Anchorage of Carbon Fiber Reinforced Polymers to Reinforced Concrete in Shear Applications

Carl W. Niemitz
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ANCHORAGE OF CARBON FIBER REINFORCED POLYMERS TO REINFORCED CONCRETE IN SHEAR APPLICATIONS

A Thesis Presented

by

CARL W. NIEMITZ

Submitted to the Graduate School of the University of Massachusetts Amherst in partial fulfillment of the requirements for the degree of

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ANCHORAGE OF CARBON FIBER REINFORCED POLYMERS TO REINFORCED CONCRETE IN SHEAR APPLICATIONS

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To the memory of my grandmothers who will forever be in my thoughts

Cecile E. Paquette

Betty O. Niemitz
ACKNOWLEDGEMENTS

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CHAPTER 1
INTRODUCTION

1.1 Fiber Reinforced Polymers

Due to ongoing deterioration and lack of maintenance, a need to rehabilitate and
lengthen the serviceable lifetime of deteriorated structures has grown. The crucial
cause in the deterioration of our infrastructure is related to environmental effects.
Infiltration of water and salt into concrete structures causes damage to both the concrete
and steel reinforcement thereby shortening the structure’s life considerably.
Earthquakes also cause damage to steel reinforced concrete (RC) structures. In many
cases the damage caused by the corrosion due to water and salt or by an earthquake is
not great enough to replace the entire RC structure, but rather it is much more cost
effective to rehabilitate individual members of the RC structure to meet the original
strength requirements. Traditionally, rehabilitating and retrofitting RC structures was
accomplished by casting new sections of concrete reinforced with steel or by fastening
steel sheets to the exterior of the damaged concrete members. The major drawback to
both of these methods is the amount of work that must be invested into their installation
(ACI Committee 440, 2002).

Within the past few decades a new technology has emerged using Fiber
Reinforced Polymers (FRP) to rehabilitate and retrofit RC structures. FRPs are
lightweight, easy to install, possess a high strength-to-weight ratio, high stiffness-to-
weight ratio, and are extremely resistant to environmental corrosion therefore making
them a proper material for retrofitting concrete structures (Garden and Hollaway, 1998).
These material properties lead to cost savings in the form of reduced installation time and labor costs and combined they outweigh the increased material cost. A design guideline issued by the American Concrete Institute (ACI) entitled *ACI 440.2R-02 Guide for the Design and Construction of Externally FRP Systems for Strengthening Concrete Structure* (ACI Committee 440) currently recognizes three systems for the external application of FRPs to RC members: wet layup systems, prepeg systems, and precured systems (ACI Committee 440, 2002). Wet layup systems consist of either unidirectional or multidirectional dry FRP fabric plys that are saturated with an epoxy resin on-site during application. Typically, a layer of epoxy resin is applied to the primed concrete surface after which a layer of FRP is adhered using rollers to remove any trapped air bubbles. Next, a second layer of epoxy resin is applied over the FRP ply to insure complete impregnation. Unlike wet layup systems, prepreg systems are saturated with resin offsite and delivered to the work site in coils. Wrapping machines can be used to automatically draw FRP from the coils and wrap the FRP around the RC element. Automated wrapping machines are typically utilized on concrete columns. Prepreg FRP systems are typically cured at a fixed temperature onsite to ensure quality control. Precured systems consist of pultruded rigid FRP laminates that are bonded to a primed concrete surface using an adhesive and rolled to insure that no air bubbles remain trapped and to remove any excess adhesive.

ACI Committee 440 (2002) currently recognizes three types of FRP composites: glass fiber reinforced polymer (GFRP), carbon fiber reinforced polymer (CFRP), and aramid fiber reinforced polymer (AFRP). Representative unidirectional material properties of each FRP fiber can be seen in Table 1.1.
Table 1.1 — Mechanical Properties of GFRP, CFRP, and AFRP Fibers (Concrete Society, 2004)

<table>
<thead>
<tr>
<th>Material</th>
<th>Elongation (%)</th>
<th>Specific Density</th>
<th>Modulus of Elasticity</th>
<th>Tensile Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>GFRP</td>
<td>3.5-4.7</td>
<td>2.6</td>
<td>10000-12500 ksi</td>
<td>350-500 ksi</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>[69-86 GPa]</td>
<td>[2.5-3.5 GPa]</td>
</tr>
<tr>
<td>AFRP</td>
<td>2.4</td>
<td>1.44</td>
<td>18000-19000 ksi</td>
<td>450-525 ksi</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>[124-131 GPa]</td>
<td>[3.1-3.6 GPa]</td>
</tr>
<tr>
<td>CFRP: High Strength</td>
<td>1.9-2.1</td>
<td>1.8</td>
<td>33000-35000 ksi</td>
<td>625-715 ksi</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>[227.5-241 GPa]</td>
<td>[4.3-4.9 GPa]</td>
</tr>
<tr>
<td>CFRP: High Modulus</td>
<td>0.7-1.9</td>
<td>1.78-1.81</td>
<td>42500-47750 ksi</td>
<td>400-800 ksi</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>[293-330 GPa]</td>
<td>[2.7-5.5 GPa]</td>
</tr>
<tr>
<td>CFRP: Ultra High Modulus</td>
<td>0.4-0.8</td>
<td>1.91-2.12</td>
<td>78000-93000 ksi</td>
<td>375-580 ksi</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>[538-641 GPa]</td>
<td>[2.6-4.0 GPa]</td>
</tr>
</tbody>
</table>

The tensile properties of FRP composites make them an excellent material for increasing the strength of RC elements. FRP fibers are anisotropic and when loaded in direct tension they are very brittle, as they do not exhibit any yielding behavior before rupture. Additionally, the material is considered to be linearly elastic until failure. The longitudinal tensile modulus of high strength CFRP is comparable to that of mild steel however the ultimate tensile strength of high strength CFRP can be six to seven times greater than that of high strength steel.

1.2 Strengthening RC Members using FRP Composites

Previous studies have shown that bonding FRP to an RC element can greatly increase the element capacity in a number of ways: (1) increase axial, flexural, and shear loading capacities; (2) increase ductility for enhanced seismic performance; (3) increase member stiffness thereby minimizing deflections; (4) increase the structures fatigue life; (5) and increase robustness against detrimental environmental effects (Buyukozturk et al., 2004). It should be emphasized that in comparison to flexural strengthening of RC members using bonded FRP, limited research exists on shear
strengthening using FRP laminates. A goal of this research program will be to contribute to the understanding of shear strengthening RC elements (beams, walls, columns, etc…) using bonded FRPs. Understanding how bonded FRP laminates increase the shear capacity of RC elements is advantageous due to the fact that shear failures are considered brittle and cataclysmic and may preclude reaching the flexural strength of an element. Increasing the shear capacity of a RC member may allow development of a flexural failure, which is generally more ductile.

1.2.1 Flexural and Shear Strengthening FRP Techniques

Using FRP to strengthen the flexural capacity of an RC beam consists of applying a single layer of FRP to the bottom face of the strength deficient member. Applying an FRP sheet to the tensile face of an RC element will provide additional flexural strength (see Figure 1.1).

![Figure 1.1 — FRP Flexural Strengthened RC Beam (ACI Committee 440, 2002)](image)

Three different techniques are used when using FRP to shear strengthen RC elements: completely wrapped elements, 3-sided wraps, and 2-sided wraps (see Figure 2.1). Completely wrapping elements is considered the most efficient wrapping scheme due to the fact that the full strength of the FRP is developed. However, completely
wrapping RC elements is not always possible due to construction limitations (for example it is not possible to completely wrap a beam that is supporting a slab on its top face). 3-sided wraps consist of using FRP sheets to wrap the web and tensile faces of an RC element. This wrapping scheme is considered moderately efficient. As discussed in Section 1.2.3 the full strength of the FRP sheet is generally not reached in this wrapping scheme due to FRP debonding. 2-sided wrapping schemes consist of bonding FRP sheets to the web faces of an RC beam. This wrapping scheme is considered the most inefficient because failure by debonding occurs at a lower strain than complete wrapping schemes.

All three shear strengthening wrapping schemes consist of applying either face plies or strips. FRP face plies consist of large sheets of FRP that encase the entire face of the RC element. FRP strips consist of thin strips of FRP sheets that get bonded to the RC element in a continuously spaced pattern. FRP strips may be placed at an angle of inclination ($\alpha$) so as to cross a crack in a perpendicular manner to gain the most strength out of the FRP laminate (see Figure 2.2).

1.2.2 Failure Modes in Flexural Strengthening Applications

Although many studies have shown that laminating FRP sheets to RC elements will increase their flexural capacity often the ultimate flexural strength of the member is not reached due to one of the following failure modes: (1) concrete crushing before yielding of the transverse or longitudinal reinforcing steel; (2) steel yielding followed by FRP rupture; (3) steel yielding followed by concrete crushing; (4) concrete cover separation; (5) FRP debonding; (6) and FRP rupture (Buyukozturk et al., 2004).
In flexural strengthening applications failure by concrete cover separation has been suggested to occur by the formation of a crack at the end of the FRP sheet due to high interfacial shear and normal stress concentrations caused by the termination of the FRP sheet (Gao et al., 2005). This failure mode is common for stiff sheets whether the strengthening application is a thick layer of FRP laminate or a steel plate. Once a crack forms at the end of the FRP sheet, the crack spreads to the longitudinal steel tension reinforcement and progresses horizontally causing concrete cover separation (see Figure 1.2).

![Figure 1.2 — Concrete Cover Separation](image)

FRP debonding consists of three separate debonding modes: (1) plate-end interfacial debonding; (2) intermediate flexural crack-induced interfacial debonding; (3) and intermediate shear crack-induced interfacial debonding (Teng, 2002). Plate-end interfacial debonding occurs from high interfacial shear and normal force concentrations near the FRP sheet-end. High stress concentrations cause debonding of the FRP sheet along with a thin layer of concrete indicating that a failure plane has developed in the concrete adjoining the FRP laminate (see Figure 1.3). Intermediate crack-induced interfacial debonding occurs from high stress concentrations induced by a flexural or shear crack. Large local interfacial stress concentrations cause debonding
of the FRP sheet along with a thin layer of concrete indicating that failure has occurred in the concrete adjoining the FRP laminate (see Figure 1.4).

![Figure 1.3 — FRP Plate-End Debonding](image)

![Figure 1.4 — Flexural or Shear Crack-Induced Intermediate Interfacial Debonding](image)

1.2.3 Failure Modes in Shear Strengthening Applications

In shear strengthening applications it has been shown that the most common modes of failure occur from intermediate crack-induced interfacial FRP debonding and FRP tensile rupture (Teng, 2002). As seen in Figure 1.5 intermediate crack-induced interfacial FRP debonding initiates at the edge of a maximum diagonal tension shear crack. As the diagonal tension shear crack becomes wider, high stress concentrations cause debonding of the FRP sheet along with a thin layer of concrete. This mode of
failure is common when diagonal tensile shear cracks occur at locations near the top of the beam because it is very difficult to develop the strength of the FRP laminate at this location therefore development of anchoring mechanisms is necessary.

**INITIATION OF DEBONDED FRP PLATE**

![Initiation of Debonded FRP Plate](image)

**Figure 1.5 — Intermediate Crack-Induced Interfacial Debonding in Shear Applications**

Failure by FRP tensile rupture may also occur at the edge of the maximum diagonal tension shear crack (see Figure 1.6). As the diagonal tension shear crack becomes wider, the strain in the FRP increases until the FRP reaches its ultimate strain at which time rupture occurs. This brittle mode of failure occurs most often at the lower end of the maximum diagonal tension shear crack the location where the crack width and tensile force are the greatest (Chen and Teng, 2003).

![FRP Rupture](image)

**Figure 1.6 — FRP Rupture**
FRP tensile rupture is the preferred mode of failure as the load-bearing capacity of the FRP sheet has been utilized. Debonding of the FRP sheet is detrimental because it does not allow the full development of the shear capacity of the FRP shear strengthened member. This mode of failure is often brittle which reveals few warning signs. ACI Committee 440 (2002) and the Concrete Society of the UK both account for the debonding failure mode by imposing a maximum strain threshold of 0.4% on the FRP for design of strengthening applications. The maximum strain threshold of 0.4% also prevents FRP rupture from occurring.

1.3 Scope of Research

The objective of this research program is to study the effects of anchoring techniques of FRP sheets used to improve the performance of strengthened reinforced concrete members primarily in shear applications including squat walls, deep beams, columns, and slender beams. Because the most common failure mode of FRP-strengthened reinforced concrete members is debonding, the goal of the research is confined to examine the effects of anchoring patterns to avoid or delay debonding of the FRP laminates from the concrete surface. No models currently exist detailing the capacity gained by anchoring FRP laminates to RC elements in shear applications. Tests are developed to study the gain in strengthening capacity of FRP sheets when anchored to the concrete surface using FRP anchors. The tests are also intended to provide an understanding of the various failure modes that occur when using this technique. The ultimate goal of this research is to contribute to the understanding of strengthening RC members with FRP laminates so a series of design equations can be
created to predict values of the additional shear strength gained by anchoring FRP laminates to RC elements using FRP anchors.
2.1 Introduction

This chapter presents a design overview of the ACI 440.2R-02 Guide for the Design and Construction of Externally FRP Systems for Strengthening Concrete Structures and Technical Report No. 55 Design guidance for strengthening concrete structures using fibre composite materials. Several bond strength models based on simple shear tests are presented. No bond strength models exist which include the capacity gained by anchoring the FRP to RC elements in shear applications. This chapter also outlines some of the most recent experimental research performed on the various methods proposed to anchor FRP sheets to RC elements.

2.2 FRP Shear Strength Design Philosophy

A design overview of the ACI 440.2R-02 Guide for the Design and Construction of Externally FRP Systems for Strengthening Concrete Structures and Technical Report No. 55 Design guidance for strengthening concrete structures using fibre composite materials is presented. Particular attention is given to FRP shear strengthening design recommendations presented in both guidelines.

2.2.1 ACI 440.2R-02 FRP Contribution to Shear Strength

The Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures as reported by ACI Committee 440 (2002) details the most recent guidelines for the application, installation, flexural
strengthening, and shear strengthening of FRP systems. Most importantly, the guidelines are based on a limit-states-design that sets limits on both serviceability and ultimate limit states. Due to the fact that little is known about strengthening concrete structures with FRP systems, ACI Committee 440 (2002) recommends using additional strength reduction factors \( \psi_f \) on top of the nominal strength reduction factors \( \phi \) to account for the unknowns. No rational justification is presented regarding the development of the additional strength reduction factors.

When designing and analyzing an FRP system ACI Committee 440 (2002) regards FRP as a linear elastic material until failure. Therefore, the design modulus of elasticity is determined from Hooke’s Law:

\[
E_f = \frac{f_{se}}{\varepsilon_{fe}} \quad [2.1]
\]

where:

\( E_f \) = tensile modulus of elasticity of FRP, psi [MPa]
\( f_{se} \) = effective stress level in FRP, psi [MPa]
\( \varepsilon_{fe} \) = effective strain level in FRP, in./in. [mm/mm]

When designing FRP shear strengthening systems three wrapping methods are proposed as shown in Figure 2.1. These include: complete wrapping schemes which encase the concrete member on all sides, U-wrapping which encases the member on three sides, and side applications which reinforce the concrete member on two sides. Complete wrapping schemes are considered to be the superior and most efficient method because failure of this application generally does not allow for debonding. U-wraps and side applications are considered to be weaker wrapping schemes because
failure by debonding often occurs limiting the final FRP strain to values lower than complete wrapping schemes.

![Shear Strengthening Wrapping Methods](image)

**Figure 2.1 — Shear Strengthening Wrapping Methods (ACI Committee 440, 2002)**

ACI Committee 440 (2002) does not provide any guidelines for anchored FRP shear strengthening systems.

