at the original installation of the FRP wrapping.

4.2 Pull-Off Test Procedure

The following steps were followed for each selected bridge:

As part of field evaluation of FRP strengthened bridge components, a site inspection log was prepared for each bridge, indicating the bridge location, date of inspection, number of components repaired with FRP, total number of spans, lanes to be closed for the pull off test and general remarks on the bridge condition.

Based on the location of the FRP strengthened components in relation to the traffic lanes, lane closures were scheduled for the day of the Pull-Off test.

The Bridge Divisions of the TxDOT Dallas District and Fort Worth District was contacted in order to get the required permissions for performing the lane closures. The lane closure operations were performed by N-Lane Traffic Lines, Inc.

Thereafter, the Pull-Off test was performed according to the specifications of ASTM D7522/D7522M (2009) on selected locations of FRP strengthened girders and columns. The test locations were selected in such a way that the testing surface is free of cracks, voids and pitting, located away from edges, discontinuities and test dolly would be bonded to sound concrete.

A diamond coated hole saw with a diameter of 2 in. was used to create the test surfaces. Figures 7 and 8 shows the drilling process using diamond-coated hole saw and cored test section respectively. The depth of the groove varied, though an equal depth in all samples was desired.

Figure 4-2 Core drilling using diamond core drill

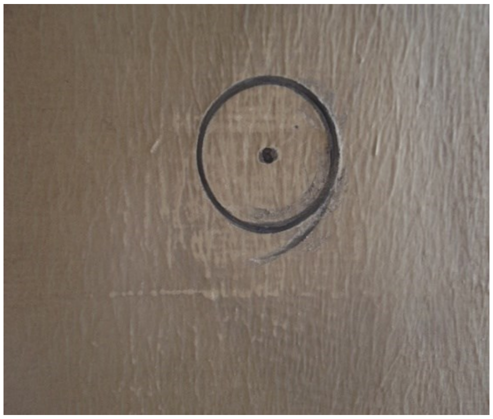


Figure 4-3 Test section after core drilling

After the hole saw had been used to create the test surfaces, each surface was sanded with 120-grit sand paper to remove small surface imperfections. Each test surface was then cleaned with rubbing alcohol and a coarse brush. A two-part high strength epoxy (Loctite), was mixed in a 1:1 ratio and applied evenly over the bottom surface of a single test dolly and test surface. Figures 4-4 and 4-5 shows the cleaning alcohol and Loctite epoxy used for bonding dollies.

Figure 4-4 Alcohol used for cleaning test surface

Figure 4-5 Epoxy used for bonding test dollies

Once both the dollies and CFRP surfaces were prepared and cleaned, the dollies were adhered to the CFRP at the cored locations with epoxy. Figure 4-6 shows the dollies used for Pull-Off test. Bonding of test dollies to test surfaces was done individually because the cure time of the epoxy was only ten minutes.

Figure 4-6 Dollies used for pull off test

Figure 4-7 shows the applied epoxy to the test dollies. The test dolly was placed on the test surface, and pressure was applied manually to remove excess epoxy from between the bonded surfaces. Excess epoxy was removed with a cotton swab. After the test dolly had been held in place on the test surface for two minutes, a glue tape was attached to hold the dolly in place (Figure 4-8).

Figure 4-7 Epoxy applied to testing dollies

Figure 4-8 Glue tape attached to hold the dolly in place

After allowing the Loctite epoxy to cure for 22 hours and the Pull-Off tests were performed on the following day. The central grip of the Positest adhesion tester was connected to the loading fixture, and the force indicator was set to zero. Mechanized loading was applied until a failure occurred, or the maximum capacity of the Positest adhesion tester was reached (Figure 4-11). Figure 4-9 shows the Positest Pull-Off adhesion tester used for pulling the test dollies. If a failure had occurred, the test dolly was removed, put into a bag and labeled. The reading from the tester and a brief description of the failure were recorded for each test (Figure 4-11). The pull off test was performed by Steel Inspectors of Texas, Inc.

Figure 4-9 Positest pull-off adhesion tester

Figure 4-10 Performing pull off test on a loading fixture

Figure 4-11 Failure stress observed after pull off test

4.3 CFRP Repair after Pull-Off Test

After the completion of pull off tests, Sika Dur 31 epoxy was used to fill the residual dents. Figure 4-12 shows the epoxy used for repairing the dents. The epoxy used for the repair was a two part Sika Dur 31, Hi-Mod Gel. One part of component B is mixed with 1 part of component A for 3 minutes with a paddle on low speed (400-600 rpm) drill until uniform in color. Once the two-part epoxy was well mixed, the primer was distributed to the repair areas using 9” rollers. The CFRP fabric was saturated by applying the epoxy to both sides of the fabric and then placed on the dents using hand pressure and rollers. Figure 4-13 shows the repaired sections after re-application of FRP.

Figure 4-12 Epoxy used for reapplication

Figure 4-13 Repaired sections after Pull- Off test

<u>Pull off Testing Apparatus used:</u>

- 2-1/8 inch diameter core drill.
- 2 inch diameter loading fixtures (Dollies).
- Loctite 2 part metal or concrete epoxy glue (3200 Psi) for attaching loading fixture to concrete surface.
- Sika Dur 31 epoxy for bonding the applied FRP on to the test section after test.
- Sika FRP wrap.
- Positest pull-off adhesion tester model AT-A.

Chapter 5

Summary and Discussion of Pull off Test Results

5.1 Pull-Off Test Results

A total of twenty-nine pull-off tests were performed in accordance with ASTM D7522 specifications on the eight selected FRP strengthened bridges. The results of the Pull-Off tests are presented in tables 5-1 to 5-8. Failure modes A, B, E, F, and G as defined in chapter 2 occurred during the Pull-Off testing. Images of the tested samples representing different types of failure modes are shown below.

Bridge 1: SH 183 Over Loop 12

This bridge on SH 183 across Loop 12 is located in Dallas County (Figure 5-2). As shown in Figure 5-1, seven girders (3, 4, 10, 11, 13, 14, 15 and 16) were previously repaired and strengthened with FRP on 11/06/2006. Sample locations of the Pull-Off testing are presented in Table 5-1. The test sections selected were cored on 09/14/2013. Failure of core occurred in sample 4 while core drilling (Figure 5-3). Dollies were glued in place on top of the cored areas. Temperature was around 90° F and humidity was approximately 40% when the dollies were attached. The dollies were pulled on 09/15/2013.The different types of failure modes observed after Pull-Off test are shown in Figure 5-4. The results of the Pull-Off test are presented in Table 5-1.

Table 5-1 Test Results, SH 183 over loop 12 (Bridge 1)

Sample Number	Location of Sample	Failure Stress (psi)	Failure mode per ASTM D7522
1	Girder 3, north face of bottom flange	159	F (98% E, 2% G)
2	Girder 4, north face of 45 degree chamfered portion	26	A*
3	Girder 10, north face of bottom flange	261	70% G, 30% C
4	Girder 11, south face of web	-	Core broke off in concrete while drilling
5	Girder 14, south face of bottom flange	254	10% B, 40% top coat cohesion and 50% FRP top layer to top coat adhesion

*Glue mixed incorrectly, not cured

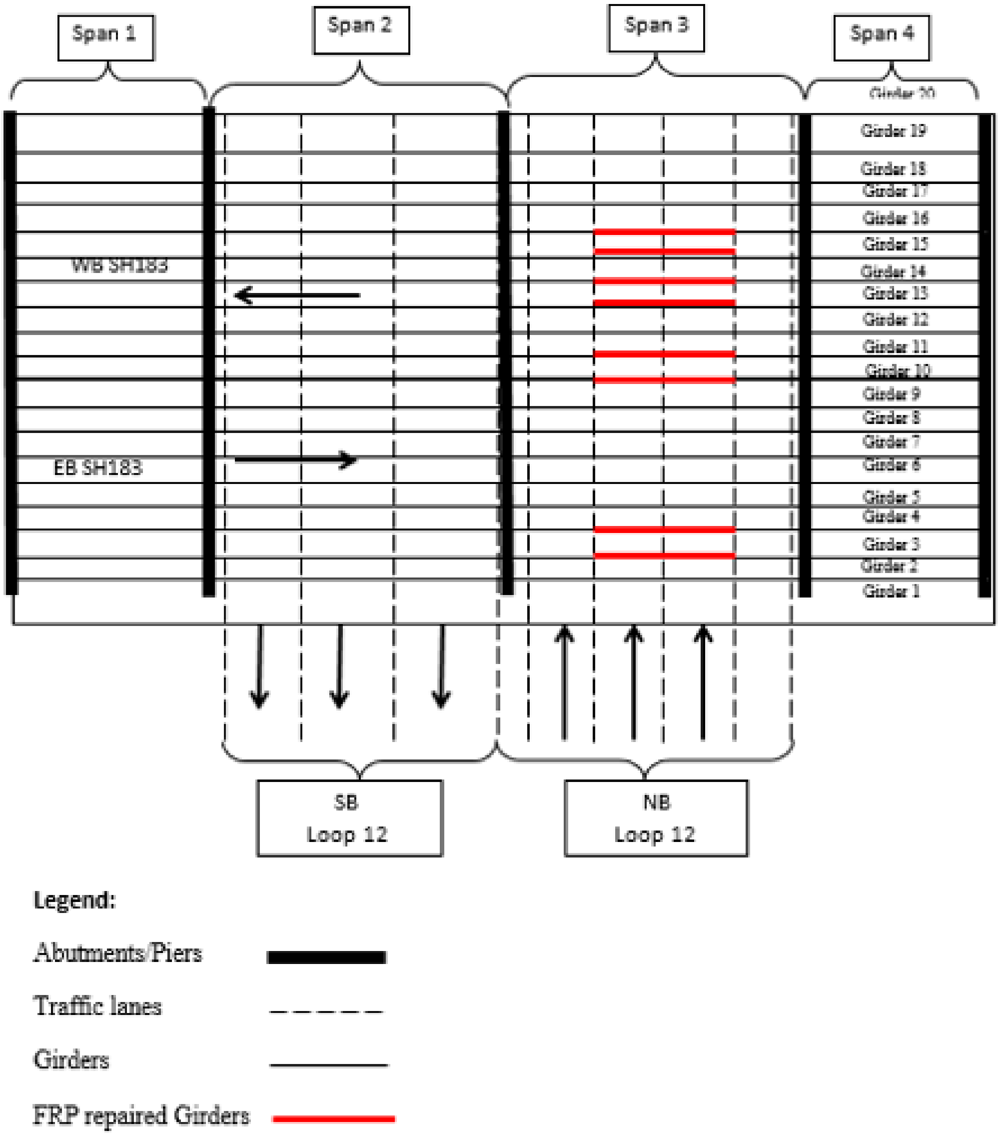


Figure 5-1 FRP strengthened girders in Bridge 1

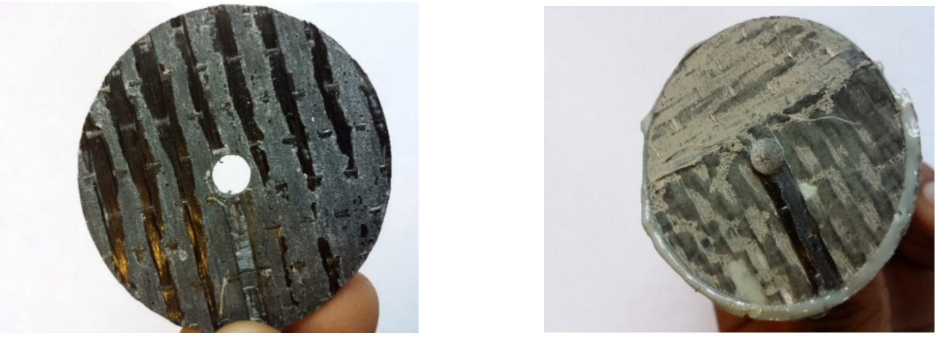
Figure 5-2 Bridge 1 profile

Figure 5-3 Core failure in sample 4 after drilling

(a) Failure Mode F in sample 1

(b) Failure Mode A in sample 2

(c) Failure Modes G and C in sample 3



(d) Core failure in sample 4

(e) Mixed failure mode in sample 5

Figure 5-4 Failed samples, Bridge 1

Bridge 2: Loop 12 over Irving Blvd

This bridge on Loop 12 across Irving Blvd. is located in Dallas County (Figure 5-6). As shown in Figure 5-5, girder 4 was previously repaired and strengthened with FRP on 07/14/2011. Locations of the Pull-Off testing are presented in Table 5-2. The test sections selected for Pull-Off strength test were cored on 09/14/2013 (Figure 5-7). Dollies were glued in place on top of the cored areas. Temperature was around 90° F and humidity was approximately 40% when the dollies were attached. The dollies were pulled on 09/15/2013 (Figure 5-8). The different types of failure modes observed after Pull-Off test are shown in Figure 5-9. The results of the Pull-Off test are presented in Table 5-2.

Table 5-2 Test Results, Loop 12 over Irving Blvd (Bridge 2)

Sample Number	Location of Sample	Failure Stress (psi)	Failure mode per ASTM D7522
1	Girder 4, South face of web	352	100 % G
2	Girder 4, North face of bottom flange	281	A*

* Test dolly came off glue at 281 psi. When area was cleaned for repair, it was found that the glue/FRP piece was loose and the entire 2 in. circle came off in one piece at concrete level.

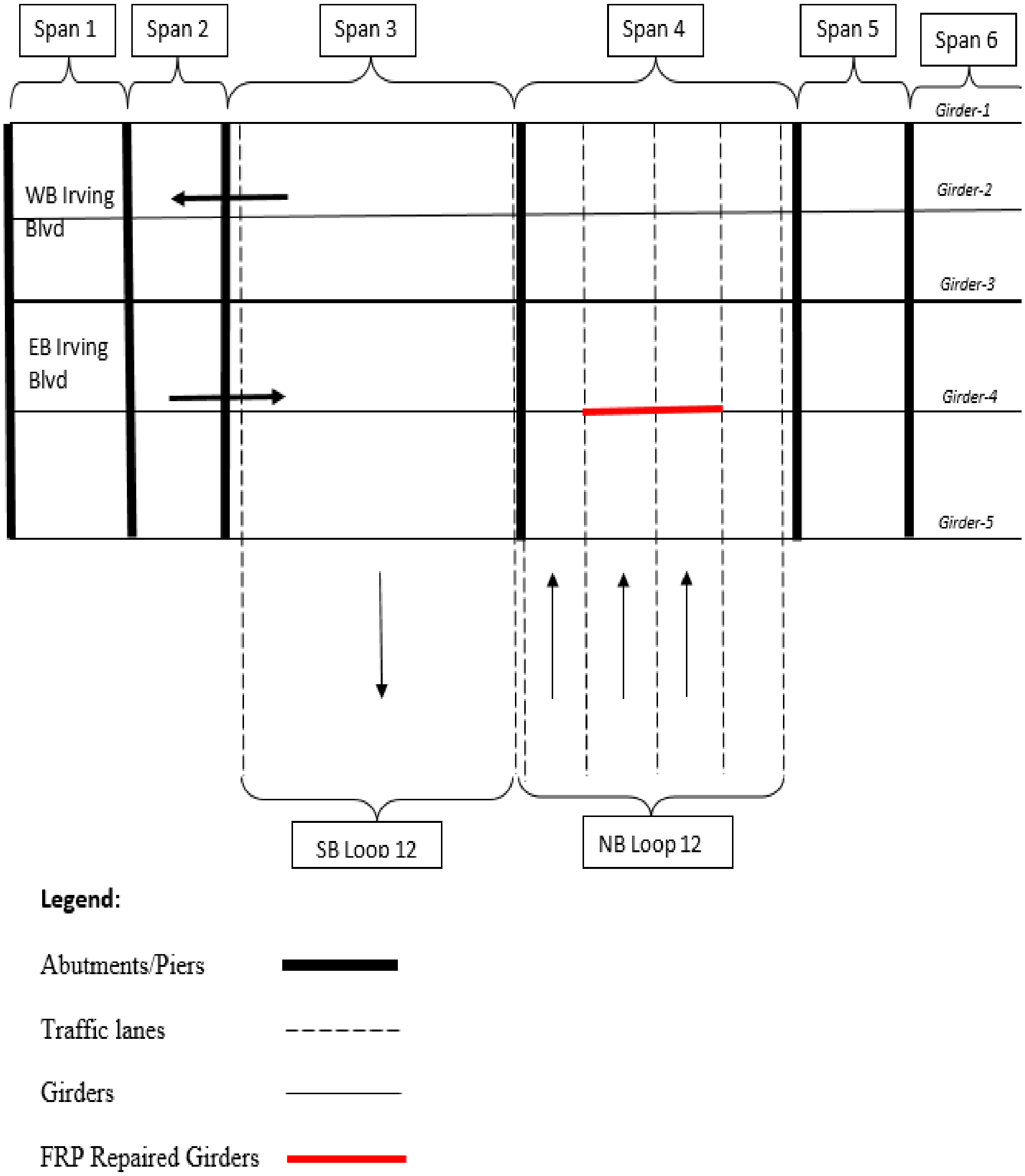


Figure 5-5 FRP repaired girders in Bridge 2

Figure 5-6 Bridge 2 Profile

Figure 5-7 Core drilling on girder 4 (sample 1), Bridge 2

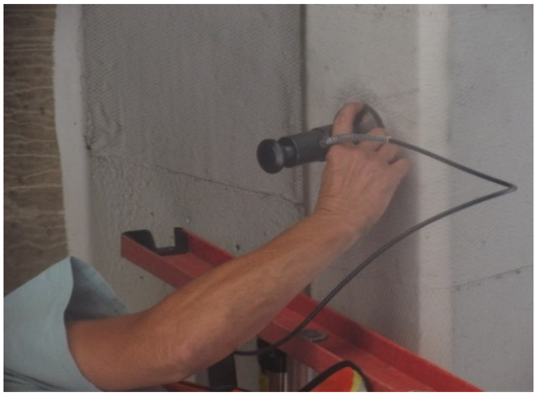
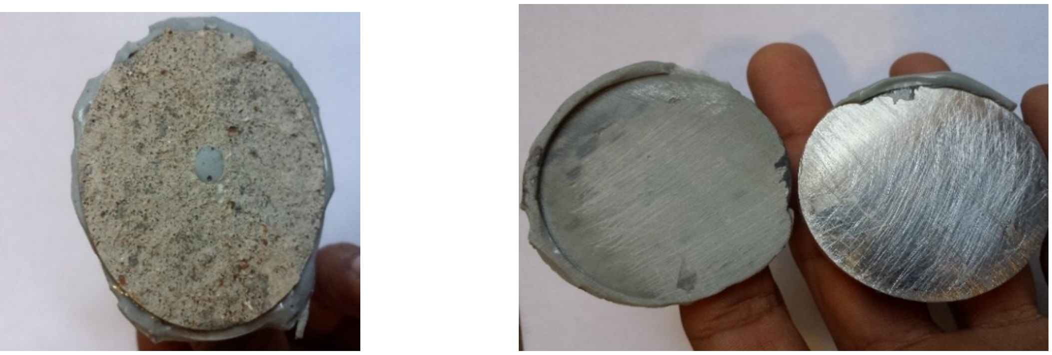


Figure 5-8 Pull-Off test on girder 4 (sample 2), Bridge 2



(a) Failure Mode G in sample 1

(b) Failure Mode A in sample 2

Figure 5-9 Failed samples, Bridge 2

Bridge 3: SH 183 over MacArthur Blvd

This bridge on SH 183 across MacArthur Blvd. is located in Dallas County (Figure 5-11). As shown in Figure 5-10, four girders (1, 2, 3 and 4) and three columns were previously repaired and strengthened with FRP on 12/21/2005. Locations of the Pull-Off testing are presented in Table 5-3. The test sections selected for Pull-Off strength test were cored on 09/14/2013(Figure 5-12). Dollies were glued in place on top of the cored areas. The temperature was around 90° F and humidity was approximately 40% when the dollies were attached. The dollies were pulled on 09/15/2013 (Figure 5-13). Different types of failure modes observed after Pull-Off test are shown in Figure 5-14. The results of the Pull-Off test are presented in Table 5-3.