The ACI Committee 440 (2002) guideline approaches the design of an FRP reinforced member similarly to the design of a steel RC member such that the nominal shear capacity of a strengthened RC member multiplied by a strength-reduction factor is to be greater than the required shear strength of the member. Required shear strength refers to load effects calculated from factored loads.

\[ \phi V_n \geq V_u \]  

where:

\( \phi \) = nominal strength reduction factor

\( V_n \) = nominal shear strength, lb [N]

\( V_u \) = ultimate shear strength, lb [N]

The nominal shear strength of the FRP strengthened RC member is calculated by summing the individual shear strength contributions from the concrete, steel stirrup reinforcement, and FRP reinforcement. An additional strength reduction factor is
applied to the strength contribution of the FRP reinforcement depending on the type of wrapping scheme applied.

\[ \phi V_n = \phi (V_c + V_s + \psi V_f) \]  \hspace{1cm} [2.3]

where:

\( V_c \) = nominal shear strength provided by concrete with steel flexural reinforcement, lb [N]

\[ V_c = 2\sqrt{f'c b_w d} \]  \hspace{1cm} [2.4]

where

\( f'c \) = concrete compressive strength, psi [MPa]

\( b_w \) = web width, in. [mm]

\( d \) = distance from extreme compression fiber to centroid of tension reinforcement, in. [mm]

\( V_s \) = nominal shear strength provided by steel stirrups, lb [N]

\[ V_s = \frac{A_y f_y d}{s} \]  \hspace{1cm} [2.5]

where

\( A_y \) = area of shear reinforcement, in.\(^2\) [mm\(^2\)]

\( f_y \) = yield strength of reinforcement, psi [MPa]

\( d \) = distance from extreme compression fiber to centroid of tension reinforcement, in. [mm]

\( s \) = spacing of transverse reinforcement, in. [mm]

\( V_f \) = nominal shear strength provided by FRP stirrups, lb [N]
The additional strength-reduction factor $\psi_f$ is 0.95 for complete wrapping schemes and 0.85 for U-wrap and side wrap schemes.

ACI Committee 440 (2002) establishes the additional shear strength gained when applying FRP wraps to a RC element by calculating the force resulting from the tensile stress in the FRP across an assumed crack.

$$V_f = \frac{A_{f_s}f_{fe}(\sin \alpha + \cos \alpha)d_f}{s_f} \quad [2.6]$$

where:

$$A_{f_s} = 2nt_fw_f \quad [2.7]$$

$$f_{fe} = e_{fe}E_f \quad [2.8]$$

where:

$A_{f_s}$ = area of FRP shear reinforcement with spacing, $s_f$, in$^2$ [mm$^2$]

$f_{fe}$ = effective stress in FRP; stress level attained at section failure, psi [MPa]

$\alpha$ = angle of inclination of FRP wraps [degrees]

$d_f$ = effective depth of the FRP strengthening, measured from the top of the FRP to the tension reinforcement, in. [mm]

$s_f$ = spacing of FRP shear reinforcement, in. [mm]

$n$ = number of plies of FRP reinforcement

$t_f$ = nominal thickness of one ply of FRP reinforcement, in. [mm]

$w_f$ = width of the FRP reinforcing plies, in. [mm]

$e_{fe}$ = effective strain level in FRP reinforcement, in./in. [mm]
Because the debonding failure mode, which is considered to be a brittle failure mode, occurs at a strain well below the FRP rupture strain, ACI Committee 440 (2002) has imposed a maximum attainable strain of 0.4% in the FRP wraps without providing any rationale for this upper limit. It appears as though this limit may have been adopted from Khalifa, Ahmed, et al. (1998) who proposed a maximum strain limit of 0.4% to maintain the shear integrity of the concrete and prevent loss of aggregate interlock. For completely wrapped elements:

\[
\varepsilon_{fc} = 0.004 \leq 0.75 \varepsilon_{fu}
\]

However, for U-wraps and side applications the ACI Committee 440 (2002) report introduces a bond-reduction coefficient, \( \kappa_v \), as these FRP applications are susceptible to the debonding failure mode. The bond reduction coefficient, \( \kappa_v \), which was experimentally derived (Khalifa et al., 1998) is dependent upon several factors including concrete strength, type of wrapping scheme used, and stiffness of the FRP laminate.

\[
\varepsilon_{fc} = \kappa_v \varepsilon_{fu} \leq 0.004 \quad \text{[2.9]}
\]
where:

\[ \kappa_v = \frac{k_1 k_2 L_e}{468 \varepsilon_{fu}} \]  \[2.10\]

\[ L_e = \frac{2500}{(nt_f E_f)^{0.58}} \]  \[2.11\]

\[ k_1 = \left( \frac{f'_c}{4000} \right)^2 \]  \[2.12\]

\[ k_2 = \frac{d_f - L_e}{d_f} \text{ for U-wraps} \]  \[2.13\]

\[ k_2 = \frac{d_f - 2L_e}{d_f} \text{ for two sides bonded} \]  \[2.14\]

where:

- \( k_1 = \) modification factor applied to \( \kappa_v \) to account for the concrete strength
- \( k_2 = \) modification factor applied to \( \kappa_v \) to account for the wrapping scheme
- \( L_e = \) active bond length of FRP laminate, in. [mm]
- \( f'_c = \) compressive strength of concrete, psi [MPa]
- \( n = \) number of plies of FRP reinforcement
- \( t_f = \) nominal thickness of one ply of FRP reinforcement, in. [mm]
- \( E_f = \) tensile modulus of elasticity of FRP, psi [MPa]

### 2.2.2 Concrete Society Committee: Technical Report No. 55

The *Design Guidance for Strengthening Concrete using Fibre Composite Materials* as reported by the Concrete Society Committee of England (2004) details the most recent guidelines for the application, design, installation, and maintenance of FRP
In beam shear-strengthening applications TR 55 recognizes three FRP bond configurations: side only, U-wrapped, and fully wrapped. It is recommended that the FRP be fully wrapped around the entire member in order to reduce separation failure of the FRP, however this is not easily accomplished in real applications due to the limited access to the full perimeter of a RC member. It is acknowledged that failure by FRP separation occurs in the concrete adjoining the FRP laminate and is related to the propagation of a failure plane in the concrete (Concrete Society, 2004).

In order to determine the ultimate shear capacity of an FRP strengthened RC beam TR 55 recommends the following equation:

\[ V_u = V_c + V_s + V_f \]  \hspace{1cm} [2.15]

where:

\( V_c \) = contribution from the concrete to the shear capacity as provided by Section 3.4.5.4 Table 3.8 in BS 8110 1997 [N]

\( V_s \) = contribution from the steel to the shear capacity as provided by Section 3.4.5.3 Table 3.7 in BS 8110 1997 [N]

\( V_f \) = contribution from the FRP to the shear capacity [N]

\( V_u \) = ultimate shear capacity of FRP strengthened section [N]

Partial safety factors for strength of materials are taken into account for the ultimate limit state.
Table 2.1 — Partial Material Safety Factors (Allen and MyiLibrary, 1997)

<table>
<thead>
<tr>
<th>Reinforcement</th>
<th>1.05</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete in flexure or axial load</td>
<td>1.50</td>
</tr>
<tr>
<td>Shear strength without shear reinforcement</td>
<td>1.25</td>
</tr>
<tr>
<td>Bond Strength</td>
<td>1.40</td>
</tr>
<tr>
<td>Others</td>
<td>1.50</td>
</tr>
</tbody>
</table>

Assuming that a 45° shear crack will form the $V_f$ can be calculated using the following equation:

$$V_f = E_{fd} \varepsilon_{fse} A_{fs} \frac{d_f}{s_f} \left( \frac{n}{3} \frac{l_{t,\max}}{s_f} \right) (\cos \beta \sin \beta)$$  \[2.16\]

where:

- $n = 0$ for a fully wrapped beam, 1 when the FRP laminate is bonded continuously to the sides and bottom of the beam, and 2 when the FRP laminate is bonded only to the sides of the beam.
- $\beta = \text{angle between the principal fibres of the FRP and a line perpendicular to the longitudinal axis of the member.}$  $eta$ is positive when the principal fibres of the FRP are rotated away from the direction in which a shear crack will form.
- $\varepsilon_{fse} = \text{effective strain in the FRP}$
- $d_f = \text{effective depth of the FRP strengthening, measured from the top of the FRP to the tension reinforcement [mm]}$
- $s_f = \text{longitudinal spacing of the FRP laminates used for shear strengthening.}$

For a continuous FRP sheet, $s_f$ is taken as 1.0 [mm]

$$E_{fd} = \frac{E_f}{\gamma_{mE}}$$  \[2.17\]
where:

$$E_{fd} = \text{design elastic modulus of the FRP laminate \left[ \frac{N}{mm^2} \right]}$$

$$E_t = \text{tensile modulus of the FRP laminate \left[ \frac{N}{mm^2} \right]}$$

$$\gamma_{mtE} = \gamma_E \times \gamma_{mm} \quad \text{[2.18]}$$

where:

$$\gamma_{mtE} = \text{design partial safety factor for modulus of elasticity of FRP}$$

$$\gamma_E = \text{partial safety factor for modulus of elasticity of FRP}$$

$$\gamma_{mm} = \text{partial safety factor for manufacture of FRP}$$

$$A_{fs} = b_f t_f \quad \text{[2.19]}$$

where:

$$A_{fs} = \text{area of FRP for shear strengthening measured perpendicular to the direction of the fibres. When FRP laminates are applied symmetrically on both sides of a beam, } A_{fs} \text{ is the sum of the areas of both laminates [mm}^2\text{].}$$

$$b_f = \text{width of the FRP laminate measured perpendicular to the direction of the fibres. For a continuous sheet } b_f \text{ is taken as } \cos(\beta)[mm].$$

$$t_f = \text{thickness of the FRP laminate [mm]}$$

$$l_{r,\text{max}} \text{ is the maximum anchorage length needed to activate the bond between the FRP laminate and concrete.}$$

$$l_{r,\text{max}} = 0.7 \sqrt{\frac{E_{fd} t_f}{f_{cm}}} \quad \text{[mm]} \quad \text{[2.20]}$$
\[ f_{ctm} = \text{tensile strength of concrete} \left[ \frac{N}{mm^2} \right] \]

Any anchorage length beyond \( l_{r,\text{max}} \) will produce no increase in the ultimate bond force, \( T_{k,\text{max}} \).

\[ T_{k,\text{max}} = 0.5k_fb_f \sqrt{E_{fd}t_f f_{ctm}} \text{ [N]} \quad [2.21] \]

where:

\[ k_f = 1.06 \sqrt{\frac{2-b_f}{b_w-b_f}} > 1.0 \quad [2.22] \]

\( b_w = \text{beam width or sheet spacing for solid slab [mm]} \)

Similar to ACI 440.2R-02, TR 55 imposes a maximum strain level attainable by the FRP laminate. The ultimate strain limit should be taken as the minimum of:

(i) \( \frac{\varepsilon_{fd}}{2} \), \quad [2.23]

(ii) \( 0.64 \sqrt{\frac{f_{ctm}}{E_{fd}t_f}} \), or \quad [2.24]

(iii) 0.004

\[ \varepsilon_{fd} = \frac{\varepsilon_fk}{\gamma_{\mu\varepsilon}} \quad [2.25] \]

where:

\[ \gamma_{\mu\varepsilon} = \gamma_e \times \gamma_{mm} \quad [2.26] \]

where:

\( \varepsilon_{fd} = \text{design ultimate strain of FRP} \)
\[ \varepsilon_{fu} = \text{characteristic failure strain of FRP} \]

\[ \gamma_{\varepsilon} = \text{design partial safety factor for strain of FRP} \]

\[ \lambda_{\varepsilon} = \text{partial safety factor for strain of FRP} \]

\[ \gamma_{mm} = \text{partial safety factor for manufacture of FRP} \]

The first strain limit of half the ultimate strain capacity corresponds to the average FRP strain when fracture of the FRP occurs as proposed by Chen and Teng (Concrete Society, 2004). The second strain limit accounts for debonding of the FRP and is based upon an anchorage model proposed by Neubauer and Rostasy (Concrete Society, 2004). The third strain limit of 0.004 has limited justification and appears to be based upon “rule of thumb” (Concrete Society, 2004). This limit may have been adopted from Khalifa, Ahmed, et al. 1998 who proposed a maximum strain limit of 0.4% to maintain the shear integrity of the concrete and prevent loss of aggregate interlock. It appears as though this strain limit is based on the limited knowledge about shear strengthening reinforced concrete in comparison with flexural strengthening. This strain limit of 0.004 is believed to provide conservative results by avoiding debonding failures and was imposed because it would be “cautious to do so” (Concrete Society, 2004).

Due to the fact that the contribution from the FRP to the shear capacity of a RC beam is based upon an assumption that the FRP laminate will span a 45° shear crack, limits are imposed on the spacing of FRP strips to ensure that the assumption is satisfied. If spacing of the FRP strips becomes large the assumption is void. If strips are used there center-to-center spacing (s_i) should not exceed the least of:

(i) 0.8d,

[2.27]
(ii) \( d_f - \frac{n}{3} l_{t, \text{max}} \), or \[2.28\]

(iii) \( b_f + \frac{d_f}{4} \) \[2.29\]

2.3 FRP Bond Strength

In shear strengthening applications it has been shown that the most common modes of failure occur from intermediate crack-induced interfacial FRP debonding and FRP tensile rupture (Teng, 2002). Intermediate crack-induced interfacial FRP debonding is a shear anchorage failure mode that can be studied directly using a simple shear test experimental setup (see Figure 2.3). To account for FRP debonding several empirical bond strength models based on simple shear tests have been developed by researchers to calculate the maximum transferable load in the FRP concrete joint. No bond strength models currently exist that account for FRP anchorage systems.
2.3.1 Chen and Teng Model

Using the results from theoretical and experimental simple shear tests (as seen in Figure 2.3) Chen and Teng (2001) developed a bond strength model taking into account the effective bond length limit of the FRP. A width coefficient is defined for the model that accounts for the relative widths of the FRP laminate and concrete element.

A key aspect to strengthening RC elements with FRP laminates is that an effective bond length limit exists where upon an increase beyond the critical bond length results in no increase in the ultimate stress in the FRP. Once the effective bond length limit is fully established the effective bond length translates along the length of the FRP laminate away from the applied load towards the free end of the FRP. As defined by Chen and Teng (2001):
where:

\[ L_e = \frac{E_p t_p}{\sqrt{f_c'}} \]  

[2.30]

\( L_e \) = effective bond length [mm]

\( E_p \) = modulus of elasticity of FRP [MPa]

\( t_p \) = thickness of FRP [mm]

\( f_c' \) = concrete cylinder compressive strength [MPa]

Equally as important in strengthening RC elements with FRP is the fact that increasing the width of the FRP laminate will increase the bond strength. Chen and Teng (2001) define a width coefficient, \( \beta_p \), taking into account the widths of the bonded FRP laminate and the RC element.

\[ \beta_p = \sqrt{\frac{2 - \frac{b_p}{b_c}}{1 + \frac{b_p}{b_c}}} \]  

[2.31]

where:

\( b_p \) = width of sheet [mm]

\( b_c \) = width of concrete [mm]

Chen and Teng (2001) propose an ultimate strength for design, \( P_u \), which is defined as the force in the bonded laminate at joint failure:

\[ P_u = 0.427 \beta_p \beta_L \sqrt{f_c'} b_p L_e \]  

[2.32]

where:
\[ P_u = \text{ultimate bond strength [N]} \]

\[ \beta_L = \text{bond length coefficient} \]

\[ \beta_L = 1 \text{ if } L \geq L_e \]

\[ \beta_L = \sin\left(\frac{\pi L}{2L_e}\right) \text{ if } L \leq L_e \]  \[2.33\]

\textbf{2.3.1.1 FRP Length and Width Effects}

Several researchers have been shown that an increase in the FRP width will produce an increase in the nominal stress in the FRP at debonding (Subramaniam et al., 2007). Recent research by Subramaniam et al. (2007) using a full-field optical technique on simple shear tests (test setup seen in Figure 2.4) have suggested that there is a stress transfer zone along the length of the FRP laminate which is independent of the FRP laminates width.