Table 5-3 Test Results, SH 183 over MacArthur Blvd (Bridge 3)

Sample Number	Location of Sample	Failure Stress (psi)	Failure mode per ASTM D7522
1	Column 1, Southeast face (4 ft. above ground level)	261	25% B, 75% top coat cohesion
2	Column 2, Northwest face (4 ft. above ground level)	177	100% G
3	Column 3, Southwest face (4 ft. above ground level)	N/A	During drilling process core broke off in concrete.
4	Column 3, Northwest side (5 ft. above ground level)	271	100% G
5	Girder 1, South face of web	37	100% C
6	Girder 1, Bottom face	158	F(90% G, 10% E)
7	Girder 2, North Face of web	451	F (80% E, 10% G and 10% top coat to FRP adhesion)

Table 5-3—Continued

Sample Number	Location of Sample	Failure Stress (psi)	Failure mode per ASTM D7522
8	Girder 4, North face of web	364	100% G
9	Girder 5, Bottom face	323	100% G

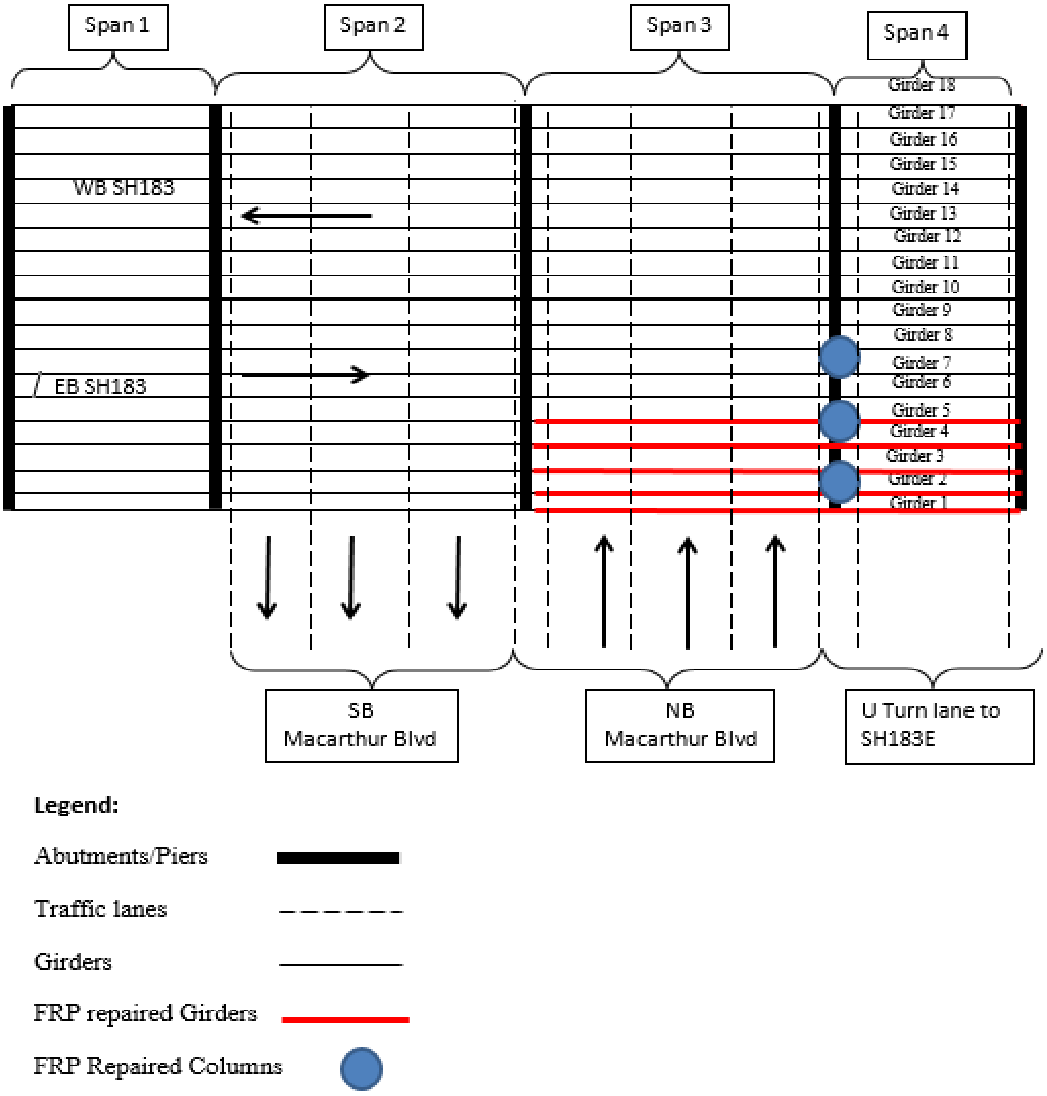


Figure 5-10 Repaired components in Bridge 3

Figure 5-11 Bridge 3 profile

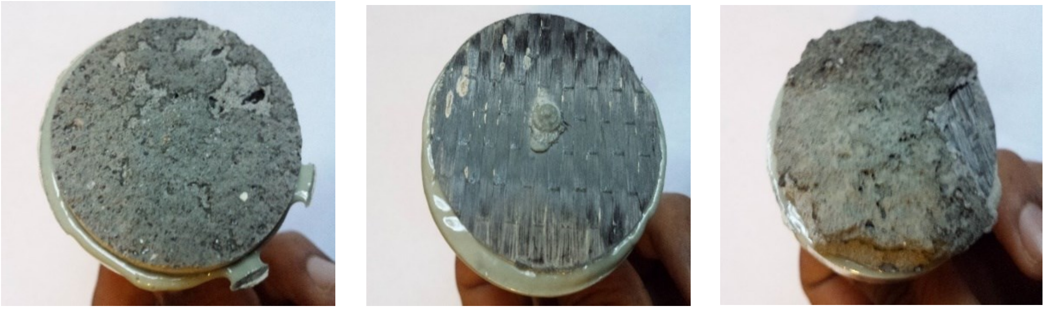
Figure 5-12 Core drilling (sample 1) in column 1, Bridge 3

Figure 5-13 Pull-Off test on column 1 (sample 2), Bridge 3

(a) Mixed failure Mode B in sample 1

(b) Failure Mode G in sample 2

(c) Failure of core in sample 3



(d) Failure Mode G in sample 4

(e) Failure Mode C in sample 5

(f) Failure Mode F in sample 6

(g) Failure Modes F in sample 7

(h) Failure Mode G in sample 8

(i) Failure Mode G in sample 9

Figure 5-14 Failed samples, Bridge 3

Bridge 4: Gross Road over U.S. 80

This bridge over highway U.S. 80 is located in Dallas County (Figure 5-16). As shown in Figure 5-15, girder 3 on span 3 and girders 3/4 on span 4 were previously strengthened with FRP on 03/04/2011. Locations of the Pull-Off testing are presented in Table 5-4. The test sections selected for Pull-Off strength test were cored on 10/26/2013. Dollies were glued in place on top of the cored areas .Temperature was around 60° F and humidity was approximately 70% when the dollies were attached. The dollies were pulled on 10/27/2013 (Figure 5-17). Different types of failure modes observed after Pull-Off test are shown in Figure 5-18. The results of the Pull-Off test are presented in Table 5-4.

Table 5-4 Test Results, Gross Road over U.S. 80 (Bridge 4)

Sample Number	Location of Sample	Failure Stress (psi)	Failure mode per ASTM D7522
1	Girder 3, West face of bottom flange	352	100 % G
2	Girder 3, Bottom face	478	C (1%), G (99%)

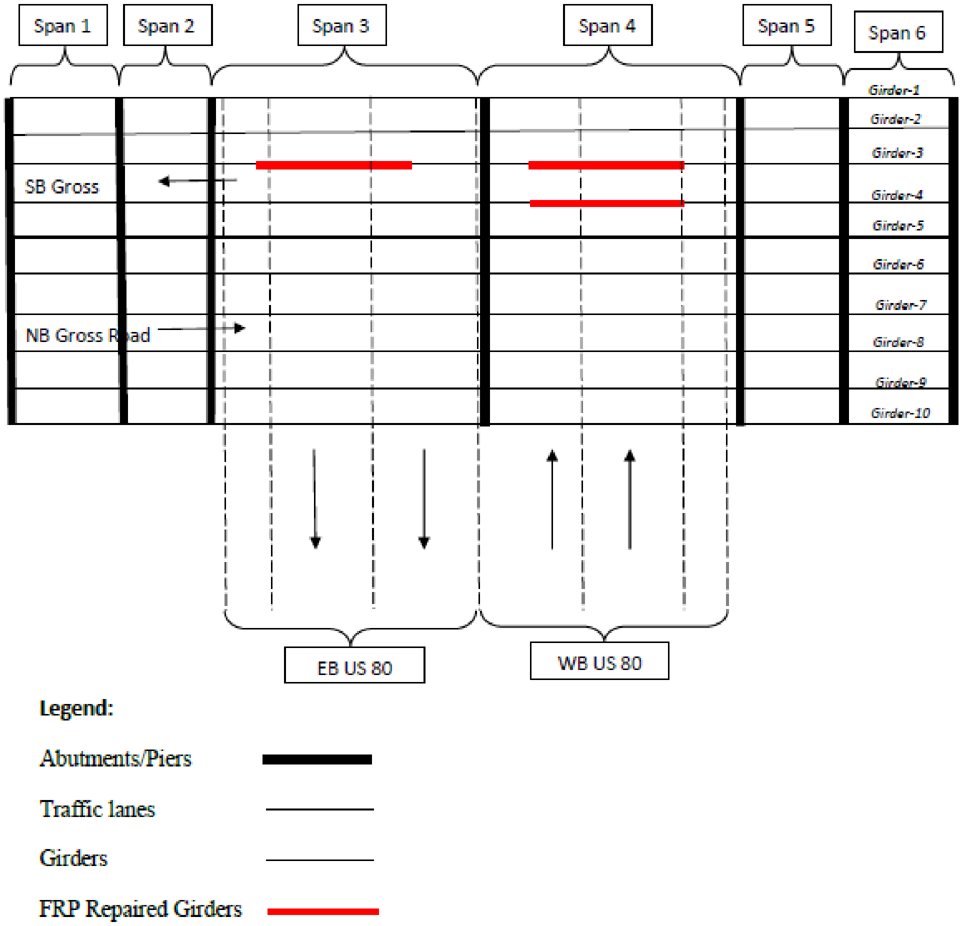


Figure 5-15 Repaired girders in Bridge 4

Figure 5-16 Bridge 4 profile

Figure 5-17 Pull-Off test on (sample 1), girder 3, Bridge 4

(a) Failure Mode G in sample 1

(b) Failure Mode G in sample 2

Figure 5-18 Failed samples, Bridge 4

Bridge 5: Corinth St. over Trinity River

This bridge is located on the Corinth Street overpass over Trinity River near downtown Dallas in the Dallas County (Figure 5-19). Only one FRP strengthening was found on this bridge, a small section of the pier cap on the second bent coming from the side of Rock Island St. was previously strengthened with FRP on 02/09/2009. Locations of the Pull-Off testing are presented in Table 5-5. The test sections selected for Pull-Off strength test were cored on 10/26/2013 (Figure 5-20). Dollies were glued in place on top of the cored areas. Temperature was around 60° F and humidity was approximately 70% when the dollies were attached. The dollies were pulled on 10/27/2013 (Figure 5-21). Different types of failure modes observed after Pull-Off test (Figure 5-22).The results of the Pull-Off test are presented in Table 5-5.