![Figure 2.4 — Shear Test Setup (Subramaniam et al., 2007)](image-url)
The length of the stress transfer zone is fixed and is bounded by FRP that is free of axial strain and FRP that has debonded at the loaded end of the FRP sheet as seen in Figure 2.5. Figure 2.5 illustrates the axial strain in the FRP along the length of the FRP for a bonded width \( b_1 \) of 25 mm (1 in) (Subramaniam et al., 2007). This axial strain distribution was determined along the centerline of the FRP laminate sheet (Subramaniam et al., 2007). The trend of ‘x’s corresponds to the measured axial strain the FRP laminate sheet (Subramaniam et al., 2007). It is believed that the fluctuations in the measured axial strain are due to local material variations in the FRP sheet (Subramaniam et al., 2007).

As increased load is applied, once the stress transfer zone is fully established the stress transfer zone translates along the length of the FRP laminate away from the

Figure 2.5 — Axial Strain Distribution along FRP Length (Subramaniam et al., 2007)
applied load towards the free end of the FRP laminate with its shape remaining constant (Subramaniam et al., 2007). When analyzing measured experimental strain distributions in the FRP laminate of shear tests performed by Yao et al. (2005), as seen in Figure 2.6 and Figure 2.7, translation of the stress transfer zone along the length of the FRP towards the free end of the FRP was observed as increased load was applied. The strain distribution analyzed in Figure 2.6 and Figure 2.7 was for a bonded length of FRP ($L_{FRP}$) equal to 190 mm (7.5 in). Figure 2.6 and Figure 2.7 depicts the strain distribution over the normalized length of FRP where $L_e$ is the effective bond length as defined by the Chen and Teng bond strength model and $x$ is the distance measured from the loaded end of the FRP. The axial strain in the FRP was measured by mounting strain gages on top of and down the centerline of the FRP laminate sheet.

Bizindavyi and Neale (1999) suggest that different transfer phases along the length of the FRP can be identified as the level of loading is increased. The first phase is recognized to occur during initial loading corresponding to exponentially decreasing values of axial strain towards the unloaded end termed the initial transfer length. The maximum load with this strain distribution is believed to be the load to initiate a crack front in the concrete block at the loaded end initiating local debonding. The second phase is believed to occur with an additional increase in load moving the point of stress transfer a small distance towards the unloaded end of the FRP. The final phases are believed to occur at higher load levels causing the propagation of debonding towards the unloaded end of the FRP. Decreased values of axial strain at a post-peak load level signify a weakening due to concrete cracking in these FRP concrete bonded regions.
Once the stress transfer zone reaches the free end of the FRP laminate complete debonding occurs. This is an important concept because although longer bond lengths
do not increase bond strength, longer bond lengths do lead to a longer debonding process as the stress transfer zone translates to the free end of the FRP sheet leading to increased ductility (Yao et al., 2005).

As seen in Figure 2.8 and Figure 2.9 three zones are defined across the width of the FRP within the stress transfer zone: (1) a central region ($b_s$); (2) edge regions ($b_d$); and (3) regions outside of the edge region (Subramaniam et al., 2007). The central region exists within the center of the FRP laminate width and is defined by no shearing strains and maximum constant axial strains. The edge regions are of constant width and consist of an area outside of the central region that is composed of both high axial and shear strain gradients. The edge regions extend from the edge of the central region past the edge of the FRP laminate into the RC. The regions outside of the edge regions are free from both axial and shear strains. Subramaniam et al. (2007) propose that since the axial strain measured in the central region, $b_s$, is considerably greater than the axial strain measured in the edge regions, the central region is principally accountable for shear stress transfer between the FRP and concrete member. Figure 2.8 and Figure 2.9 were developed using experimental data from shear tests where $y$ corresponds to the location along the length of bonded FRP (Subramaniam et al., 2007).
Figure 2.8 — Shear Strain Distribution across FRP Width (Subramaniam et al., 2007)

Figure 2.9 — Axial Strain Distribution across FRP width (Subramaniam et al., 2007)
Subramaniam et al. (2007) propose that the size of the edge regions remain constant for any FRP laminate width; therefore, the width of the central region, $b_s$, enlarges with increasing FRP laminate width explaining the increased nominal stress in the FRP at debonding. As long as the width of the concrete is great enough to allow the full formation of the edge regions, increasing the width of the FRP increases the bond strength.

### 2.3.2 Model Presented by Khalifa, Gold, Nanni, and Aziz (1998)

The bond strength model presented by Khalifa et al. (1998) is based upon a model previously presented by Maeda et al. (1997) with a modification made in calculating the ultimate bond shear stress. Maeda et al. (1997) propose that the ultimate bond shear stress, $\tau_u$, can be calculated as:

$$\tau_u = 110.2 \times 10^{-6} E_p t_p \text{ [MPa]}$$

where:

$E_p = \text{modulus of elasticity of FRP [MPa]}$

$t_p = \text{thickness of FRP [mm]}$

However, a weakness of this model is that concrete compressive strength is not taken into account, therefore, Khalifa et al. (1998) propose:

$$\tau_u = 110.2 \times 10^{-6} E_p t_p \left( \frac{f_{c'}^{*}}{42} \right)^{\frac{2}{3}} \text{ [MPa]}$$

where 42 represents a concrete compressive strength of 42 MPa that was used in Maeda et al. (1997) experiments.

Maeda et al. (1997) propose that the effective bond length may be calculated as:
$L_c = e^{6.13 - 0.58 \ln E_{fr}} \text{ [mm]}$  \hspace{1cm} [2.36]

Finally it is proposed that the ultimate bond strength can be calculated as:

$$P_u = L_c \tau_s b_p \text{ [N]}$$  \hspace{1cm} [2.37]

where:

$b_p = \text{width of the FRP sheet [mm]}$

### 2.4 FRP Systems to Delay Debonding Failures in Shear Applications

Due to the fact that debonding of FRP occurs at loads well below their rupture strength various anchorage systems have been developed to fasten the FRP laminate to the concrete element to prevent the debonding failure mode therefore employing the full tensile capacity of the FRP. An overview of four anchorage systems is presented in this section.

#### 2.4.1 Near Surface Mounted FRP Laminates

The use of near surface mounted (NSM) reinforcement started in the 1940s with the idea of strengthening RC members with steel reinforcing bars fastened into preformed grooves on the exterior of a concrete member (Asplund, 1949). Two thin slits were cut into the concrete member using a diamond circular saw. A chisel was used to remove the concrete between the two slits leaving a groove into which a reinforcing bar was grouted into place (Asplund, 1949). However, due to the development of high strength epoxies, FRP laminates, and FRP rods a new technique has emerged whereby FRP rods have replaced the steel reinforcing rods. This technique has shown major improvements due to the non-corrosive properties of FRP as
well as the ease of installation of lightweight FRP rods in comparison to heavy steel reinforcing rods. Application of the NSM rods is achieved by cutting grooves into the concrete surface in the desired location and direction, filling the groove approximately half way with an epoxy paste, lightly pressing the rod into the paste, and finally filling the groove with more paste until the groove is leveled with the concrete surface (Khalifa et al., 2000).

Research completed by Parretti and Nanni (2004) has yielded a limit-states shear design method for NSM FRP rods. The approach is similar to that of the ACI Committee 440 (2002) design guide for externally bonded FRP bonded laminates in shear. The shear design capacity of a strengthened RC beam is calculated using:

$$\phi V_n = \phi (V_c + V_s + \psi_f V_f)$$  \[2.38\]

where,

- $\phi$ = strength reduction factor
- $V_n$ = nominal shear strength
- $V_c$ = nominal shear strength provided by concrete
- $V_s$ = nominal shear strength provided by steel stirrups
- $\psi_f$ = additional FRP strength reduction factor
- $V_f$ = nominal shear strength provided by FRP stirrups

The shear capacities of the NSM FRP rods are modified depending on the type of FRP rod, dimensions of the groove, and quality of the substrate material (Parretti and Nanni, 2004). For circular bars the shear contribution of the FRP rods is calculated as:

$$V_f = 2\pi d \tau_{\text{eff}} L_{\text{tot_min}}$$  \[2.39\]
where,

\[ d_b = \text{diameter of FRP bar} \]

\[ \tau_b = \text{average bond stress of the bars crossed by the an assumed shear crack of 45°} \]

\[ L_{tot\,min} = \text{sum of the lengths of each vertical FRP rod crossed by an assumed shear crack of 45°} \]

\( L_{tot\,min} \) is limited by two failure modes: debonding of the FRP system from the substrate and loss of the concrete shear integrity. Debonding of a FRP rod is determined by calculating the effective length of the rod crossing an assumed shear crack of 45°. A maximum FRP strain threshold of 0.004 in the rod is the greatest allowed by ACI Committee 440 (2002) to ensure that no loss of aggregate interlock in the concrete occurs.

Using a similar technique as NSM CFRP rods a more recent technology has emerged whereby FRP strips are placed into slits cut into the concrete cover. Using a conventional concrete saw thin slits approximately 5 mm wide and 12 mm deep are cut at the desired spacing in the concrete cover (Barros and Dias, 2006). After being cleaned and filled with an epoxy resin, thin FRP laminate strips are inserted into the slits with any excess epoxy being removed. The proposed shear design method is the same for NSM FRP rods and strips with the exception that a greater average bond stress \( \tau_b \) is proposed for FRP strips due to a constant rectangular slit and strip thickness in comparison to NSM FRP rods which are circular rods inserted into rectangular grooves (Barros and Dias, 2006).
2.4.2 Mechanically Fastened FRP Laminates

Mechanically fastened FRP is a relatively new technology developed to rapidly attach FRP strips to strengthen RC elements in flexure. The use of powder-actuated fasteners allows for quick installation of FRP strips due to the fact that no concrete surface preparation is needed and no curing time is necessary as in the case of conventional FRP’s laminated with epoxy resins. Gunpowder actuated guns are used to shoot metal fasteners through the FRP strip into the RC element at a predetermined spacing and pattern.

Bank (2004) has suggested to pre-drill holes into the concrete substrate in order to increase tensile and shear capacities of the metal fasteners and reduce spalling during fastener driving. It should be noted however that pre-drilling is not a required part of this method and that shooting the fastener directly through the FRP strip into the concrete is acceptable. No literature has been found to date where this method has been used in shear strengthening RC elements. An analytical model is presented by Bank (2004) for applications involving flexural strengthening of RC beams.
2.4.3 FRP Anchors

FRP anchors are made from fibers used as part of the FRP sheets. The anchors are made by bundling fibers into a roll and splaying the upper end of the anchor so that fibers spread out of the FRP sheet.

Figure 2.11 — FRP Anchor

Some advantages to using FRP anchors are that no mechanical fasteners are necessary to attach the anchors to the RC element, the method requires very little installation time, and the method is simple. Holes are to be drilled into the RC element at the desired depth and spacing after which the hole is cleaned and filled with epoxy resin approximately halfway into which the FRP anchor is placed. A layer of epoxy resin is placed on the concrete surface. After the anchor has been placed into the hole the splayed end is passed through the FRP laminate and distributed over the sheet. Once all of the anchors have been passed through the FRP sheet epoxy resin is applied over the sheet and splayed anchor ends to ensure that the two elements will bond together during curing.
Previous experiments by Orton et al. (2006) using a modified beam test have studied the number, diameter, and spacing of fastened FRP anchors to RC elements with and without height transitions. Results from the tests without height transitions and without any additional FRP anchors revealed that the FRP sheet debonded at 40% of the FRP composite sheets ultimate capacity. Orton et al. (2006) concluded that the use of FRP anchors allowed the FRP sheet to reach its ultimate capacity depending on the number and size of FRP anchors used (see Table 2.2).
Table 2.2 — FRP Anchor Test Results (Orton et al., 2006)

<table>
<thead>
<tr>
<th>Anchorage System</th>
<th>Debonding as % of Ultimate Strength</th>
<th>Normalized Area of CFRP</th>
<th>Failure Mechanism</th>
</tr>
</thead>
<tbody>
<tr>
<td>none</td>
<td>40</td>
<td>2</td>
<td>Peeling</td>
</tr>
<tr>
<td>none</td>
<td>35</td>
<td>2</td>
<td>Peeling</td>
</tr>
<tr>
<td>5/8&quot; anchor at 5&quot; and 19&quot;</td>
<td>71</td>
<td>3.5</td>
<td>Fracture anchor, partially delaminate</td>
</tr>
<tr>
<td>2 - 3/8&quot; anchors at 5&quot; and 19&quot;</td>
<td>71</td>
<td>3</td>
<td>Fracture of 4.5&quot; width, peeling of 1.5&quot;</td>
</tr>
<tr>
<td>2 - 1/2&quot; anchors at 5&quot; and 19&quot;</td>
<td>85</td>
<td>3.5</td>
<td>Fracture</td>
</tr>
<tr>
<td>2 - 5/8&quot; anchors at 5&quot; and 19&quot;</td>
<td>105</td>
<td>4.25</td>
<td>Fracture</td>
</tr>
<tr>
<td>3 - 3/8&quot; anchors at 5&quot; and 19&quot;</td>
<td>106</td>
<td>3.5</td>
<td>Fracture</td>
</tr>
</tbody>
</table>

Results signified that depending on the size of the anchor each is able to engage a limited width of the FRP sheet: increasing the anchor width from ⅜ inch to ½ inch (33% increase) netted an increase in the effective width from two inches to three inches (Orton et al., 2006). Concrete surface preparation was found to be insignificant due to the fact that bonding strength between the FRP sheet and concrete substrate was not a critical factor in developing the ultimate capacity of the FRP sheets.

2.5 Summary

This literature review has presented a general review of ACI 440.2R-02 and Technical Report No. 55 FRP shear strength design guidelines. Both design guides recognize that debonding is the dominant failure mode in shear strengthening applications, which occurs well below the FRPs ultimate strength capacity. To prevent
debonding failure both guidelines place a strain limit on the FRP that appears to be arbitrarily defined due to the limited knowledge of shear strengthening RC members with FRPs. The strain limit of 0.004 enforced by both guidelines reduces the usefulness of the application because it limits the maximum attainable strain in the FRP by up to 75% of the FRPs ultimate strain depending on the FRP system used. Neither report provides guidelines for anchored FRP shear strengthening systems.

In FRP shear strengthening applications it is largely recognized that intermediate crack-induced interfacial FRP debonding is the prevailing failure mode. Intermediate crack-induced interfacial FRP debonding is a shear anchorage failure mode that can be studied directly using a simple shear test experimental setup. FRP to concrete bond strength models based upon simple shear tests are presented. Both bond strength models recognize an effective bond length limit exists beyond which an increase in the critical bond length results in no increase in the ultimate bond strength. The Chen and Teng (2001) model recognizes that a width coefficient exists taking into account the fact that increasing the FRP laminate width increases the ultimate bond strength. A discussion of the FRP width effect is presented where the concepts of central regions, edge regions, and regions outside of the edge regions is discussed concluding that increasing the FRP laminate width will result in a stronger bond strength as long as the full formation of edge regions is permitted (Subramaniam et al., 2007).