Table 5-5 Test Results, Corinth St. over Trinity River (Bridge 5)

Sample Number	Location of Sample	Failure Stress (psi)	Failure mode per ASTM D7522
1	North face of pier cap	336	C (12%), G (88%)
2	South face of pier cap	372	F (5% E , 95%G)

Figure 5-19 Bridge 5 profile

Figure 5-20 Core drilling on pier cap (sample 1), Bridge 5

Figure 5-21 Pull-Off test on pier cap (sample 1), Bridge 5

(a) Failure Mode G in sample 1

(b) Failure Mode G in sample 2

Figure 5-22 Failed samples, Bridge 5

Bridge 6: Corinth St. over IH 35E

This bridge is located on IH-35E in Denton County (Figure 5-24). As shown in Figure 5-23, on IH 35E North span 3, Girders 2, 3, 4, and 5 were repaired, and on IH 35E south span 2, Girders 1 and 2 were repaired previously and strengthened with FRP on 03/08/2007. Locations of the Pull-Off testing are presented in Table 5-6. The test sections selected for Pull-Off strength test were cored on 10/26/2013 (Figure 5-25). Dollies were glued in place on top of the cored areas. Temperature was around 60° F and humidity was approximately 70% when the dollies were attached. The dollies were pulled on 10/27/2013. Different types of failure modes observed after Pull-Off test are shown in Figure 5-26.The results of the Pull-Off test are presented in Table 5-6.

Table 5-6 Test Results, Corinth St. Over IH 35E (Bridge 6)

Sample Number	Location of Sample	Failure Stress (psi)	Failure modes per ASTM D7522
1	Girder 2, South face of web	416	A (40% Glue to dolly, 5% FRP finish coat adhesion, 55% glue to finish coat adhesion)
2	Girder 4, South face of bottom flange	239	F (20% G, 80% E at grout to concrete adhesion)
3	Girder 4, Bottom face	-	Core broke off in concrete while drilling
4	Girder 4, Bottom face	278	100 % G

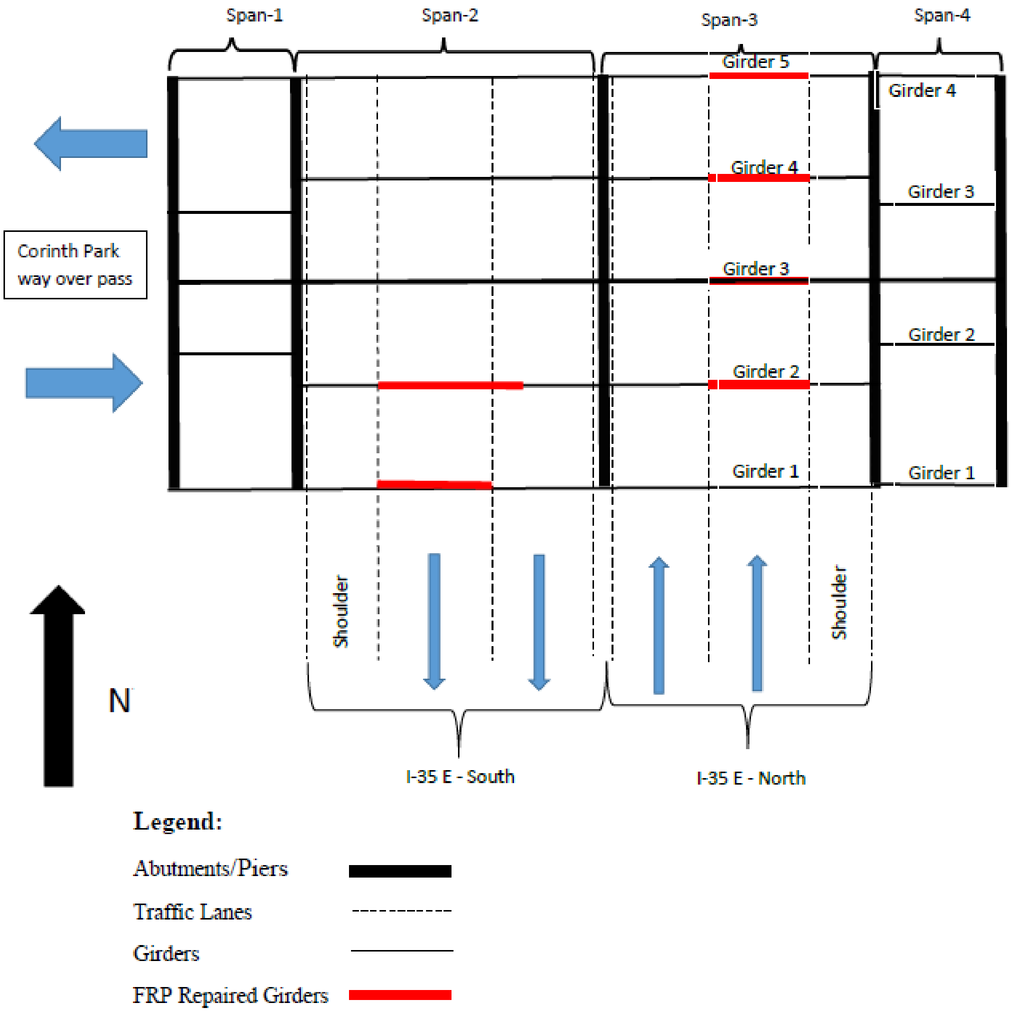


Figure 5-23 Repaired girders in Bridge 6

Figure 5-24 Bridge 6 profile

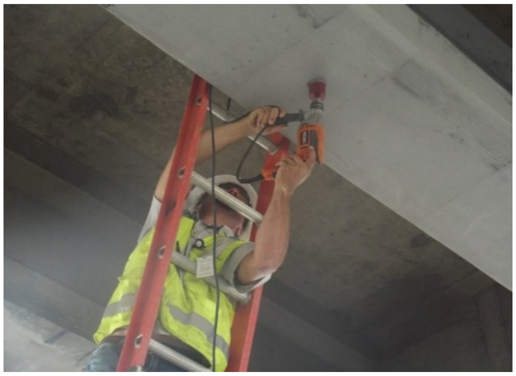


Figure 5-25 Core drilling (sample 3) in girder 4, Bridge 6

(a) Failure Mode A in sample 1

(b) Failure Modes F in sample 2

(c) Failure of core in sample 3

(d) Failure Mode G in sample 4

Figure 5-26 Failed samples, Bridge 6

Bridge 7: CR 470 over IH 20

This bridge is located in Parker County (Figure 5-28). As shown in Figure 5-27, only one column was repaired previously and strengthened with FRP on 09/01/2007. Locations of the Pull-Off testing are presented in Table 5-7. The test sections selected for Pull-Off strength test were cored on 12/17/2013 (Figure 5-29). Dollies were glued in place on top of the cored areas. Temperature was around 60° F and humidity was approximately 40% when the dollies were attached. The dollies were pulled on 12/18/2013 (Figures 5-30). Different types of failure modes observed after Pull-Off test are shown in Figure 5-31.The results of the Pull-Off test are presented in Table 5-7.

Table 5-7 Test Results, CR 470 over IH 20, (Bridge 7)

Sample Number	Location of Sample	Failure Stress (psi)	Failure mode per ASTM D7522
1	Top of 1st column	156	100% G
2	Middle of 1st column	290	100% G
3	Bottom of 1st column	294	100% G

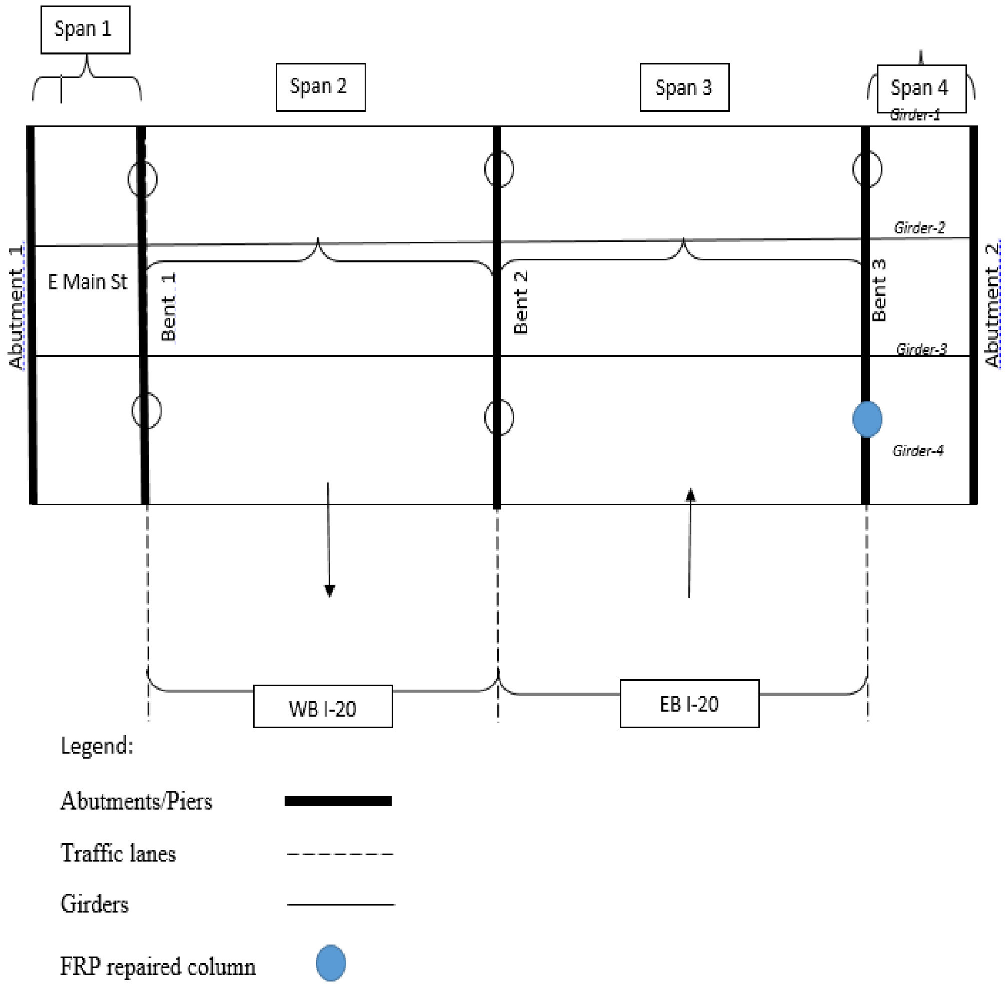


Figure 5-27 Repaired components in Bridge 7

Figure 5-28 Bridge 7 Profile

Figure 5-29 Core drilling (sample 3) in column 1, Bridge 7

Figure 5-30 Pull-Off test on sample 1 in column 1, Bridge 7

(a) Failure Mode G in sample 1

(b) Failure Mode G in sample 2

(c) Failure Mode G in sample 3

Figure 5-31 Failed samples, Bridge 7

Bridge 8: Loop 344 over SH 199

This bridge is located in Tarrant County (Figure 5-33). As shown in Figure 5-32, only one girder (#2) was repaired previously and strengthened with FRP on 10/01/2008. Locations of the Pull-Off testing are presented in Table 5-8. The test sections selected for Pull-Off strength test were cored on 12/17/2013. Dollies were glued in place on top of the cored areas. Temperature was around 60° F and humidity was approximately 40% when the dollies were attached. The dollies were pulled on 12/18/2013 (Figure 5-34). At the time of Pull-Off test on the dollies, the gauge stopped at maximum Pull-Off pressure (480 psi) of the adhesion tester without any rupture (Figure 5-35). The dollies were detached manually by gentle hammering. Different types of failure modes observed after Pull-Off test are shown in Figure 5-36. The results of the Pull-Off test are presented in Table 5-8.

Table 5-8 Test Results, Loop 344 over SH 199 (Bridge 8)

Sample Number	Location of Sample	Failure Stress (psi)	Failure mode per ASTM D7522
1	Girder 2, East face of the web	480	G*
2	Girder 2, Bottom face	480	G*

* Gauge stopped at maximum Pull-Off pressure of the adhesion tester for 50 mm dolly.

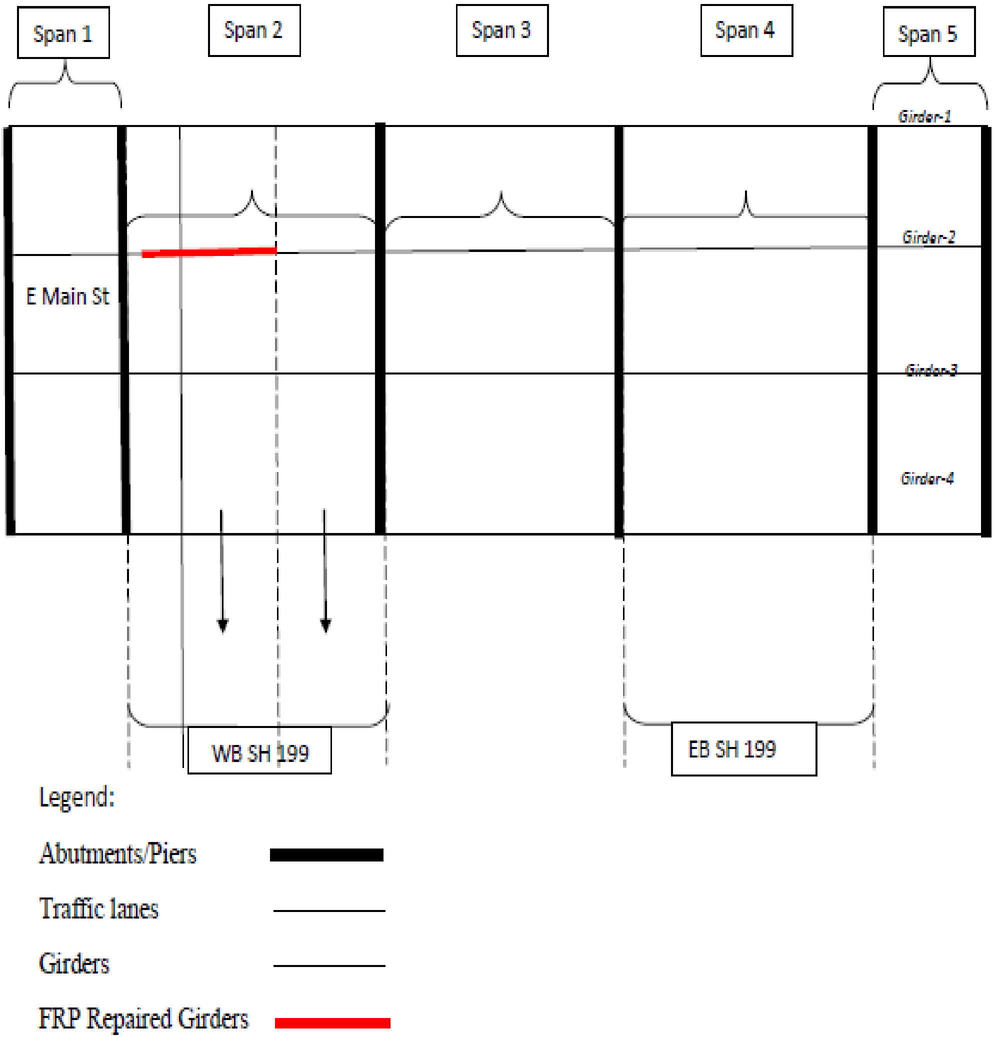


Figure 5-32 Repaired girder in Bridge 8

Figure 5-33 Bridge 8 Profile

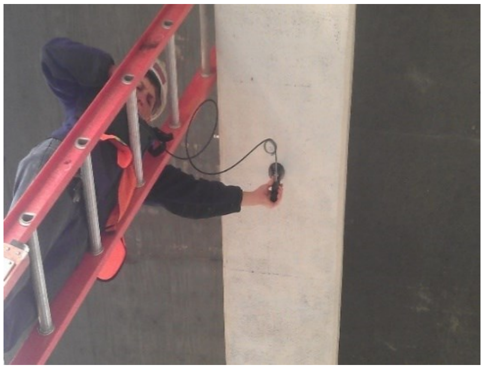


Figure 5-34 Pull-Off test on sample 2 of girder 2, Bridge 8

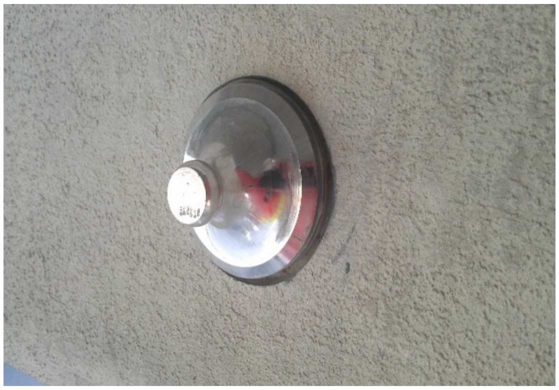


Figure 5-35 Unfailed sample 2 after reaching maximum pressure of the tester, Bridge 8

(a) Failure Mode G in sample 1

(b) Failure Mode G in sample 2

Figure 5-36 Failed samples, Bridge 8

5.2 Discussion of Results

A total of 29 test sample sections were selected for the Pull-Off test from various FRP strengthened sections on the eight selected bridges. Three of the samples failed in the core while scouring the FRP laminate before attaching the dollies. The summary of Pull-Off test results is presented in Figure 5-37 and Table 5-9.

Table 5-9 Summary of Pull-Off strength test results

Sample Number	Bridge	Location of Sample	Failure Stress (psi)	Failure Mode
1	Bridge 1	Girder	159	F (98% E, 2% G)
2	Bridge 1	Girder	26	A*
3	Bridge 1	Girder	261	30% C, 70% G
4	Bridge 1	Girder	0	Core broke off
5	Bridge 1	Girder	254	10% B, 40% top coat cohesion and 50% FRP top layer to top coat adhesion
1	Bridge 2	Girder	352	100 % G
2	Bridge 2	Girder	281	100 % A
1	Bridge 3	Column	261	25% B, 75% top coat cohesion
2	Bridge 3	Column	177	100% G
3	Bridge 3	Column	0	Core broke off
4	Bridge 3	Column	271	100% G
5	Bridge 3	Girder	37	100% C
6	Bridge 3	Girder	158	F (10% E, 90% G)
7	Bridge 3	Girder	451	F (80% E, 10% G and 10% top coat to FRP adhesion)
8	Bridge 3	Girder	364	100% G
9	Bridge 3	Girder	323	100% G
1	Bridge 4	Girder	352	100% G
2	Bridge 4	Girder	478	(1%) C, (99%) G
1	Bridge 5	Pier Cap	336	(12%) C, (88%) G
2	Bridge 5	Pier Cap	372	F (5% E, 95%G)
1	Bridge 6	Girder	416	A (40% Glue to dolly, 5% FRP finish coat adhesion, 55% glue to finish coat adhesion)
2	Bridge 6	Girder	239	F (20% G, 80% E at grout to concrete adhesion)
3	Bridge 6	Girder	0	Core broke off
4	Bridge 6	Girder	278	100 % G

Table 5-9 continued

Sample Number	Bridge	Location of Sample	Failure Stress (psi)	Failure Mode
1	Bridge 7	Column	156	100 % G
2	Bridge 7	Column	290	100 % G
3	Bridge 7	Column	294	100 % G
1	Bridge 8	Girder	480	100 % G*
2	Bridge 8	Girder	480	100 % G*

A* Glue mixed incorrectly, not cured.
G* Gauge stopped at maximum Pull-Off pressure of the adhesion tester for 2 inch dolly.

The predominant failure mode observed was a cohesive concrete failure, also known as Mode G per ASTM D7522, in 14 samples (53.84%) out of 26 total samples. The mean and standard deviation of the Pull-Off test results are 290.23 psi and 120.51 psi respectively. A considerable amount of concrete remained bonded to the test dolly after failure in these samples. The amount of concrete that remained bonded to the test dollies varied. Strengths ranged from as low as 26 psi to as high as 480 psi. Large variations in the bond strength results were witnessed. In two of the samples, failure Mode A (bonding epoxy failure at loading fixture) was observed. Five test samples failed in Mode F (mixed cohesive failure in the substrate and epoxy failure at the epoxy/substrate interface). Four samples failed in mixed failure modes (i.e. combination of two or more failure modes). The concrete bonded to the test dollies was spotty, very thin, and contained no coarse aggregate.