A review of FRP anchorage systems is presented including near surface mounted FRP laminates, mechanically fastened FRP laminates, and FRP anchors. No models currently exist detailing the capacity gained by anchoring FRP laminates to RC
elements. The objective of this research program is to study the effects of anchoring FRP laminates to concrete members with FRP anchors thereby avoiding or delaying the debonding failure mode. A simple shear test experiment is developed to study the effects of anchoring FRP laminates with ¼-inch (0.635 cm), ½-inch (1.27 cm), and ¾-inch (1.91 cm) diameter FRP anchors with different bolt patterns and splay diameters. A goal of the research is contribute to the understanding of fastening FRP laminates with FRP anchors so a design strength model may be formed.
CHAPTER 3
EXPERIMENTAL PROGRAM

3.1 Introduction

The objective of this research is to develop techniques to reduce or eliminate debonding of FRP from reinforced concrete (RC) members dominated by shear. An experimental program was designed to study the effects of using FRP anchors to fasten FRP sheets to RC members. Six RC blocks were fabricated in the Gunness Structural Engineering Laboratory. All RC blocks had the same dimensions and steel reinforcement. Holes of ⅜-inch (0.95 cm), 9/16-inch (1.43 cm), and 7/8-inch (2.22 cm) diameters were drilled into the RC block at varying spacing to insert FRP anchors of different diameters. FRP anchors of ¼-inch (0.64 cm), ½-inch (1.27 cm), and ¾-inch (1.91 cm) diameters with 2-inch (5.08 cm) lengths were used to fasten FRP sheets to the RC blocks. A tensile force was applied to FRP sheet to generate interface shear stresses between the FRP material and concrete surface (see Figure 3.1). Strain gages were placed longitudinally and transversely on the FRP sheet to measure strains in the FRP sheet loading was increased during testing. A data acquisition system was used to record strain and load at a rate of one reading every three seconds.

3.2 Test Setup

Six reinforced concrete blocks were used to investigate different FRP sheet and anchor configurations simulating an FRP-strengthened structure. The specimens were tested in the Gunness Structural Engineering Laboratory in a load reaction frame that was custom designed to apply force in direct shear (see Figure 3.1).
To ensure that the blocks would not slip relative to the reaction frame the FRP strengthened concrete blocks were post-tensioned to two W8x35 web stiffened fastening beams (see Figure 3.2). Specimens were oriented so that the post-tensioning ducts were located directly above bolt holes located in the stiffening beams. A total of eight post-tensioning rods were used per block. The W8x35 fastening beams were bolted to one of the W24x162 floor beams in the reaction frame.
Load was applied to the FRP sheets using a double acting hydraulic ram with a compression capacity of 110-kip (489.3 kN) and a tension capacity of 56-kip (249.1 kN). The load magnitude was measured using a 50-kip (222.4 kN) load cell (see Figure 3.3, Figure 3.4, and Figure 3.5).
The load from the ram was evenly distributed to the FRP sheet by means of a slip-critical bolted connection. Two 10-inch (25.4 cm) long, 3-inch (7.6 cm) wide, ¼-inch (0.64 cm) thick steel plates were bonded to the top and bottom of the FRP sheet end. Two 14-inch (35.6 cm) long, 4.5-inch (11.43 cm) wide steel plates connected to the double acting hydraulic ram via two slip critical 1-⅛ inch (2.9 cm) A490 structural...
steel bolts clamped to the steel plates that were bonded to the FRP sheet (see Figure 3.6 and Figure 3.7). The two steel clamping plates were fabricated with a lipped edge that provided a surface for the two FRP steel bonded plates to bear against when load was applied by the hydraulic ram (see Figure 3.7 and Figure 3.8). This connection ensured an even distribution of the load from the hydraulic ram to the FRP sheet. Photographs of the direct shear test setup can be seen in Appendix O.

Figure 3.6 — FRP Bolted Connection without Clamping Plates
3.3 Concrete Block Geometry and Reinforcement

A concrete mix with a nominal compressive strength of 4000 psi (28 MPa) and a maximum aggregate size of approximately ½-inch (0.635 cm) was used to cast six reinforced concrete blocks. The block dimensions were 40-inches (101.6 cm) wide by 34-inches (86.4 cm) long by $12\frac{11}{16}$ inches (32.2 cm) deep. The concrete blocks were
covered with wet burlap and plastic sheathing for seven days to help keep moist and ensure proper curing. After the seven-day curing period the concrete forms were removed and the blocks were cured in air to their 28-day compressive strength. The concrete blocks were reinforced with Grade 60 reinforcing bar with 2-inches (5.1 cm) of clear cover on top and bottom faces. Moment resistance was provided by six #4 steel reinforcing bars running longitudinally on both the compression and tension faces spaced according to Figure 3.9. Shear reinforcement was provided by eight #4 steel-reinforcing bars running transversely across the concrete block spaced according to Figure 3.9 and Figure 3.10. In order to securely fasten the block and to keep it from slipping during load application the use of post-tensioning steel rods was required. Assuming a coefficient of static friction (µ) of 0.47 between the concrete block and steel fastening beams, eight Grade 75 fully threaded steel reinforcing bars post-tensioned to 12 kips (53.4 kN) each were required to secure the block onto the reaction frame. This post-tensioning force allowed application of a maximum horizontal force of approximately 38 kips (169 kN) applied to a 10-inch (25.7 cm) wide FRP sheet. In order to prevent a local failure around the post-tensioned anchorage zone, four #3 Grade 60 reinforcing bar hoops were provided around each anchorage zone to confine the concrete around the post-tensioning anchors and keep this area of concrete from spalling (see Figure 3.9 and Figure 3.10). Photographs of the steel reinforcement can be seen in Appendix N.
Figure 3.9 — Concrete Block Reinforcement (Side View)

Figure 3.10 — Concrete Block Reinforcement (Profile View)

3.4 FRP Strengthening Systems

One ply of unidirectional carbon fiber reinforced polymer was bonded to the reinforced concrete blocks using a two-step wet-layup process. CFRP anchors of
varying diameters, splay diameters, and spacing were used in some specimens to fasten the FRP sheet to the RC blocks. The process used to fabricate the FRP anchors is described in section 3.5.2.

3.4.1 FRP Material Description

The properties of the FRP strengthening system as provided by BASF®, the manufacturer of the composite system, can be seen in Table 3.1. The FRP system consists of unidirectional carbon fibers that are bundled together in strands to form carbon-fiber sheets. These fibers are impregnated using a two-part epoxy resin to form the FRP composite using a wet-layup procedure. Pictures showing the dry carbon fibers and epoxy resin used for the FRP composite material are shown in Figure 3.11 and Figure 3.12. Results from FRP coupon tests used to characterize the FRP materials used for this research are presented and discussed in Chapter 4.

![Figure 3.11 — Epoxy Primer and Two-Part Epoxy Resin](image)
3.4.2 FRP Anchor Fabrication

Wabo® Mbrace CF 130 unidirectional FRP sheets were used to fabricate FRP anchors. In order to fabricate ¼ inch (0.64 cm) diameter FRP anchors, 2-inch (5.1 cm) wide sheets were cut from the standard 24-inch (61 cm) carbon fiber sheet (see Figure 3.13). One-half inch (1.3 cm) diameter FRP anchors were fabricated from 4-inch (10.2 cm) wide sheets cut from the standard 24-inch (61 cm) carbon fiber sheet. Three-quarter inch (1.9 cm) diameter FRP anchors were fabricated from 8-inch (20.3 cm) wide
sheets cut from the standard 24-inch (61 cm) carbon fiber sheet. The length of the FRP sheet cut varied depending on the length of the bolt and splay diameter required. A 2-inch (5.1 cm) long anchor with a 2-inch (5.1 cm) splay diameter required the FRP sheet to be cut into a 3-inch (7.6 cm) length (see Figure 3.13). A 2-inch (5.1 cm) long anchor with a 4-inch (10.2 cm) splay required the FRP sheet to be cut into a 4-inch (10.2 cm) length.

The 2-inch (5.1 cm), 4-inch (10.2 cm), and 8-inch (20.3 cm) wide FRP sheets were then rolled until the ¼-inch (0.64 cm), ½-inch (1.3 cm), and ¾-inch (1.9 cm) diameters were attained at which time the anchor was tied using commercially available plastic ties at the bolt end and at the start of splay region (see Figure 3.14). Both plastic ties were left in place during anchor installation.
Figure 3.14 — Step 2: FRP Anchor Fabrication

The top end of the anchor was then splayed so a top diameter of 2-inches (5.1 cm) or 4-inches (10.2 cm) was attained to ensure complete bonding with the FRP sheet during curing (see Figure 3.15). The anchor diameter was varied in order to cover different widths of carbon fiber sheet.
3.4.3 FRP Application Process

Prior to application of the FRP composite materials preparation of the concrete specimens consisted of using a mechanical grinder to grind off any uneven spots and the top layer of mortar until the aggregate was just visible and a uniform concrete surface had been established followed by blowing particles off of the concrete surface using pressurized air (see Figure 3.16 and Figure 3.17).
Figure 3.16 — Typical Concrete Block before Surface Preparation

Figure 3.17 — Typical Concrete Block after Surface Preparation
Bolt holes to insert FRP anchors were drilled at the proper spacing and to the appropriate depth for each specimen followed by the removal of any particles using pressurized air. Application of a low viscosity, high solids primer (Wabo® MBrace Primer) was followed by a high solids saturant (Wabo® MBrace Saturant) onto which the FRP sheet was applied.

![Figure 3.18 — Application of Primer](image)

The time of application between the primer and saturant was 45 minutes minimum but not before the primer exhibited a tacky surface condition. After filling the bolt holes approximately half way with epoxy resin and applying an initial layer of epoxy resin to the surface of the RC blocks, the FRP anchors were inserted into the hole with the splayed end passing through one ply of a Wabo® MBrace CF 130 FRP Anchor Holes.
unidirectional FRP sheet (see Figure 3.19). The anchor fibers were passed through the carbon fiber sheets by opening the carbon-fiber sheet bundles transversely without damaging or cutting any carbon fibers. The FRP sheet and splayed FRP anchors were then rolled in the fibers longitudinal direction using an air removal roller to remove trapped air and to impregnate the fibers.

![Anchor Splay](image)

**Figure 3.19 — FRP Anchor following Initial Saturant Application Layer**

After waiting approximately 30 minutes to allow the saturant to impregnate into the carbon fiber sheet and into the embedded FRP anchors a second application of epoxy resin was then applied to the top of the FRP sheet and the splayed FRP anchor fibers to ensure that the two elements bonded together during curing followed by rolling to remove any trapped air bubbles (see Figure 3.20).
To prevent the edge of the concrete block from being damaged during testing, which possibly would have limited the use of the concrete block to one side, an unbonded length between the surface of the concrete and FRP material of 5-inches (12.7 cm) was provided at the loaded end of the specimen. The FRP debonded by means of a 5-inch (12.7 cm) wide plastic transparency sheet that was placed on the concrete surface prior to forming the FRP composite. The FRP composite extended 11-inches (27.9 cm) from the end of the concrete block to the loading tabs next to the hydraulic ram connection to allow for the distribution of forces. The 11-inch (27.9 cm) extension FRP was formed on top of falsework to avoid it from sagging during the fiber saturation process. The falsework was covered with 0.002 inch (0.051 mm) thick plastic sheathing to keep the epoxy from bonding to the wood surface.
3.4.4 Strengthening Configurations

The primary objective of the tests was to gain an understanding of the anchor-laminate behavior in order to develop design guidelines for this type of anchorage system. FRP anchor spacing, diameter, and splay diameter were varied to determine the most efficient placement of FRP anchors to fasten a sheet of FRP laminate and prevent
the debonding failure mode. FRP laminate sheet width and length were varied to study the ability of FRP anchors to fasten FRP sheets with varying properties. An FRP anchor length of 2-inches (5.1 cm) was kept constant throughout all tests because anchor pullout was not observed in any of the initial test specimens that had 2-inch (5.1 cm) long anchors. A specimen identification key was assigned for all specimens as described in Figure 3.23.

![Figure 3.23 — Specimen Designation Key](image)

The specimens were instrumented using 119.5 ± 0.5 Ω electrical resistance strain gages with gage lengths of 5 mm and 6 mm at different locations (longitudinally and transversely) throughout the bonded length of the FRP laminate. Load was monitored using a 50-kip (222.4 kN) load cell attached to the hydraulic ram piston. Data from the strain gages and the load cell was collected in a Hewlett Packard 3852 data acquisition system and saved to a spreadsheet application for reduction and plot generation after each test.

Data acquisition initiated at the beginning of each test after all slack was removed (by a visual inspection) from the 11-inch (27.9 cm) FRP extension between the end of the concrete block and the loading tabs next to the hydraulic ram connection followed by zeroing of the strain gages and load cell. Data acquisition during the
testing Specimen A-0-0-10-0 was stopped at load increments of 2 kips (8.9 kN) so inspections could be conducted and pictures could be taken of concrete cracking and FRP debonding, however, after preliminary data reduction it was decided that all subsequent tests would be loaded with a constant uniform load until failure. The hydraulic ram was operated by means of a hydraulic cylinder hand pump as seen in Figure 3.24 with a monotonic loading rate of approximately 65-75 lb/sec. A general testing matrix is provided in Table 3.2.

Figure 3.24 — Hydraulic Cylinder Hand Pump
### Table 3.2 — Test Matrix

<table>
<thead>
<tr>
<th>Group</th>
<th>Bond Length</th>
<th>Bond Width</th>
<th>Anchorage Diameter</th>
<th>Anchorage Patterns</th>
<th>Splay Diameter</th>
<th>Purpose of Test</th>
</tr>
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<tbody>
<tr>
<td>A</td>
<td>30 in [76.2 cm]</td>
<td>5 in [12.7 cm]</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>Control Specimen</td>
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<tr>
<td></td>
<td></td>
<td>10 in [25.4 cm]</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>30 in [76.2 cm]</td>
<td>5 in [12.7 cm]</td>
<td>1/4 in [0.64 cm]</td>
<td>Z</td>
<td>2 in. [5.1 cm]</td>
<td>FRP Anchor Bolts</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1/2 in [1.3 cm]</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1/2 in [1.3 cm]</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3/4 in [1.9 cm]</td>
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<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>12.5 in [31.8 cm]</td>
<td></td>
<td>1/2 in [1.3 cm]</td>
<td>W</td>
<td>2 in. [5.1 cm]</td>
<td></td>
</tr>
<tr>
<td></td>
<td>30 in [76.2 cm]</td>
<td></td>
<td>1/2 in [1.3 cm]</td>
<td>Y</td>
<td>2 in. [5.1 cm]</td>
<td></td>
</tr>
<tr>
<td></td>
<td>15 in [38.1 cm]</td>
<td></td>
<td>1/2 in [1.3 cm]</td>
<td>X</td>
<td>2 in. [5.1 cm]</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>15 in [38.1 cm]</td>
<td>10 in [25.4 cm]</td>
<td>3/4 in [1.9 cm]</td>
<td>X</td>
<td>4 in. [10.2 cm]</td>
<td>FRP Anchor Bolts</td>
</tr>
<tr>
<td></td>
<td>30 in [76.2 cm]</td>
<td></td>
<td></td>
<td>Y</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>30 in [76.2 cm]</td>
<td></td>
<td>1/2 in [1.3 cm]</td>
<td>U</td>
<td>2 in. [5.1 cm]</td>
<td></td>
</tr>
</tbody>
</table>

Note: Anchor patterns are presented in section 3.4.4.2 and 3.4.4.3.

### 3.4.4.1 Specimen Group A

Specimen group A, which was a control group, consisted of a total of 2 tests conducted on one block and had one ply of bonded FRP with no FRP anchors. The goal of specimen group A was to set a baseline for subsequent tests to establish the ultimate load when FRP debonding occurred and to establish the distribution of strains throughout the FRP bonded length and width during the debonding process. Two tests were conducted on one block by flipping the block over after the end of each test. Side
one of specimen group A had one ply of bonded FRP 10-inches (25.4 cm) wide by 30-inches (76.2 cm) long (see Figure 3.25). Side two of specimen group A had one ply of bonded FRP 5-inches (12.7 cm) wide by 30-inches (76.2 cm) long (see Figure 3.26).

Figure 3.25 — Specimen A-0-0-10-0

Figure 3.26 — Specimen A-0-0-5-0
3.4.4.2 Specimen Group B

The goal of specimen group B, which consisted of a total of 7 tests conducted on four concrete blocks, studied the effects of using one row of ¼-inch (0.64 cm), ½-inch (1.27 cm), and ¾-inch (1.9 cm) diameter FRP anchors to study the efficiency of individual anchors to engage a given width of FRP material. An FRP anchor length of 2-inches (5.1 cm) was kept constant throughout all tests in this specimen group as anchor length was determined to be a non-controlling factor due to the fact that anchor pullout was not observed in any of the initial test specimens.

The first pattern tested in specimen group B was anchor pattern Z. Four tests were performed using this pattern to study the capacity gained over the control specimen by fastening a bonded FRP sheet using FRP anchors spaced according to Figure 3.27. The objective of using anchor pattern Z was to achieve FRP rupture by varying the FRP anchor splay diameter and anchor diameter. Subsequent to anchor pattern Z, one test was performed using anchor pattern W to determine the shear capacity of an FRP anchor as well as to establish the contribution of an FRP anchor to the increase in capacity gained over the control specimen by using FRP anchors spaced according to Figure 3.28. Following the testing of anchor pattern W, one test was performed using anchor pattern Y to verify the concepts learned from the previous test specimens, mainly that providing anchors at the lead edge of the FRP laminate is the major factor in achieving FRP rupture. The purpose of using anchor pattern Y was to reinforce the concept that FRP rupture could be obtained across a width of the FRP laminate by providing a single row of FRP anchors at the lead edge of the FRP sheet spaced according to Figure 3.29. Lastly, one test was performed using anchor pattern X.
(see Figure 3.30) to determine if bond length is a significant parameter in the additional capacity gained by utilizing FRP anchors.

Test one of specimen group B (Specimen B-Z-2-5-2) had one ply of bonded FRP 5-inches (12.7 cm) wide by 30-inches (76.2 cm) long anchored with one line of $\frac{1}{4}$-inch (0.64 cm) diameter, 2-inch (5.1 cm) splay diameter FRP anchors spaced according to Figure 3.27. Test two of specimen group B (Specimen B-Z-2-5-4) had one ply of bonded FRP 5-inches (12.7 cm) wide by 30-inches (76.2 cm) long anchored with one line of $\frac{1}{2}$-inch (1.27 cm) diameter, 2-inch (5.1 cm) splay diameter FRP anchors spaced according to Figure 3.27. Test three of specimen group B (Specimen B-Z-4-5-4) had one ply of bonded FRP 5-inches (12.7 cm) wide by 30-inches (76.2 cm) long anchored with one row of $\frac{1}{2}$-inch (1.27 cm) diameter, 4-inch (10.2 cm) splay diameter FRP anchors spaced according to Figure 3.27. Test four of specimen group B (Specimen B-W-2-5-4) had one ply of bonded FRP 5-inches (12.7 cm) wide by 12.5-inches (31.8 cm) long anchored with one line of $\frac{1}{2}$-inch (1.27 cm) diameter, 2-inch (5.1 cm) splay diameter FRP anchors spaced according to Figure 3.28. Test five of specimen group B (Specimen B-Z-4-5-6) had one ply of bonded FRP 5-inches (12.7 cm) wide by 30-inches (76.2 cm) long anchored with one row of $\frac{3}{4}$-inch (1.91 cm) diameter, 4-inch (10.2 cm) splay diameter FRP anchors spaced according to Figure 3.27. Test six of specimen group B (Specimen B-Y-2-5-4) had one ply of bonded FRP 5-inches (12.7 cm) wide by 30-inches (76.2 cm) long anchored with two rows of $\frac{1}{2}$-inch (1.27 cm), 2-inch (5.1 cm) splay diameter FRP anchors spaced according to Figure 3.29. Test seven of specimen group B (Specimen B-X-2-5-4) had one ply of bonded FRP 5-inches (12.7 cm) wide by 15-inches (38.1 cm) long anchored with two rows of $\frac{1}{2}$-inch (1.27 cm), 2-
inch (5.1 cm) splay diameter FRP anchors spaced according to Figure 3.30. Table 3.3 summarizes the characteristics for this group of specimens. Tests in this specimen group allowed for the determination of the shear strength and single anchor influence region on the FRP sheet.
**Figure 3.28 — Group B Anchor Pattern W**

- POST-TENSIONING DUCT
- FRP ANCHOR
- BONDED FRP
- UNBONDED FRP

**Figure 3.29 — Group B Anchor Pattern Y**

- POST-TENSIONING DUCT
- FRP ANCHOR
- BONDED LENGTH
Figure 3.30 — Group B Anchor Pattern X

Table 3.3 — Group B Specimen Properties

<table>
<thead>
<tr>
<th>Specimen</th>
<th>FRP Sheet Width [in]</th>
<th>Unbonded Length (leading end) [in]</th>
<th>Bonded Length [in]</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-Z-2-5-2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B-Z-2-5-4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B-Z-4-5-4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B-W-2-5-4</td>
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<td></td>
</tr>
<tr>
<td>B-Z-4-5-6</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>B-Y-2-5-4</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>B-X-2-5-4</td>
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<td></td>
<td></td>
</tr>
<tr>
<td></td>
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<td>5</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>15</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
3.4.4.3 Specimen Group C

The goal of specimen group C, which consisted of three tests conducted on two concrete blocks, studied the effects of using FRP anchors to fasten one ply of bonded FRP sheet having a 10-inch (25.4 cm) width. Concepts regarding FRP anchor diameter, FRP anchor splay diameter, FRP anchor length, FRP anchor spacing, and FRP bonded sheet length studied during the testing of specimen group B were applied to specimen group C to confirm that the theories developed in the experiments worked for a wider bonded FRP sheet. An FRP anchor length of 2-inches (5.1 cm) was kept constant throughout all tests as was done in tests for group B.

The three patterns tested in specimen group C, patterns X, Y, and U, were all used to verify the failure modes and behavior observed during the testing of specimen group B. Anchor patterns in this group were kept identical to anchor patterns in specimen group B while the FPR anchor sizes were varied according to Figure 3.31, Figure 3.32, and Figure 3.33. Bond length in group C anchor patterns X, Y, and U were kept identical to group B anchor patterns X and Y, whereas the bond width was doubled in specimen group C and the size of the FRP anchors was increased to attempt to achieve FRP rupture.
Test one of specimen group C (Specimen C-X-4-10-6) had one ply of bonded FRP 10-inches (25.4 cm) wide by 15-inches (38.1 cm) long anchored with two rows of $\frac{3}{4}$-inch (1.91 cm) diameter, 4-inch (10.2 cm) splay diameter FRP anchors spaced according to Figure 3.31. Test two of specimen group C (Specimen C-Y-4-10-6) had one ply of bonded FRP 10-inches (25.4 cm) wide by 30-inches (76.2 cm) long anchored with two rows of $\frac{3}{4}$-inch (1.91 cm) diameter, 4-inch (10.2 cm) splay diameter FRP anchors spaced according to Figure 3.32. Test three of specimen group C (Specimen C-U-2-10-4) had one ply of bonded FRP 10-inches (25.4 cm) wide by 30-inches (76.2 cm) long anchored with four rows of $\frac{1}{2}$-inch (1.27 cm), 2-inch (5.1 cm) splay diameter FRP anchors spaced according to Figure 3.33. Table 3.4 summarizes the characteristics for this group of specimens.

![Figure 3.31 — Group C Anchor Pattern X](image-url)
Figure 3.32 — Group C Anchor Pattern Y

Figure 3.33 — Group C Anchor Pattern U
Table 3.4 — Group C Specimen Properties

<table>
<thead>
<tr>
<th>Specimen</th>
<th>FRP Width [in]</th>
<th>Unbonded length (leading end) [in]</th>
<th>Bonded Length [in]</th>
</tr>
</thead>
<tbody>
<tr>
<td>C-X-4-10-6</td>
<td>10</td>
<td>5</td>
<td>15</td>
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<tr>
<td>C-Y-4-10-6</td>
<td>10</td>
<td>5</td>
<td>30</td>
</tr>
<tr>
<td>C-U-2-10-4</td>
<td>10</td>
<td>5</td>
<td>30</td>
</tr>
</tbody>
</table>

3.5 Instrumentation and Data Acquisition

In order to capture longitudinal strains, strain gages were bonded longitudinally and transversely on the FRP bonded sheet surface area. Within all specimens strain gages were placed on the fiber bundle rise to reduce the possibility of inaccurate strain gage measurements. Relative slip between the concrete block and FRP sheet was measured by position transducers placed near the centerline of the FRP sheet for Specimen group A, however, it was determined that the collected data was too erratic to provide meaningful results (see Appendix B and C) so this instrumentation was eliminated in subsequent tests. Specimen A-0-0-10-0 had strain gage and potentiometer spacing according to Figure 3.34. Specimen A-0-0-5-0 had strain gage and potentiometer spacing according to Figure 3.35.
Specimens in group B had strain gage spacing according to Figure 3.36, Figure 3.37, Figure 3.38, Figure 3.39, Figure 3.40, and Figure 3.41. After preliminary data reduction and calculations it was determined that a smaller amount of edge strain gages
were needed therefore Specimen B-Z-2-5-4 and all subsequent tests generally consisted of strain gages placed along the centerline of the FRP sheet only.

Figure 3.36 — Specimen B-Z-2-5-2 Gage Location

Figure 3.37 — Specimen B-Z-2-5-4 Gage Location
Figure 3.38 — Specimen B-Z-4-5-4 and B-Z-4-5-6 Gage Location

Figure 3.39 — Specimen B-W-2-5-4 Gage Location
Specimens in group C had strain gage spacing according to Figure 3.42, Figure 3.43, and Figure 3.44.
Figure 3.42 — Specimen C-X-4-10-6 Gage Location

Figure 3.43 — Specimen C-Y-4-10-6 Gage Location
Strain gage locations were identified according to the labeling shown in Figure 3.45 and Figure 3.46 to facilitate data interpretation and reduction. Strain gages were numbered according to the total number of strain gages on each individual specimen and not to absolute strain gage location. Position transducers were numbered sequentially starting from the free unloaded end of the FRP sheet.
3.6 Summary

The FRP material properties, specimen dimensions, strengthening systems, and strengthening configurations that were used to strengthen six reinforced concrete blocks are presented in this chapter. Three groups of specimens have been designated (A, B, and C). Illustrations of the test setup, specimen geometry, and concrete block reinforcement are presented.

Specimen group A was a control group to test debonding strengths of FRP sheets with no anchorage system other than the epoxy adhesive. Specimen group B tested debonding of 5-inch (12.7 cm) wide FRP sheets using single and double rows of ¼-inch (0.64 cm), ½-inch (1.27 cm), and ¾-inch (1.91 cm) diameter FRP anchors with 2-inch (5.1 cm) and 4-inch (10.2 cm) splay diameters. Specimen group C tested debonding of 10-inch (10.2 cm) wide FRP sheets using ½-inch (1.27 cm) and ¾-inch
(1.27 cm) diameter FRP anchors with 2-inch (5.1 cm) and 4-inch (10.2 cm) splay diameters. An FRP anchor length of 2-inches (5.1 cm) was kept constant throughout all tests as anchor length was determined to be a non-controlling factor due to the fact that anchor pullout was not observed in any of the initial test specimens. Illustrations of anchor hole patterns, data acquisition locations, and naming designations are presented.
CHAPTER 4

OBSERVED SPECIMEN RESPONSE

4.1 Introduction

The overall test results of the experimental program described in Chapter 3 are presented in this chapter. Results from concrete cylinder tests and FRP tensile coupons are also presented.

4.2 Concrete Cylinder Testing

Six reinforced concrete blocks were cast in two pours with three blocks per pour using a nominal 4000 psi (28 MPa) prepackaged concrete mix (Sakrete®). Following the manufacture’s recommendations, 9 pounds of water were approximately required for each 80-pound bag of Sakrete®. The exact amount of water varied depending on the air temperature and humidity during casting. Slump tests were conducted during each pour to ensure that a slump of 2-inches (5.1 cm) to 3-inches (7.6 cm) was obtained to ensure workability. Slump tests were conducted in accordance with ASTM C-143 standard test method. Three or four 4-inch (10.2 cm) by 8-inch (20.3 cm) cylinders were cast for each of the six blocks to verify the concrete compressive strength after 28 days of curing. Concrete cylinder test results are presented in Table 4.1 for all the test specimens. Concrete cylinder tests were conducted in accordance with ASTM C-39 standard test method.
Table 4.1 — 28-day Concrete Compressive Strengths

<table>
<thead>
<tr>
<th>Pour Number</th>
<th>Block Designation</th>
<th>Test Specimens</th>
<th>Cylinder Number</th>
<th>Compressive Strength, $f_c$ [psi]</th>
<th>Compressive Strength, $f'_c$ [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>A</td>
<td>B-Z-4-5-6, C-U-2-10-4</td>
<td>1</td>
<td>5966</td>
<td>41.1</td>
</tr>
<tr>
<td></td>
<td></td>
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Compressive Strength Average 5999.2 41.4

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<th>Pour Number</th>
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<th>Test Specimens</th>
<th>Cylinder Number</th>
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<th>Compressive Strength, $f'_c$ [MPa]</th>
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<td></td>
<td></td>
<td></td>
<td>4</td>
<td>5004</td>
<td>34.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Average</td>
<td>5208.3</td>
<td>35.9</td>
</tr>
</tbody>
</table>

Compressive Strength Average 4831.4 33.3

4.3 FRP Coupon Testing

Tensile testing FRP coupons fabricated in Gunness Structural Engineering Laboratory was conducted to validate the manufacturer-published FRP material properties. Ten 1-inch (2.5 cm) wide by 7-inch (17.8 cm) long FRP coupons were fabricated according to the manufacturer’s instructions.

FRP coupons were fabricated by cutting 1-inch (2.5 cm) wide by 7-inch (17.8 cm) long strips of carbon fibers. Four 1-inch (2.5 cm) square by 1/16-inch (0.16 cm)
thick steel tabs were bonded to each FRP coupon to be able to grip the FRP material with the testing machine without failing fibers locally. In order to saturate the dry carbon fibers, 5-inch (12.7 cm) long by 1-inch (2.5 cm) wide by 1/16-inch (0.16 cm) thick steel spacers were fabricated and wrapped in plastic sheathing to provide a working surface that would not bond to the epoxy during carbon fiber saturation. A 1/16-inch (0.16 cm) steel tab was then placed at each end of the steel spacer followed by applying a layer of epoxy saturant over the tabs and spacer (see Figure 4.1). A single 1-inch (2.5 cm) wide by 7-inch (17.8 cm) long FRP strip was then placed on the epoxy saturant followed by rolling out any trapped air bubbles. A 30-minute waiting period then followed to allow the epoxy saturant to impregnate into the carbon fiber strip. A second layer of epoxy saturant was then applied to the top of the carbon fiber strip followed by rolling out any trapped air bubbles to form the FRP composite. Lastly, one steel tab was placed at each end on top of the FRP strip therefore allowing the grips of the tensile testing machine to clasp the FRP coupon during loading. Figure 4.1 illustrates the process used to fabricate FRP coupons in this testing program.
Coupons were loaded until failure using an Instron 4400 load frame and 2-inch (5.1 cm) extensometer. The 2-inch (5.1 cm) extensometer was attached to the FRP coupon using two rubber o-rings (see Figure 4.2).
A data acquisition system was used to record displacement and load. Recorded data was used to construct stress-strain graphs that were utilized to calculate rupture stress, tensile modulus, and ultimate rupture strain of the FRP.