Possible reasons that may have caused non-G type or mixed type of failures are as follows:

a. Improper application of FRP strengthening at the time of repair.
b. Experimental errors (Misalignment between the circular loading fixture and the adhesion tester, improper surface preparation and improper curing of bonding epoxy).

c. Application of FRP strengthening at a non-standard temperature or relative humidity.
d. Degradation of bond strength between CFRP laminate and concrete substrate over time due to environmental conditions.
e. Reduction in the FRP quality due to aging.
f. Improper preparation of the testing surface, such as the presence of moisture as well as torsional and thermal stresses applied during the core drilling process, and variations in the depth of the core cut.

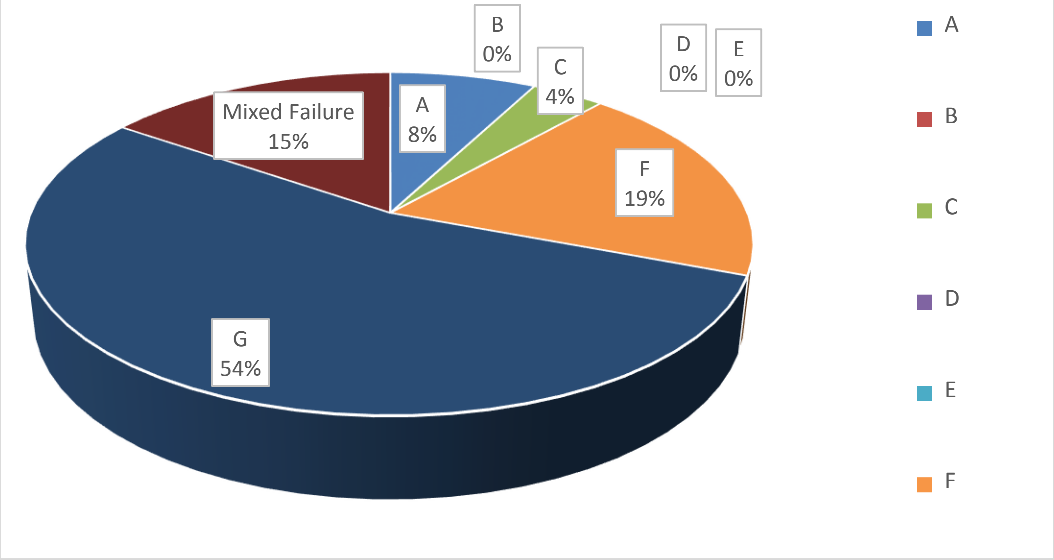


Figure 5-37 ASTM Failure modes from Pull-Off tests

None of the five test samples in Bridge 1 failed in Mode G and the failure modes varied from A through F, which suggest that the bond between FRP and concrete substrate either weakened with time due to various factors, or improper application of FRP. Apart from Bridge 1, majority of samples in all other bridges failed in the desirable Mode G.

However, the Pull-Off strengths were low in some of the samples, indicating the presence of damage to the FRP-concrete bond. The minimum failure stress specified for a Pull-Off test is 200 psi (Master Builders, 1993), and 23% (6 of 26) of the test results fell below this level. In Bridge 8, the gauge of the adhesion tester reached the maximum capacity (480 psi) before any failure. The samples were detached manually by gentle hammering. The bridge was not very old, and the bond between FRP and concrete was possibly strong.

5.2.1 Column results

A total of seven samples from various locations of FRP strengthened columns were tested (Table 5-9). One of the samples from Bridge 3 failed in core during the scoring process and could not be evaluated. Strengths after Pull-Off test ranged between 156 psi and 294 psi. The mean and standard deviation of the Pull-Off test results are 240.5 psi and 59.71 psi respectively. The dominant failure mode observed in the majority of samples (5) was type G (83.3%). The other sample failed in mixed mode. The results of Pull-Off tests on columns are presented in Figure 5-38.

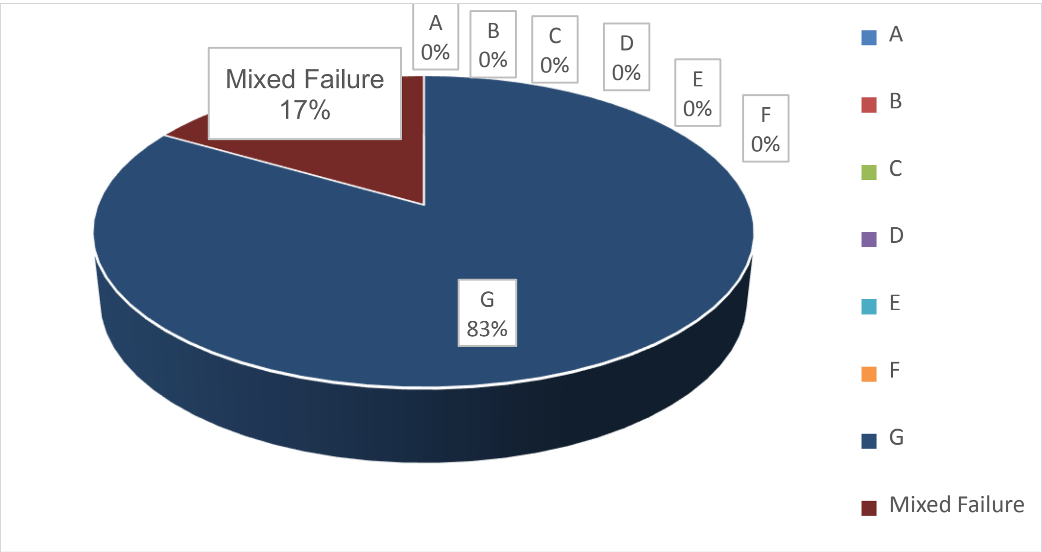


Figure 5-38 Failure modes in FRP strengthened columns

Large variation in the bond strength was witnessed. For example, for Bridge 7, failure stress of sample 1 was 156 psi, whereas the failure stresses of other two samples are 290 psi and 294 psi. It is worth noting that all three pull-off tests showed the desirable Mode G failure. Likewise in Bridge 3, sample 1 failed in mixed mode (A and B) with a failure stress of 261 psi, whereas sample 3 failed in Mode G type with failure stress of 177 psi. The reasons for this inconsistent failure stress pattern might be the improper application of FRP at the time of repair and variations in the depth of core cut before attaching dollies, which in turn might have affected the Pull-Off test failure stress.

5.2.2 Girder Results

A total of 22 samples were tested from various locations on FRP strengthened girders. Two samples failed in the core during the scoring operation. The pre-dominant failure mode observed was type G (44%). The results were scattered, with failure modes varying from Mode A through Mode G. Pull-Off strengths ranged between 26 psi to 480 psi. The mean and standard deviation of the Pull-Off test results are 299 psi and 140 psi, respectively. The results are presented in Figure 5-39. The high standard deviation shows the large scatter in the obtained results. Unlike the slight variations in failure stress patterns observed in columns, large variations in the failure stresses for samples failing in the similar mode was witnessed.

The percentage of samples failing in Mode G (44%) was less, compared to columns (83%). The probable reasons for this are:

a. Improper application of FRP. Application of FRP on girder surfaces is more difficult (due to various sides and accessibility issues) than application on columns.
b. The depth of core cut prior to attachment of dollies was not uniform in all the samples which might have affected the failure stress.

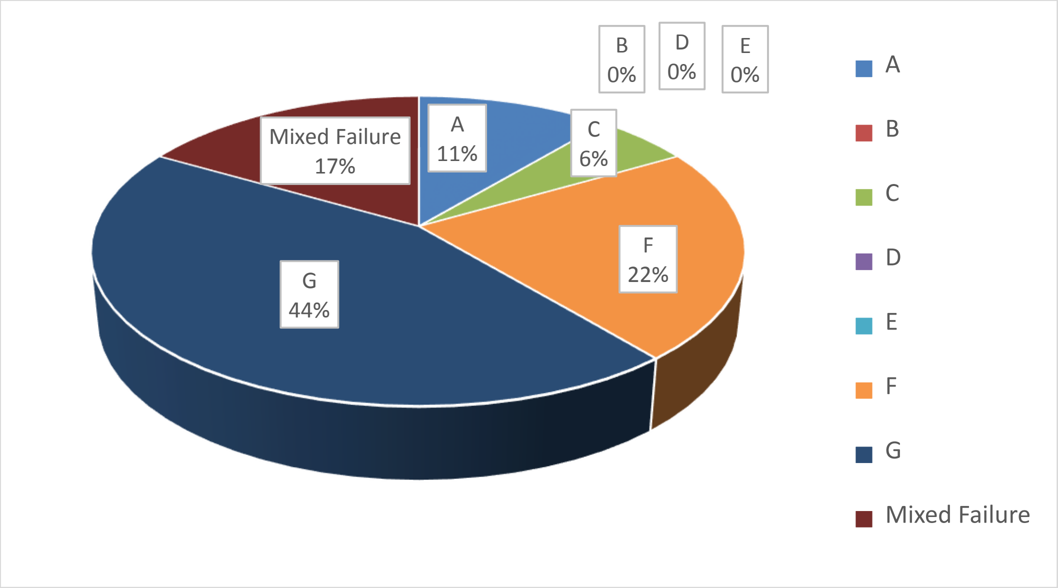


Figure 5-39 Failure modes in FRP strengthened girders

5.2.3 Column vs. Girder Results

To understand the behavior of FRP strengthening with respect to its location, Pull-Off test results from girders and columns in Bridge 3 having the same FRP age was investigated. The results shown in Table 5-10 indicate that the Pull-Off strengths from columns were lower than those from the girders at the same FRP age. The strength of the concrete substrate plays a major role in the bond strength of FRP-concrete system. Due to the controlled manufacturing conditions at the precast site and also the likelihood of higher concrete strengths, precast girders may have more strength as compared to the columns cast in place.

Table 5-10 Pull-Off test results from Bridge 3

Sample Number	Location of sample	Failure Stress (psi)	Average Failure Stress (psi)	Failure Mode
1	Column	261	236	25% B, 75% top coat cohesion
2	Column	177		100% G
3	Column	0		Core broke off
4	Column	271		100% G
5	Girder	37	266	100% C
6	Girder	158		F (10% E, 90% G)
7	Girder	451		F (80% E, 10% G and 10% top coat to FRP adhesion)
8	Girder	364		100% G
9	Girder	323		100% G

The failure stresses for CFRP strengthened Girders and Columns in Bridge 3 are plotted in Figure 5-40.