Table 4.2 — Mbrace® CF 130 Material Properties

<table>
<thead>
<tr>
<th>Material</th>
<th>Thickness</th>
<th>Tensile Strength</th>
<th>Tensile Modulus</th>
<th>Ultimate Rupture Strain</th>
<th>Poissons Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unidirectional Fiber</td>
<td>0.0065 in/ply [0.165 mm/ply]</td>
<td>550 ksi [3800 MPa]</td>
<td>33000 ksi [227 GPa]</td>
<td>1.67%</td>
<td>N/A</td>
</tr>
<tr>
<td>Epoxy Primer</td>
<td>5.9 mils [0.150 mm]</td>
<td>&gt;1740.5 psi [&gt;12 MPa]</td>
<td>&gt;101.5 ksi [&gt;700 MPa]</td>
<td>3%</td>
<td>N/A</td>
</tr>
<tr>
<td>Saturant</td>
<td>N/A</td>
<td>7252 psi [&gt;50 MPa]</td>
<td>435 ksi [&gt;3000 MPa]</td>
<td>2.5%</td>
<td>0.40</td>
</tr>
</tbody>
</table>

Failure of the FRP coupons was characterized by an abrupt brittle failure where splinters of the FRP exploded away from the Instron testing machine. Typically, only a few splinters would remain attached to the steel gripping tabs within the Instron machines grips (see Figure 4.3).

Figure 4.3 — Typical FRP Coupon Failure
In order to calculate the ultimate stress at failure a fiber thickness of 0.0065 in/pl (0.165 mm/ply) as provided by BASF® was used to calculate the equivalent fiber area. A typical stress-strain graph of the FRP coupons can be seen in Figure 4.4.

Complete results of FRP coupon tests are presented in Appendix A.

Before failure of the FRP coupon occurred individual FRP strand rupture was observed as seen in Figure 4.4. It was observed that the stress-strain curves had approximately the same slope before and after individual strand rupture, and remained constant until coupon failure. There was no apparent reduction of tensile modulus after individual strand rupture.

Table 4.3 lists the modulus of elasticity, ultimate stress, and ultimate strain for all ten FRP coupons that were tested to characterize the FRP material tensile properties for this research. It must be noted that the data from coupon FRP_H was not used when
computing the average ultimate stress, average ultimate strain, and tensile modulus values as slippage of the extensometer was observed visibly during this test causing erroneous data (see Figure 4.5).

![Figure 4.5 — FRP Extensometer Slip](image)

<table>
<thead>
<tr>
<th>Coupon Designation</th>
<th>Modulus [ksi]</th>
<th>$f_u$ [ksi]</th>
<th>$\varepsilon_u$ [in/in]</th>
</tr>
</thead>
<tbody>
<tr>
<td>FRP_A</td>
<td>37718</td>
<td>580.9</td>
<td>0.01611273</td>
</tr>
<tr>
<td>FRP_B</td>
<td>31764</td>
<td>461.7</td>
<td>0.015867648</td>
</tr>
<tr>
<td>FRP_C</td>
<td>40360</td>
<td>490.9</td>
<td>0.01277748</td>
</tr>
<tr>
<td>FRP_D</td>
<td>38810</td>
<td>392.6</td>
<td>0.011678284</td>
</tr>
<tr>
<td>FRP_E</td>
<td>35296</td>
<td>637.9</td>
<td>0.017115773</td>
</tr>
<tr>
<td>FRP_F</td>
<td>37619</td>
<td>492.1</td>
<td>0.012847185</td>
</tr>
<tr>
<td>FRP_G</td>
<td>36966</td>
<td>584</td>
<td>0.015459663</td>
</tr>
<tr>
<td>FRP_H</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>FRP_I</td>
<td>35402</td>
<td>656.3</td>
<td>0.017557138</td>
</tr>
<tr>
<td>FRP_J</td>
<td>36221</td>
<td>560.4</td>
<td>0.014892073</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>36684.0</strong></td>
<td><strong>539.6</strong></td>
<td><strong>1.49%</strong></td>
</tr>
</tbody>
</table>
A comparison between the FRP material properties provided by BASF® and the FRP coupons tested in the Gunness Structural Engineering Laboratory is provided in Table 4.4.

Table 4.4 — FRP Test Values versus Published Values

<table>
<thead>
<tr>
<th></th>
<th>Modulus [ksi]</th>
<th>$f_{tu}$ [ksi]</th>
<th>$\varepsilon_{tu}$ [in/in]</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Coupon Average</strong></td>
<td>36684.0</td>
<td>539.6</td>
<td>1.49%</td>
</tr>
<tr>
<td><strong>BASF</strong></td>
<td>33000.0</td>
<td>550.0</td>
<td>1.67%</td>
</tr>
<tr>
<td><strong>Absolute % Difference</strong></td>
<td>11.2%</td>
<td>1.9%</td>
<td>10.6%</td>
</tr>
</tbody>
</table>

4.4 Strengthening Effects of FRP Anchors

General observations on the effect that FRP anchors had on the behavior of specimens tested in this research project are presented in this section. The use of FRP anchors to fasten FRP composite sheets to reinforced concrete members subjected to direct shear proved to be an effective measure to prevent or delay the debonding failure mode of FRP laminates from the concrete surface. This result could be advantageously used in a wide variety of applications, particularly in concrete elements where limited bonding length is available or where loads are applied cyclically. In the experiments conducted for this research project, the utilization of FRP anchors increased the ultimate load capacity of all of the strengthened specimens in Groups B and C compared with the two control specimens in Group A. Details about the failure modes of each specimen are presented in section 4.6.

Fastening FRP composites with FRP anchors allowed development of higher interface shear stresses between the FRP sheets and the concrete surface. The ultimate strength of the FRP composite laminate was reached in multiple specimens as numerous FRP rupture failures were recorded. However, several failures in the FRP anchor
systems were also observed in a few specimens as test parameters were being varied to
the most efficient anchor configuration and anchor size. FRP anchor failure modes
appeared to be governed by the diameter of the FRP anchor splay, the diameter of the
FRP anchor, spacing of the FRP anchors, bonded length of the FRP sheet behind the
anchor, delamination between the FRP anchor splay and FRP composite sheet, and FRP
anchor pullout. When the correct FRP anchor splay diameter to anchor diameter ratio
was determined and an acceptable FRP anchor spacing was selected it was possible to
prevent debonding between the FRP sheet-concrete interfaces and to reach the rupture
strength of the FRP composite sheet. Table 4.5 presents data regarding the failure
observed in control and strengthened specimens in Groups A, B, and C. The ultimate
strength is defined as the FRP rupture strength based on the material test results
discussed in section 4.3.
Table 4.5 — Summary of Specimen Failure Loads and Modes

<table>
<thead>
<tr>
<th>Test Specimen</th>
<th>Test Failure Load, $P_{\text{test}}$ [k]</th>
<th>Ult. Strength, $P_{\text{ult.}}$ [k]</th>
<th>$P_{\text{test}}/P_{\text{ult.}}$ [%]</th>
<th>Length of Crack from End at Failure [in]</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-0-0-10-0</td>
<td>11.43</td>
<td>35.1</td>
<td>32.6%</td>
<td>16</td>
<td>Debonding</td>
</tr>
<tr>
<td>A-0-0-5-0</td>
<td>8.00</td>
<td></td>
<td>45.5%</td>
<td>5</td>
<td>Debonding</td>
</tr>
<tr>
<td>B-Z-2-5-2</td>
<td>10.18</td>
<td></td>
<td>57.8%</td>
<td>N/A</td>
<td>Anchor Shear, Debonding</td>
</tr>
<tr>
<td>B-Z-2-5-4</td>
<td>11.92</td>
<td></td>
<td>67.7%</td>
<td>9</td>
<td>FRP Rupture, Debonding</td>
</tr>
<tr>
<td>B-Z-4-5-4</td>
<td>11.01</td>
<td></td>
<td>62.6%</td>
<td>9</td>
<td>Anchor Shear, Debonding</td>
</tr>
<tr>
<td>B-W-2-5-4</td>
<td>9.28</td>
<td>17.6</td>
<td>52.7%</td>
<td>N/A</td>
<td>FRP Rupture, Debonding</td>
</tr>
<tr>
<td>B-Z-4-5-6</td>
<td>13.08</td>
<td></td>
<td>74.3%</td>
<td>16</td>
<td>FRP Rupture, Debonding</td>
</tr>
<tr>
<td>B-Y-2-5-4</td>
<td>12.42</td>
<td></td>
<td>70.6%</td>
<td>10</td>
<td>FRP Rupture, Debonding</td>
</tr>
<tr>
<td>B-X-2-5-4</td>
<td>13.61</td>
<td></td>
<td>77.3%</td>
<td>5</td>
<td>Debonding</td>
</tr>
<tr>
<td>C-X-4-10-6</td>
<td>19.69</td>
<td>35.1</td>
<td>56.1%</td>
<td>9</td>
<td>FRP Rupture, Debonding,</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Anchor Pullout, Delamination</td>
</tr>
<tr>
<td>C-Y-4-10-6</td>
<td>21.71</td>
<td></td>
<td>61.9%</td>
<td>12</td>
<td>FRP Rupture, Debonding,</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Delamination</td>
</tr>
<tr>
<td>C-U-2-10-4</td>
<td>29.00</td>
<td></td>
<td>82.6%</td>
<td>11</td>
<td>FRP Rupture</td>
</tr>
</tbody>
</table>

4.5 Observed Response of Specimens during Testing

Representative characteristics of the response of the test specimens are presented to illustrate features of the behavior of the FRP sheet-concrete interface to direct shear loading. Increased loading caused the initiation of debonding to occur at the end of the sheet closest to load application in all specimens. As a result of the two-
step wet-layup FRP application process, the composite sheet had an area of epoxy saturant on both sides of the carbon fiber sheet. These areas provided an easy means of tracking the crack front propagation across the width of the FRP sheet as the crack front moved from the loaded to unloaded end of the FRP composite sheet. As the crack front propagated towards the unloaded end of the composite sheet, whitening of the epoxy resin along the edges of the sheet was noticeable as seen in Figure 4.6.

![Cracking of Epoxy](image)

**Figure 4.6 — Cracking of Epoxy**

Propagation of the crack front towards the unloaded end of the composite sheet was accompanied by audible cracking and popping sounds indicating debonding of the composite sheet from the reinforced concrete substrate. During the testing of Specimen A-0-0-10-0 loading and data collection was stopped at intervals of 2-kips (8.9 kN) to observe the damage and mark cracks in the epoxy areas, however, during the load stops audible cracking and popping sounds were heard indicating that debonding was occurring during the load hold, therefore, all subsequent tests were loaded to failure with no load stops to capture the full debonding process.
Although still photography and video photography were used during the testing of all specimens, it was difficult to determine the exact propagation geometry of the composite sheet debonding, as the crack front did not propagate exactly perpendicular to the composite sheet fibers towards the unloaded end of the sheet. The propagation of the crack front proceeded in an erratic manner almost always propagating in a skewed mode. After reviewing the video photography of each test, debonding propagation was noticeable if the video was played extremely fast. The propagation of debonding was noticeable at the sheet edges and on the FRP sheet surface due to a visible change of the reflectiveness of the composite sheet as debonding propagated towards the unloaded end of the composite sheet.

Because of the erratic behavior of the crack front during debonding nearly all edge strain gages were removed in later tests. The focus was changed to measuring strain distribution at closer intervals along the centerline of the FRP composite sheet. During the propagation of debonding of the composite sheet global failure of the specimen is discussed in section 4.6. Following global failure of the specimens, pictures were taken and post-test inspections revealed the presence of pulverized concrete between the composite sheet and the top of the reinforced concrete specimen along with a 2-3 mm layer of concrete remaining attached to the underside of the composite sheet. This behavior is consistent with the debonding process of the composite sheet reported by others, as debonding has been shown to be a failure of the concrete and not the FRP laminate (Yao et al, 2005). This failure mode has been called an interface failure between FRP sheets and the concrete surface.
4.6 Observed Failure Modes of the Specimens

In this section the failure mode of each specimen is presented in detail. The sequence of the failure modes is discussed, the final state of the specimens after failure is presented, and the maximum load obtained is stated. The North, South, East, and West sides of the specimens are described according to Figure 4.7. Pictures denoted with “E” and “W” symbols orient the picture so that the loaded end of the sheets can be identified. FRP anchors denoted with a solid white block arrow correspond to the FRP anchor closest to the loaded end of the FRP sheet, FRP anchors denoted with a hollow white block arrow correspond to the FRP anchor nearest the unloaded end of the FRP laminate, and FRP anchors with a hollow dotted white block arrow correspond to a pair of adjacent FRP anchors. Details of all specimens’ geometry and instrumentation can be seen in section 3.4.4.

Figure 4.7 — Specimen Orientation
4.6.1 Control Specimens

This section describes the failure modes of control Specimens A-0-0-5-0 and A-0-0-10-0. The control specimens had no FRP anchors and a bond length of 30-inches (76.2 cm). The parameter studied in the controls specimens was bonded FRP width as seen in Figure 4.8.

![Figure 4.8 — Control Specimen Parameter](image)

4.6.1.1 Specimen A-0-0-10-0

Failure of Specimen A-0-0-10-0 occurred due to FRP debonding with a 2-3 mm layer of concrete remaining attached to FRP composite sheet (see Figure 4.10) indicating that the failure mode was cracking of the concrete under shear, described by Chen and Teng (2001) as an interface failure. During the testing of Specimen A-0-0-10-0 load stops were made at 2-kip (8.9 kN) intervals therefore it was possible to follow the propagation of the debonding crack front as it moved towards the unloaded end of the FRP sheet. Load stops were made at loads of 2-kips (8.9 kN), 4-kips (17.8 kN), and 6-kips (26.7 kN) however no visible signs of FRP debonding were noticeable. At the 8-
kip (35.6 kN) load stop it was evident that debonding had initiated and that the crack front had propagated approximately 1-inch (2.5 cm) towards the unloaded end of the FRP sheet as seen in Figure 4.9, however, debonding was only visible on the South side of the FRP composite sheet. As increased load was applied the crack front continued to propagate towards the free end of the FRP sheet. At the 10-kip (44.5 kN) load stop the crack front had propagated approximately 7.5-inches (19.1 cm) towards the unloaded end of the FRP sheet as seen in Figure 4.9, again however this was only visible on the South side of the FRP composite sheet. The final load stop occurred at 11-kips (48.9 kN) when it was observed that debonding had propagated approximately 14-inches (35.6 cm) on the South side of the FRP sheet and approximately 9-inches (22.9 cm) on the North side of the FRP laminate sheet as seen in Figure 4.10. Unsymmetrical debonding propagation is believed to have occurred due to irregular crack front propagation towards the free end of the FRP sheet either occurring from slightly non-uniform load application or varying local concrete material properties. Failure by debonding occurred with at a maximum load of 11.43-kips (50.8 kN). Following failure it was noticed that lumps of concrete were missing from the concrete specimen where the FRP sheet end had debonded. This phenomenon is believed to have occurred due to stress concentrations at the sheet end as explained by Chen et al., 2001.
4.6.1.2 Specimen A-0-0-5-0

Failure of Specimen A-0-0-5-0 occurred due to FRP debonding with a 2-3 mm layer of concrete remaining attached to the FRP composite sheet as seen in Figure 4.12. Load stops were not performed during testing of this specimen so it was not possible to correlate debonding propagation with applied load using still photography. As increased load was applied debonding initiated at the loaded end of the FRP laminate and propagated towards the unloaded end of the FRP laminate sheet. Similar to Specimen A-0-0-10-0, debonding propagation did not occur uniformly across the width of the sheet so debonding evidence on the North and South sides of the FRP laminate
sheet did not coincide at given cross sections as seen in Figure 4.11. Just before failure debonding had propagated to within 5-inches (12.7 cm) of the FRP sheet end as seen in Figure 4.12. Similar to Specimen A-0-0-10 concrete indentations were noticed on the surface of the concrete block near the end of the FRP sheet. Failure by debonding occurred at a maximum load of 8.00-kips (35.6 kN). It was observed that debonding propagated to within approximately 5-inches (12.7 cm) of the FRP sheet end before failure occurred (see Figure 4.12).