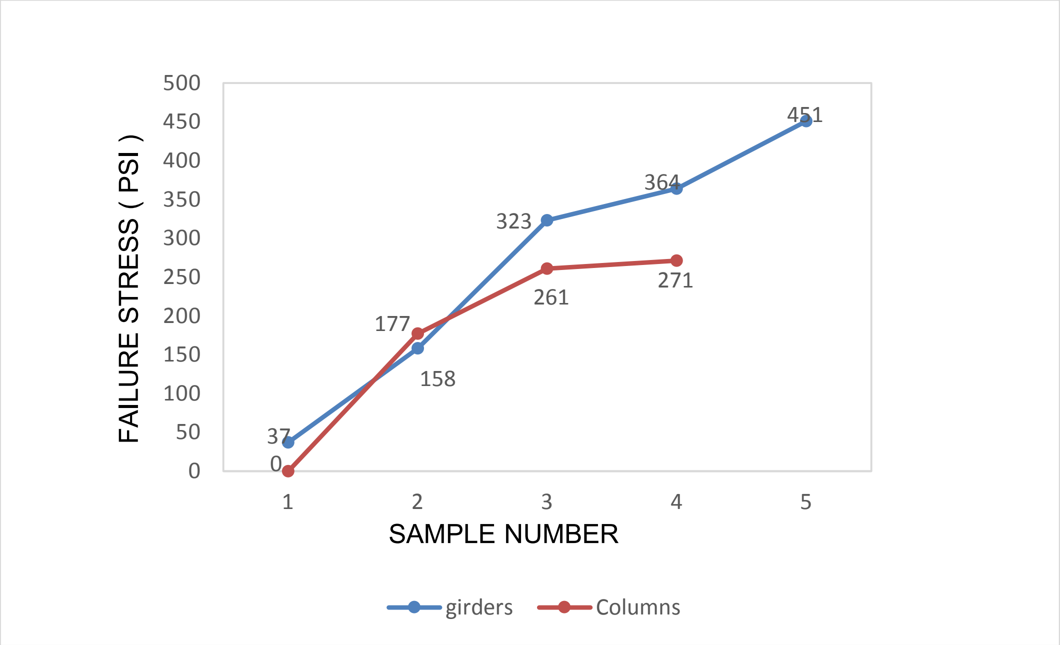


Figure 5-40 Variation of failure stresses in girders and columns, Bridge 3

5.2.4 Location on girder

Out of 13 samples taken at the face of girder web, five samples (38.46%) failed in Mode G. The results were scattered, with failure modes varying from Mode A through Mode G. Pull-Off strengths ranged between 26 psi and 480 psi. Five samples were taken at the bottom face of girders, of which four samples (80%) failed in mode G. Pull-Off strengths ranged between 158 psi to 480 psi. This indicates that the FRP-concrete bond strength is higher at the bottom face of the girder as compared to the web face. Relative difficulty of installing FRP wrapping on the chamfered girder sides may be a reason for this discrepancy. The comparison results are presented in Table 5-11.

Table 5-11 FRP failure modes in relation to location of test section

Mode of Failure	Location of sample on girder	
	Web face	Bottom face
A	2	0
B	0	0
C	1	0
D	0	0
E	0	0
F	2	1
G	5	4
Mixed Failure	3	0
Total	13	5

5.2.5 Humidity and Temperature

Previous experimental research on the effect of humidity and temperature on bond strength of FRP concluded that the bond strength of FRP is affected by the temperature and humidity prevailing at the time of FRP repair work. The conditions at the time of testing do not affect FRP Pull-Off strength. An attempt was made to locate the ambient conditions data at the time of the initial installation of the FRP wrapping on the eight bridges; however, no data was available from the contractors.

5.2.6 Age Effect

The performance of the FRP strengthening in relation to age of FRP was determined by studying its relationship with failure stresses. The results are presented in Table 5-12. Some important observations are:

a. It is evident that, with the increase of FRP age, the bond strength decreased, and the mode of failure changed from G to other types. The strength values for all Mode G failures are plotted as a function of FRP age in Figure 5-41. The best fit trend line is also plotted, and the trend line equation is shown (x = age, y = strength). The equation can be conveniently used to predict the Pull-Off strength of FRP strengthening on concrete.

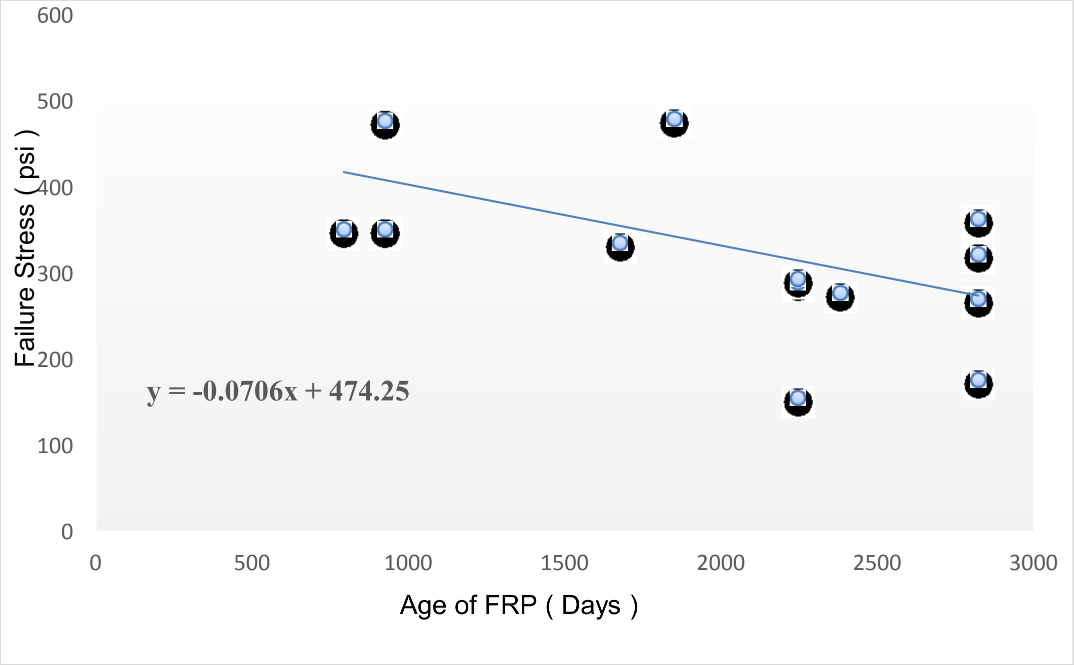


Figure 5-41 Relationship between FRP age and strength in Mode G type of failure

b. Failure stress of sample with FRP age of 926 days was observed to be 478 psi whereas the sample with FRP age of 2825 days was only 177 psi. A decrease of 62.9% in the failure strength of FRP is observed. A possible explanation for this reduction is the degradation of bond strength between CFRP laminate and concrete substrate over time due to the effect of environmental conditions.

c. It is observed that a sample in Bridge 2 with a FRP age 794 days and sample in Bridge 4 with a FRP age of 926 days failed in Mode G and the observed failure stresses are 352 and 478 days, respectively. This result suggests the high bond strength of FRP concrete substrate in the early stages.

d. In the case of Bridge 3 and Bridge 1 with FRP ages of 2825 and 2504 days, respectively, different modes of failure A through G were witnessed. These variations in the Pull-Off test failure modes could be due to the degradation of FRP–concrete bond with time.

Table 5-12 Variation of bond strength with FRP age

Sample Number	Age of FRP (Days)	Failure stress (psi)	Mode of failure
SH 183 over Loop 12 (Bridge 1)			
1	2504	159	F (98% E, 2% G)
2	2504	26	100% A
3	2504	261	Mixed
4	2504	-	Core broke off
5	2504	254	Mixed
Loop 12 over Irving Blvd. (Bridge 2)			
1	794	352	100% G
2	794	281	100% A
SH 183 over MacArthur Blvd. (Bridge 3)			
1	2825	261	25% B, 75% top coat cohesion
2	2825	177	100% G

Table 5-12 continued

3	2825	-	Core broke off
4	2825	271	100% G
5	2825	37	100% C
6	2825	158	F (10% E, 90% G)
7	2825	451	F (80% E, 10% G and 10% A)
8	2825	364	100% G
9	2825	323	100% G
Gross Road over U.S. 80 (Bridge 4)			
1	926	352	100% G
2	926	478	(1%) C, (99%) G
Corinth St. over Trinity River (Bridge 5)			
1	1679	336	F (5% E,95%G)
2	1679	372	100% G
Corinth St. Bridge over IH 35E (Bridge 6)			
1	2383	416	Mixed
2	2383	239	F (20% G, 80%E)
3	2383	-	Core broke off
4	2383	278	100 % G
CR 470 over IH 20, (Bridge 7)			
1	2248	156	100% G
2	2248	290	100% G
3	2248	294	100% G
Loop 344 over SH 199 (Bridge 8)			
1	1852	480	G*
2	1852	480	G*

G* Gauge stopped at maximum Pull-Off pressure of the adhesion tester for 50 mm dolly.

Chapter 6

Conclusions and Recommendations based on this Study

6.1 Conclusions from this study

The purpose of this study was to evaluate the long-term performance of FRP strengthened bridge components by means of Pull-Off tests. After the analysis of test results, it was determined that the results were scattered among the test group, and no significant pattern was found. Situations such as strength increase over time and varying failure modes was observed. Lack of previous Pull-Off test data at the time of repair to compare with the current test results made it impossible to draw any firm conclusions. The following conclusions were reached:

1. Based on the test results for various samples of FRP Bridge strengthening, desired type of failure Mode (G) was witnessed in a majority of samples 54%(14 of 26). This suggests that the bond between FRP and concrete is very strong in the majority of bridges.
2. Overall, 23% (6 of 26) of Pull-Off strength test results fell below a specified minimum Pull-Off test strength (200 psi). Two of those failed due to experimental errors. Therefore, it can be inferred that, the overall performance of FRP strengthening is good and the FRP-concrete bond is strong.
3. The average age of samples whose strength fell below specified minimum strength of 200 psi was 6.91 years, which indicate that FRP- concrete bond was weakened over time. However, no specific pattern of variation of strength with respect to FRP age was found. For example, maximum and minimum failure stresses for an FRP of age 2825 days were 461psi and 151psi respectively. Whereas samples with FRP age of 794 days showed much less maximum and minimum strengths of 352 psi and 281 psi respectively.