![Figure 4.11 — Specimen A-0-0-5-0 Debonding Propagation](image)

Figure 4.11 — Specimen A-0-0-5-0 Debonding Propagation
4.6.1.3 Summary

Failure of the control specimens was as expected. Both specimens failed due to FRP debonding. FRP anchors were used in subsequent specimens to allow development of higher forces in the composite sheets and improve the efficiency of this strengthening technique.

4.6.2 Specimens with Longitudinal FRP Anchors

This section describes the failure modes of specimens with FRP anchors spaced longitudinally along the FRP sheet length as seen in Figure 4.13. The bond length and width were kept constant at 5-inches (12.7 cm) and 30-inches (76.2 cm), respectively. The longitudinal distance between the anchors was 10-inches (25.4 cm) for all
specimens within the group. The parameters studied in this group were FRP anchor diameter and splay diameter.

![Figure 4.13 — Longitudinal FRP Anchors](image)

4.6.2.1 Specimen B-Z-2-5-2

Failure of Specimen B-Z-2-5-2, which had the FRP anchor pattern illustrated in Figure 3.27, was due to a combination of FRP debonding and FRP anchor shear. It was observed that a 2-3 mm layer of concrete remained attached to FRP laminate sheet at failure as seen in Figure 4.14. As increased load was applied debonding initiated at the loaded end of the FRP laminate sheet and propagated toward the unloaded end of the FRP laminate sheet as observed in control specimens A. The load at debonding initiation was estimated at approximately 6-kips (26.7 kN) after review of the video photography revealed a change in surface reflectivity of the sheet and because audible cracking and popping sounds were recorded. Once the debonding crack front approached the unloaded end of the specimen, sheet debonding and FRP anchor shear failure occurred almost simultaneously. Localized fiber rupture occurred at locations of
the FRP anchors as seen in Figure 4.15. A 1-inch (2.5 cm) wide piece of the FRP laminate sheet ruptured, but it is believed to have ruptured as the FRP laminate whip lashed after debonding failure (see Figure 4.15). Similar to Specimens A-0-0-10-0 and A-0-0-5-0 concrete divots were pulled from the concrete block near the FRP sheet end. Failure by debonding and anchor shear occurred at a maximum load of 10.18-kips (45.3 kN).

Figure 4.14—FRP Laminate and Concrete Specimen at Failure
4.6.2.2 Specimen B-Z-2-5-4

Failure of Specimen B-Z-2-5-4, which had the anchor pattern illustrated in Figure 3.27, was due to a combination of FRP debonding and FRP rupture. As increased load was applied debonding initiated at the loaded end of the FRP laminate and propagated towards the unloaded end of the FRP laminate sheet as seen in Figure 4.16. The load at debonding initiation was estimated between 5.5-kips (24.5 kN) and 6-kips (26.7 kN) after review of the video photography. Similar to Specimens A-0-0-5-0 and A-0-0-10-0 debonding propagation did not occur uniformly on the North and South sides of the FRP laminate sheet as seen in Figure 4.16. It was observed that debonding propagated to within approximately 9-inches (22.9 cm) of the FRP sheet end before failure occurred (see Figure 4.16). Failure of the specimen initiated by local rupture of a 1.75-inch (4.4 cm) width of FRP sheet in front of the leading FRP anchor (see Figure 4.17). Once localized rupture of the FRP sheet occurred, redistribution of forces to the North and South sides of the FRP sheet, caused edges of the sheet to debond. It was observed that a 2-3 mm layer of concrete remained attached to FRP sheet at failure as
seen in Figure 4.17. Failure by debonding and rupture occurred at a maximum load of 11.92-kips (45.3 kN).

Figure 4.16 — Specimen B-Z-2-5-4 Debonding Propagation
4.6.2.3 Specimen B-Z-4-5-4

Failure of Specimen B-Z-4-5-4, with the FRP anchor pattern illustrated in Figure 3.27, was due to a combination of FRP debonding and FRP anchor shear. A 2 to 3 mm layer of concrete remained attached to FRP laminate sheet at failure as seen in Figure 4.19. As increased load was applied debonding initiated at the loaded end of the FRP laminate sheet and propagated towards the unloaded end of the FRP laminate sheet. The load at debonding initiation was estimated at approximately 6.5-kips (28.9 kN) after reviewing the video photography. Unlike previous specimens debonding propagation occurred uniformly on the North and South sides of the FRP laminate sheet as seen in Figure 4.18. It was observed that debonding propagated to within approximately 9-inches (22.9 cm) of the FRP sheet end before failure occurred as seen in Figure 4.18. Once the debonding crack front reached the unloaded end of the
specimen, sheet debonding and FRP anchor shear failure occurred almost simultaneously. Localized fiber rupture occurred at locations of the FRP anchors as seen in Figure 4.19. Similar to previous specimens concrete divots were observed to be missing from the concrete block where the FRP sheet end was bonded. Failure by debonding and anchor shear occurred at a maximum load of 11.01-kips (49.0 kN).

Figure 4.18 — Specimen B-Z-4-5-4 Debonding Propagation
4.6.2.4 Specimen B-W-2-5-4

Due to the distinct anchor pattern and unbonded regions of specimen group B anchor pattern W, the failure sequence of Specimen B-W-2-5-4 was unique. Failure of Specimen B-W-2-5-4 was due to a combination of FRP debonding and FRP rupture. As increased load was applied two longitudinal cracks, noticeable due to the whitening of the epoxy saturant used to impregnate the FRP laminate, were visible starting at the East (loaded end) of the specimen and running towards the West unloaded end of the specimen (see Figure 4.20). The propagation of each crack occurred between carbon fiber bundles that passed underneath the outermost edges of the leading FRP anchor splay with the distance between the two FRP cracks being approximately 2-inches (5.1 cm). It was observed that each crack propagated approximately halfway between the two FRP anchors (see Figure 4.20). After reviewing the video photography, at a load of
approximately 9-kips (40.0 kN) a loud audible cracking and popping sound was heard as well as a 2-kip (8.9 kN) drop in load believed to be caused by failure of the leading FRP anchor closest to the applied load. Following failure of the leading FRP anchor, debonding initiated on the bonded area of the FRP laminate on the North side of the specimen as seen in Figure 4.21. Soon after debonding initiated, failure of the specimen due to localized FRP rupture of approximately 2-inches (5.1 cm) width in front of the leading FRP anchor and debonding of the non-ruptured fibers, which were to the North and South sides of the FRP anchors, occurred at a maximum load of 9.28-kips (41.3 kN) (see Figure 4.21). It was observed that a 2-3 mm layer of concrete remained attached to debonded FRP sheet at failure as seen in Figure 4.21.
4.6.2.5 Specimen B-Z-4-5-6

Failure of Specimen B-Z-4-5-6, which had an anchor patterning according to Figure 3.27, was due to a combination of FRP debonding and FRP rupture. As increased load was applied debonding initiated at the loaded end of the FRP laminate and propagated towards the unloaded end of the FRP laminate sheet as seen in Figure 4.22. Similar to Specimen B-Z-4-5-4 debonding propagation occurred uniformly on the North and South sides of the FRP laminate sheet as seen in Figure 4.22. It was observed that debonding propagated to within approximately 16-inches (40.6 cm) of the FRP sheet end before failure occurred (see in Figure 4.22). Failure of the specimen initiated by local rupture of a 3-inch (7.6 cm) width of FRP sheet in front of the leading FRP anchor (see Figure 4.23). Once localized rupture of the FRP sheet occurred,
redistribution of forces to the North and South sides of the FRP sheet, caused edges of the sheet to debond. Minimal localized delamination also occurred during the failure sequence between both FRP anchor splays and the FRP laminate sheet (see Figure 4.23). It was observed that a 2-3 mm layer of concrete remained attached to FRP sheet at failure as seen in Figure 4.23. Failure by debonding and rupture occurred at a maximum load of 13.08-kips (58.2 kN).

Figure 4.22 — Specimen B-Z-4-5-6 Debonding Propagation
4.6.2.6 Summary

The failure of specimens with longitudinal FRP anchors illustrated that the FRP splay diameter and anchor diameter were important factors in determining the shear strength of an FRP anchor and the width of the FRP sheet engaged. Due to the prescribed failure modes it was determined that the trailing FRP anchor was not necessary to obtain FRP rupture. Furthermore, the failure modes indicated that only a region in the FRP composite sheet of approximately equal width to the FRP anchor splay was effectively engaged by the leading anchor.

4.6.3 Specimens with Transverse FRP Anchors

This section describes the failure modes of specimens with FRP anchors spaced transversely across the FRP sheet width as seen in Figure 4.24. The parameters studied
in this group were FRP anchor diameter, splay diameter, bond length, and bond width. Subsequent to the testing of specimens with longitudinal FRP anchors it was decided to test specimens strictly with transverse FRP anchors to investigate if FRP rupture across the full sheet width could be obtained by engaging the entire width of sheet with several anchors placed transversely. Additionally, the trailing anchor was eliminated from the tests because previous tests indicated that most of the force in the FRP sheet developed in front of the leading anchor.

**Figure 4.24 — Transverse FRP Anchors**

4.6.3.1 Specimen B-Y-2-5-4

Failure of Specimen B-Y-2-5-4, which had the anchor pattern illustrated in Figure 3.29, was due to a combination of FRP debonding and FRP rupture. As increased load was applied debonding initiated at the loaded end of the FRP laminate and propagated towards the unloaded end of the FRP laminate sheet as seen in Figure 4.25. The load at debonding initiation was estimated at approximately 6-kips (26.7 kN) after review of the video photography. Similar to Specimens B-Z-4-5-4 and B-Z-4-5-6 debonding propagation occurred uniformly on the North and South sides of the FRP
laminate sheet as seen in Figure 4.25. It was observed that debonding propagated approximately 10-inches (25.4 cm) from the loaded end before specimen failure (see Figure 4.25). Failure of the specimen initiated by local rupture of a 3.75-inch (9.5 cm) width of FRP sheet in front of the FRP anchors (see Figure 4.26). Once localized rupture of the FRP sheet occurred, redistribution of forces to the north and south sides of the FRP sheet, caused edges of the sheet to debond. It was observed that a 2-3 mm layer of concrete remained attached to FRP sheet at failure as seen in Figure 4.26.

Failure by debonding and rupture occurred at a maximum load of 12.42-kips (55.2 kN).

Figure 4.25 — Specimen B-Y-2-5-4 Debonding Propagation
Failure of Specimen B-X-2-5-4, with the anchor pattern illustrated Figure 3.30, was due to a combination of FRP debonding, FRP rupture, and delamination between the FRP anchor splay and FRP laminate sheet. As increased load was applied debonding initiated at the loaded end of the FRP laminate and propagated towards the unloaded end of the FRP laminate sheet as seen in Figure 4.27. The load at debonding initiation was estimated at approximately 5.5-kips (24.5 kN) after review of the video photography. Similar to previous specimens debonding propagation occurred uniformly on the North and South sides of the FRP laminate sheet as seen in Figure 4.27. It was observed that debonding propagated to within approximately 5-inches (12.7 cm) from the unloaded end before specimen failure. Failure of the specimen
initiated with debonding of the FRP laminate sheet from the concrete surface followed by localized fiber rupture around the splay region of the North FRP anchor as well as delamination between the South FRP anchor splay and FRP laminate sheet (see Figure 4.28). Due to delamination and rupture failure between the FRP anchors and the laminate sheet, it was noticed that splitting of the FRP laminate sheet occurred at the location of the FRP anchors as seen in Figure 4.28. It was observed that a 2-3 mm layer of concrete remained attached to FRP laminate sheet at failure as seen in Figure 4.28. Failure occurred at a maximum load of 13.61-kips (60.5 kN).

Figure 4.27 — Specimen B-X-2-5-4 Debonding Propagation
4.6.3.3 Specimen C-X-4-10-6

Failure of Specimen C-X-4-10-6, with the FRP anchor pattern illustrated in Figure 3.31, was due to a combination of FRP debonding, FRP anchor shear, delamination between the FRP anchor splays and FRP laminate sheet, and FRP anchor pullout. As increased load was applied debonding initiated at the loaded end of the FRP laminate and propagated towards the unloaded end of the FRP laminate sheet as seen in Figure 4.29. The load at debonding initiation was estimated at approximately 9-kips (40.0 kN) after review of the video photography. It was observed that debonding propagated approximately 9-inches (22.9 cm) from the loaded end before specimen failure. Although it is not exactly known, failure of the specimen is believed to have initiated as a debonding failure between the FRP laminate and the concrete substrate. Along with the debonding failure, delamination occurred between the South FRP anchor splay and the FRP laminate sheet as well as shearing of the South FRP anchor as
seen in Figure 4.29 and Figure 4.30. It was observed that FRP anchor pullout occurred on the North FRP anchor as seen in Figure 4.29. It was observed that a 2-3 mm layer of concrete remained attached to FRP laminate sheet at failure as seen in Figure 4.29.

Failure occurred at a maximum load of 19.69-kips (87.6 kN).

Figure 4.29 — Specimen C-X-4-10-6 Debonding (right) and at Failure (left)

Figure 4.30 — Specimen C-X-4-10-6 Anchor Failures
4.6.3.4 Specimen C-Y-4-10-6

Failure of Specimen C-Y-4-10-6, which had the FRP anchor pattern illustrated in Figure 3.32, was due to a combination of FRP debonding, FRP rupture, and delamination between the FRP anchor splays and FRP laminate sheet. As increased load was applied debonding initiated at the loaded end of the FRP laminate and propagated towards the unloaded end of the FRP laminate sheet as seen in Figure 4.31. It was observed that debonding propagated to within approximately 12-inches (30.5 cm) from the unloaded end before specimen failure. Although it is not exactly known, failure of the specimen is believed to have initiated as a delamination failure between the FRP laminate and the FRP anchors (see Figure 4.32). Along with delamination failure, debonding occurred between the FRP laminate and the concrete substrate. FRP rupture occurred at the North FRP anchor splay region as seen in Figure 4.32, where fiber rupture in this region occurred in the FRP laminate sheet as well as the FRP anchor splay. Following failure of the specimen it was also noted that minor FRP rupture occurred at the location of the South FRP anchor. Similar to all other specimens it was observed that a 2-3 mm layer of concrete remained attached to FRP laminate sheet at failure. Failure occurred at a maximum load of 21.71-kips (96.6 kN).
4.6.3.5 Specimen C-U-2-10-4

Failure of Specimen C-U-2-10-4, which had the anchor pattern illustrated in Figure 3.33, was due to FRP rupture. As increased load was applied debonding initiated at the loaded end of the FRP laminate and propagated towards the unloaded
end of the FRP laminate sheet as seen in Figure 4.33. The load at debonding initiation was estimated at approximately 10-kips (44.5 kN) after review of the video photography. Failure by FRP rupture occurred at a maximum load of 29.00-kips (129.0 kN) as seen in Figure 4.34. Due to specimen failure by FRP rupture, it was possible to closely track the crack front propagation after failure by using a quarter to lightly tap on the FRP laminate sheet to detect audibly how far the crack front had progressed. By tapping on the FRP laminate sheet it was possible to distinguish between the bonded and unbonded FRP laminate sheet as debonded FRP had a hollow sound when struck. It was observed that debonding propagated approximately 11-inches (27.9 cm) from the loaded end before specimen failure as seen in Figure 4.34. Similar to all other specimens a 2-3 mm layer of concrete remained attached to FRP laminate sheet at failure.