4. None of the five samples in Bridge 1 failed in Mode G and the failure modes varied from A through F, which suggests that the bond between FRP and concrete substrate either weakened with time due to various factors, or improper application of FRP.
5. The percentage of samples failing in Mode G increased from (44%) in case of girders to (83%) in case of columns. This suggests that the performance of FRP as a column strengthening material is better as compared to girder application. However, several factors, such as FRP age, variations in depth of the core cut, and location of tested sample, can make a significant difference in the test strength and can potentially increase the variability in the failure modes.
6. In general, the strength of the concrete substrate plays a major 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 FRP repair.
7. For a single bridge, the Pull-Off strengths in the FRP strengthened columns were lower than those in the FRP strengthened girders at the same FRP age. As the strength of the concrete substrate plays a major role in the bond strength of FRP-concrete system, the controlled manufacturing conditions prevailing at the precast sites, may have induced more strength in girders compared to the columns casted on field.
8. The bond strength is greater at the bottom of girders, as compared to the strengths from the girder sides. Relative difficulty of installing FRP wrapping on the girder sides may be a reason for this discrepancy.
9. Previous research showed that variations in temperature and humidity at the time of initial FRP installation affects the Pull-Off test failure modes and failure stresses.

Due to the lack of available initial data for the selected bridges, no specific conclusions could be made.

10. The results of Pull-Off tests indicate the decrease of failure stress and change of failure mode from G (Most desirable) to other less desirable failure modes over time.
11. There are large variations in the test results. Reasons for this inconsistency are unknown. Listed below are several possible factors:

- Improper application of FRP strengthening at the time of repair.
- Failure to follow the guidelines specified in ACI 440-2R-2008.
- Inconsistencies in the depth of the core drilling prior to dolly adhesion. The recommended depth per ASTM D7522 (2009) is 0.25 in. to 0.5 in. A core drill depth of 0.5 in. could result in different strength results than a 0.25 in. depth.
- Varying volumes of epoxy used per dolly. Since the dollies are manually installed, a slight difference in the volume of epoxy used per dolly could decrease precision of results.
- Irregularities on the surface of the specimen that would prevent a fully flat adhesion. If the surface is not completely flat, more epoxy will have to be used on the side that is not in contact with the dolly. This could lead to variations in epoxy thickness across the bond surface.
- Twisting of the dollies during the installation process. Twisting could create minor air voids which could decrease the adhesion performance. Therefore, an uniform pressure with no dolly rotation 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 manually, if the ideal 1:1 ratio of resin to hardener is not used, performance of the epoxy could be affected.

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

6.2 Recommendations for improvement in Pull-Off testing procedure

In this study Pull-Off test method was used to evaluate the long-term performance of FRP strengthened bridge components based on the test results. During the testing process many limitations were encountered which could influence the bond strength and failure modes resulting in varying test results .However, this variance could be reduced by improving the testing procedure. The following section discusses the limitations encountered and additional recommendations to the testing procedure.

Lateral translation of coring bit: To start the coring is the most difficult part of the entire core drilling process. While coring, the drill moves laterally which could damage the specimen surface. This kind of damage to the test specimen could be avoided by using a wooden jig. This jig is nothing but a wooden board having a circular hole with diameter slightly greater than the core drilling bit. This jig prevents the lateral translation by providing a perpendicular axis for the drilling operation. Once the core hole was established, the jig can be removed and the rest of coring can be completed normally until the desired depth is reached.

Damage to the core while drilling: During the coring process, torsional and thermal stresses are induced around the circular area. Improper fixturing of hole cutter relative to the specimen can lead to non- circular hole or damage to the FRP composite/substrate interface around the perimeter of the hole. The stresses can be minimized by using less aggressive core bits like diamond drill bits instead of toothed bits. Also the drilling device should be aligned perpendicular to the coring surface. This will reduce the chance of failure at the CFRP-concrete interface.

Inconsistent core depth: ASTM D7522 M-2009 specifies the coring depth to be in the range of 0.25 in to 0.50 in. During the regular coring process with core drill, it is hard to achieve uniform depth on the entire circular core due to difficulties in holding the coring equipment and rotating action of drilling bit. However, variations in the core depth with in the same specimen and also in other test specimens could vary the Pull-Off test results. To avoid this varying core depth, first the outer surface of the coring bit should be painted for the length of desired core depth and drilling should be stopped once the drill bit reaches the painted length.

Improper preparation of test surface: After coring to the required depth, the test surface should be cleaned before attaching the loading fixtures. Improper cleaning of epoxy could result in bonding adhesive failure at loading fixture. To avoid this undesirable failure mode, the testing surface should be cleaned with compressed air, alcohol, sanded with medium grit sand paper, and allowed to dry.

Non-Uniform epoxy thickness and improper curing: Varying levels of epoxy thickness between specimens can cause biased or scattered test results. Synthetic nap rollers should be used to apply epoxy uniformly and any extra amount of FRP on the edges of the loading fixture should be removed with the help of cotton swabs. Also epoxy should be applied and allowed to cure as per the manufactures recommended temperature and humidity. Improper curing of FRP could result in bonding adhesive type of failure at the loading fixture.

Difficulties in testing columns with curved surface: In general, Pull-Off testing is a technique in which direct tension force is applied normally on to the concrete surface to measure the bond strength of the FRP concrete substrate. Improper alignment of the adhesion tester grip can lead to biased or scattered test results and non-symmetric or mixed-mode failure patterns. However, it is not possible to apply a uniform tension force normally on to the column surface because of its curved surface. Advanced testing techniques should be developed to test this type of curved column surface. Till then, Pull-Off tests should be performed following all the specifications of ASTM D7522M-2009.

Establish sample sizes, durations and frequencies of Pull-Off tests: Large number of test samples provide more accurate results and can reduce the variability in the results. Also, more frequent testing provides more information regarding the long-term performance of FRP concrete bond. Sample size and testing frequency should be achieved by following the guidelines specified in the ASTM D7522M-2009. Also more than 5 specimens per test condition should be tested in case of varying Pull-Off test results.

Document Pull-Off test procedure followed and test results: The quality of FRP repair using wet layup process highly depends on the application techniques followed. Pull-Off tests should be performed at the time of repair to test the quality of repair work. Without this initial test results, it is difficult to compare the test results of samples taken at later point of time as they may be due to degradation over time or may be due to the poor workmanship. It is essential to document all the Pull-Off test results and testing procedure throughout the life span of retrofits or repairs to understand long performance of FRP. In this way testing process will be more consistent and less variations in the test results.

Choose samples with easy access: Lane closures, power sources, safety and aerial access equipment may limit the feasibility of conducting multiple Pull-Off tests. For more accurate test results test specimens should be selected which are easily accessible. Instead of using ladders to reach the girders on the bridges, modern equipment like fork lifts should be used. This not only makes the testing process easier but also reduces the experimental errors occurring while testing.

6.3 Recommendations in FRP application procedure for better performance

As part of this research, all the bridges were inspected manually to check the in-service condition of FRP strengthening, air pockets, delamination, debonding and FRP degradation. During the inspection process some defects in FRP application at the time of initial repair which could influence its performance were noticed. ACI 440.2R (2008) specifies the guidelines for Design and construction of externally bonded FRP systems. The following recommendations should be followed to achieve best results from FRP strengthening..

The performance of the FRP system can be affected by the environmental conditions like temperature, humidity, and moisture at the time of installation. To prevent this, temperature and humidity should be within the FRP manufacturer recommended limits. Also epoxy used for bonding FRP should not be applied on wet and cold surfaces.

The concrete substrate on to which FRP will be be applied should be cleaned and all the loose debris and unsound materials should be removed by sand blasting. Smaller cracks should be sealed with epoxy so that there will be no future corrosion of existing steel reinforcement. All the existing coatings , dust and oil which could interfere with the FRP-concrete bond should be removed and cleaned with alcohol solution. Large voids and cracks on the substrate surface should be filled with epoxy mortar or by shotcreting. Mixing of epoxy should be done at proper temperature until uniform color is achieved , following the FRP manufacturer’s recommendations. Epoxy should be mixed in sufficient quantities that could be used within the pot life of resin.

The primer is applied uniformly on the prepared surface as per the manufacturer specifications. Putty that is compatible with the FRP system should be used to fill voids and other surface discontinuities before the application of epoxy. Before applying the epoxy, the primer and putty should be allowed to cure completely as specified by the FRP system manufacturer or the FRP-concrete bond will be weak.

Then the epoxy should be applied uniformly on to the prepared surfaces and later reinforcing fibers should be gently pressed with the help of nap rollers in to the uncured epoxy. Any entrapped air between FPP layers should be released before the epoxy is cured. Protective coatings like epoxy which is compatible with the FRP strengthening system should be applied as per the manufacturer's recommendations. Periodic inspections and recommended maintenance work should be carried out to ensure the effectiveness of the protective coatings.

The cured FRP system should be evaluated for delaminations and air voids between the FRP system and the concrete as per the codal specifications. Methods such as acoustic sounding, ultrasonics, and thermography should be used to detect delaminations.

To test the quality of repair, Pull-Off tests should be performed immediately after the repair work. Without initial values of bond strength and material properties , it is not possible to differentiate between the FRP degradation over time and low strength values due to poor workmanship.

6.4 Recommendations for Future Research

In this study only Pull-Off tests were used to evaluate the long-term performance of FRP strengthened bridge components. In general, the results from Pull-Off test alone are not sufficient to judge the performance of FRP strengthening. The strength and behavior of the bond between FRP and concrete should be corroborated through other testing methods, such as thermal graphic imaging, acoustic sounding and differential scanning calorimetry.

Also, Pull-Off tests should be performed on Bridge components that are not reinforced with FRP so that the tensile strengths in both cases could be compared. In addition, more number of samples should be tested in each bridge component per condition so that large variations in test data could be eliminated, and firm conclusions can be reached.

Pull-Off tests should be performed on each FRP strengthened bridge component at regular intervals of time (0, 6, 12 months …etc.) .In this way, bond strengths can be compared over time so that long term performance of FRP concrete bond could be understood.

Experimental errors like variations in depth of core cut, thickness of epoxy per dolly, inconsistent epoxy mixing should be avoided to get more accurate and consistent Pull-Off test results.

Pull-Off tests should be performed on large scale beam specimens prepared in the laboratory under controlled environmental conditions. After exposing them to various environmental factors like corrosion agents and deicing agents, FRP strengthening should be applied. From the results of Pull-Off tests, the effect of environmental conditions on FRP concrete bond can be determined.

Also, large-scale beam specimens should be prepared in the laboratory and strengthened with FRP as per specifications of ACI 440-2R, 2008. Later they should be tested at regular intervals of 6months. The Pull-Off test results should be compared to that of FRP strengthened Bridge girders. In this way, the performance of FRP over time and FRP-concrete bond behavior could be studied.

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