Figure 4.33 — Specimen C-U-2-10-4 Debonding Propagation
4.6.3.6 Summary

The failure of specimens with transverse FRP anchors illustrated that the FRP splay diameter, anchor diameter, and bond length were important factors in determining the shear strength of an FRP anchor, the width of the FRP sheet engaged, and the ductility of the debonding process. Further analysis of the observed failure modes with a discussion comparing different specimens is presented in Chapter 5.

4.7 FRP Strain Measurements

The measured strain readings on the surface of the FRP sheet are presented in this section. The typical strain variation versus load is first presented for each specimen, followed by strain profiles across the FRP composite sheets width, and along
the FRP laminates length. The location of the strain gages for each individual specimen and a strain gage designation key was presented in section 3.5. Strain gage columns and rows are described according to Figure 4.35. Strain gages were numbered according to the total number of strain gages on each individual specimen and not to absolute strain gage location. The strain-load plots of all specimen strain gages are presented in Appendices D through O.

**Figure 4.35 — Strain Gage Designation**

4.7.1 FRP Strain vs. Applied Load

In this section typical strain versus load graphs are presented for representative specimens for Groups A, B, and C. Two graphs are presented for each selected specimen, the first plotting a column of strain gages located near the loaded end of the FRP sheet and the second plot graphing a column of strain gages located near the free end of the FRP laminate. Strain versus load plots were chosen based upon the ability of
the plot to depict the typical behavior measured in numerous strain gages within each specimen group.

4.7.1.1 Specimen Group A

Figure 4.36 presents typical load-strain curves as recorded by gages A4, B4, and C4 (strain gage column 4) on Specimen A-0-0-5-0. The load-strain plot is composed of approximately three regions designated A, B, and C. The first region, A, is linear and corresponds to the approaching crack front as debonding propagated towards the strain gage column. In region B the strain increased in the FRP with virtually no increase in load indicating that debonding occurred at approximately 6.75-kips (30.0 kN). The final region of the load-strain plot, region C, is approximately linear indicating that the debonding crack front propagated past the strain gages. Region C, which is linear until failure, displays the linear elastic behavior of FRP.
Figure 4.36 — Specimen A-0-0-5-0 Column 4 Strain

Figure 4.37 presents typical load-strain curves as recorded by strain gages in column 2 of Specimen A-0-0-5-0. The behavior of strain gage column 2, which was located 15-inches (38.1 cm) from the free end of the FRP sheet, is identical to column 4 gages except only a small region of linearly increasing FRP strain is observed in region C since failure occurred by debonding almost immediately following the propagation of the crack front past the strain gage column. It is interesting to note that the length of the strain plateau was very similar with the highest measured strains between 1.5 mε and 3.5 mε. The strain gage registering the highest recorded strain, however, was not consistent among FRP sheet sections indicating high variability obtained when measuring strains experimentally after the debonding process has initiated.
The load-strain curves of Specimen A-0-0-10-0 exhibit almost identical strain behavior to that observed in Specimen A-0-0-5-0. The load-strain plots are composed of approximately 3 regions designated as A, B, and C. A summary of the load, strain, and gage location associated with regions A, B, and C for Specimens A-0-0-5-0 and A-0-0-10-0 is presented in Table 4.6. The load-strain plots of all strain gages for Specimens A-0-0-5-0 and A-0-0-10-0 are presented in Appendices D and E.
4.7.1.2 Specimen Groups B and C

Figure 4.38 presents typical load-strain curves as recorded by strain gages in column 5 of Specimen C-X-4-10-6, which were located 2.5-inches (6.4 cm) towards the applied load in front of the FRP anchors. The load-strain plots exhibit almost identical strain behavior to that observed in the control specimens and are composed of approximately the same three regions designated A, B, and C as previously discussed.

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<th>Strain [mε]</th>
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Figure 4.39 illustrates the load-strain curves of strain gages in column 1 of Specimen C-X-4-10-6. The curve exemplifies behavior prior and subsequent to initiation of debonding at this section. Region C, however, is not observed in this plot because failure of the FRP sheet by detachment from the concrete surface followed immediately after propagation of the crack front past gage column 1.
The load-strain curves of specimens in groups B and C exhibit almost identical strain behavior to that observed in Specimen C-X-4-10-6. The load-strain plots are composed of approximately 3 regions designated as A, B, and C. A summary of the load, strain, and gage location associated with regions A, B, and C for specimens in Groups B and C, with the exception of Specimen B-W-2-5-4, are presented in Table 4.7. The load-strain plots of all strain gages for specimens in Groups B and C are presented in Appendices F through O.

After reviewing Table 4.6 and Table 4.7 two fundamental trends can be observed from the load-strain data. First, all specimens maximum strain (highlighted in bold italics) occurred at the maximum load in region C at the leading strain gage closest to the loaded end of the FRP sheet. This observation agrees with the failure modes since FRP rupture occurred in front of the leading FRP anchors near the location of the
leading strain gages. Second, the maximum strain in the trailing gage (highlighted in italics) always occurred in region C, however the maximum value in the trailing gage is consistently lower than the maximum value in the leading strain gage. The difference between the two gage maximums within an individual specimen is most obvious for specimen Groups B and C. This observation illustrates the localized effects that FRP anchors had on the FRP sheet closest to the loaded end. At the failure load in specimen Groups B or C, the FRP anchors fastened the FRP sheet in front of the FRP anchors to the point of maximum strain. However, behind the FRP anchors towards the unloaded end of the FRP laminate the FRP anchors had little influence on the propagation of debonding.

The maximum strain variability in Table 4.7 should also be noted. The maximum strain at specimen failure is highly variable between specimens, with the minimums occurring in two of the specimens that failed primarily due to FRP rupture, Specimens B-Y-2-5-4 and C-U-2-10-4. The variability illustrates the extremely localized effect of FRP anchors.
Table 4.7 — Specimen Groups B and C Load-Strain Data

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4.7.1.3 Specimen B-W-2-5-4

Due to the unique anchor arrangement of Specimen B-W-2-5-4, with setup and strain gage locations presented in section 3.5, the corresponding load-strain plots do not resemble any of the previously discussed load-strain diagrams in other specimens.
Figure 4.40 presents a load-strain curve as recorded by strain gage B10 for Specimen B-W-2-5-4. The load-strain diagram is representative of the plots of gage columns 6, 7, 8, 11, and 12. The graph illustrates a linear line until failure, which corresponds to the observed failure mode of the specimen since none of the gages in columns 6, 7, 8, 11, or 12 were attached to FRP that was bonded to the concrete surface. Strain gage columns 6, 7, 8, 10, 11, and 12 were all attached to unbonded FRP therefore linear elastic behavior was observed until failure.

![Figure 4.40 — Specimen B-W-2-5-4 Column 10 Strain](image)

Figure 4.40 presents the load-strain curve as recorded by gage column B9. Similar to strain gages B6, B7, B8, B10, B11, and B12, strain gage B9 was located in the middle of the FRP sheet in line with the two FRP anchors. Figure 4.41 illustrates linear behavior of strain gage B9 (region A) similar to Figure 4.40. Strain gages C9 and A9, which were located on the edges of the leading FRP anchor, indicate different
behavior. Load-strain plots for strain gages A9 and C9 show two regions designated B and C. Region B is linear, however the slope of the line is much greater than that of strain gage B9. Following region B a load drop occurred due to the failure of the leading FRP anchor as described in section 4.6.2.4. Subsequent to the load drop, region C illustrates a linear behavior with increasing strain and load along the same slope as region B until failure.

Figure 4.40 and Figure 4.41 lead to the concept that FRP anchors engage an effective width of FRP sheet approximately equal to the splay diameter of the anchor. Figure 4.41 illustrates that a larger strain was attained in strain gage B9, which was located in line with the FRP anchors, compared to strain gages A9 and C9, which were located outside of the splay region of the FRP anchor. Strain in the FRP sheet not within the splay region diameter was developed in the FRP sheet due to bond between the laminate and the concrete surface within the 12.5-inch (31.8 cm) bonded region near the unloaded end of the FRP sheet.
Figure 4.42 illustrates the load-strain diagram for strain gage column 3 located on the bonded FRP sheet, which shows pre-debonding and debonding regions similar to previously discussed bonded FRP strain gages. The initial region, A, is linear and shows no strain development in the FRP until approximately 9-kips (40.0 kN) at which time the leading FRP anchor failed. Quickly following the load drop associated with anchor failure, region B illustrates linearly increasing load and FRP strain. The final region, C, shows a large increase in FRP strain at failure.
4.7.2 FRP Strain across Laminate Width

In this section strain profiles across the FRP composite sheet width are presented. Strain profile plots across the FRP sheet width were studied to provide insight into the effects of FRP anchor position on strain development in unanchored edge regions. The influence of the FRP anchor position on observed failure modes were also investigated and are discussed here. Strain gage column and row notation is described according to Figure 4.35.

4.7.2.1 Specimen A-0-0-10-0

As described in section 4.6.1.1 the propagation of debonding occurred in an erratic manner during the failure of Specimen A-0-0-10-0. During load stops it was
observed that a greater length of sheet had debonded on the South side of the FRP composite laminate as seen in Figure 4.43.

![Figure 4.43 — FRP Debonding at 11-kips (48.9 kN)](image)

At an applied load of 11.10-kips (48.4 kN), which is just before the failure load of 11.43-kips (50.8 kN), the strain gage readings across the FRP composite sheet are presented in Figure 4.44. It should be noted that strain gage C6 was not plotted on the graph as the gage malfunctioned and recorded erroneous data.
Strain across the FRP composite sheets width presented in Figure 4.44 agrees with the observed propagation of debonding presented in Figure 4.43. The most noticeable observation is that strain gage column 4, which was located 10-inches (25.4 cm) from the loaded end of the FRP laminate has a skewed strain profile that shows increased strain on the South side of the FRP laminate and virtually no strain on the North side of the FRP sheet, which agrees with the observed failure mode as debonding had propagated to approximately 9-inches (22.9 cm) on the North side of the FRP sheet and had not reached the gage location.

Also noticeable is the unique strain profile of strain gage column 5. According to Figure 4.43 it appears that debonding propagated past strain gage column 5 at 11.10-kips (48.4 kN), which was located 5-inches (12.7 cm) from the loaded end of the FRP sheet, therefore it would be expected that the strain profile would be a horizontal line.
indicating uniform strain across the FRP composite sheets width. However, Figure 4.44 shows a maximum strain of 3.57 m\(\varepsilon\) in strain gage B5 and decreased values on the North and South sides of the composite sheet.

The last observation that can be seen in Figure 4.44 is that strain gage columns 1, 2, and 3, which were located 25-inches (63.5 cm), 20-inches (50.8 cm), and 15-inches (38.1 cm), from the loaded end of the FRP sheet, had no appreciable strain. This agrees with Figure 4.43 as the propagation of debonding was observed to have reached a maximum of 14-inches (35.6 cm) on the South side of the FRP sheet.

4.7.2.2 Specimen A-0-0-5-0

Typical strain profiles across the FRP composite sheets width for Specimen A-0-0-5-0 are presented in Figure 4.45 at a failure load of 7.83-kips (34.8 kN). It should be noted that strain gage A1 was not plotted on the graph as the strain gage malfunctioned and recorded erroneous data.
In view of the fact that load stops were not performed during the testing Specimen A-0-0-5-0 or any of the subsequent tests it is impossible to correlate FRP strain profiles to the observed propagation of debonding.

The typical strain profiles presented in Figure 4.45, which were recorded just before the failure load of 8.00-kips (35.6 kN), show that all of the strain gages in the specimen recorded strain indicating that the debonding crack front either passed the strain gage column line or was approaching the column position. As discussed in section 4.6.1.2, it was observed that debonding of the FRP laminate occurred erratically, which is consistent with Figure 4.45 that shows non-uniform strain distributions across the FRP sheet width along each of the gage columns.
4.7.2.3 Specimen B-Z-2-5-2

Typical transverse strain profiles on the FRP sheet for Specimen B-Z-2-5-2 are presented in Figure 4.46, which were recorded just before the failure load of 10.18-kips (45.3 kN).

![Figure 4.46 — Specimen B-Z-2-5-2 Strain vs. Width](image)

Figure 4.46 illustrates that just before specimen failure, strain was recorded in all of the strain gages indicating that the debonding crack front had approached unloaded end of the FRP sheet, which is consistent with the specimen failure mode described in section 4.6.2.1.

Strain gage columns 1, 3, 4, and 5 all had peak strains occurring in the strain gages located in row B directly in line with the FRP anchor as described in section 3.5. The increased value of strain recorded in row B occurred due to local effects of the FRP anchors. The FRP anchors were effective in securing the FRP sheet within the anchor.
splay diameter, however, decreased values of strain were recorded in the A and C rows of strain gages that were bonded in the edge regions outside of the FRP anchor splay.

4.7.2.4 Specimen B-Z-2-5-4

Typical transverse strain profiles on the FRP sheet for Specimen B-Z-2-5-4 are presented in Figure 4.47. In Specimen B-Z-2-5-4 it was decided to place strain gages across the width of the FRP sheet only at column 9 due to the erratic behavior of the crack front during debonding propagation. It was also considered more important to develop a closer strain gage array along the FRP sheet centerline to capture the longitudinal distribution of strains in better detail than previous specimens. Column 9 was located 5-inches (12.7 cm) from the loaded end of the FRP sheet in front of the leading FRP anchor. Failure of Specimen B-Z-2-5-4 occurred at 11.92-kips (45.3 kN).
A large strain increase occurred at approximately 6.03-kips (26.8 kN) indicating the initiation of debonding at the strain gage column. Following debonding a constant increase in strain occurred across the FRP sheet width to failure. The maximum strain occurred in strain gage row B. It is presumed that localized FRP anchor, bond, or load effects caused increased strain to occur in strain gage C9 compared to A9. Neither strain gage A9 or C9 is believed to have malfunctioned since strain in both gages increased incrementally with applied loading.

4.7.2.5 Specimen B-Z-4-5-4

Typical transverse strain profiles on the FRP sheet for Specimen B-Z-4-5-4 at column 8 are presented in Figure 4.48. Column 8 was located 2.5-inches (6.4 cm) from the loaded end in front of the leading FRP anchor. Failure of Specimen B-Z-2-5-4 occurred at 11.01-kips (49.0 kN).
Similar to Specimen B-Z-2-5-4 the maximum strains occurred in strain gage row B, which were bonded to the centerline of the FRP sheet directly in line with the FRP anchors. A large strain increase occurred at approximately 7.07-kips (31.4 kN) indicating debonding initiation at the strain gage column. Following debonding a constant increase in strain occurs across the FRP sheet width to failure. It is assumed that localized FRP anchor, bond, or load effects caused increased strain to occur in strain gage C9 compared to A9.

4.7.2.6 Specimen B-W-2-5-4

Due to the unique arrangement of Specimen B-W-2-5-4, which setup and strain gage locations were presented in section 3.5, two strain vs. FRP width diagrams are presented. Figure 4.49 illustrates the FRP strain at loads ranging from 5-kips (22.2 kN).
to 8-kips (35.6 kN). The maximum strains occurred in strain gage row B, which were bonded on the centerline of the FRP sheet directly in line with the FRP anchors. Strain in gage row B can be directly attributed to the ability of the FRP anchors to fasten the FRP sheet. Strain gage rows A and C, which were attached to unbonded FRP, recorded strain believed to occur because the rear end of the FRP sheet was bonded to the concrete surface, which provided resistance to unbonded edge regions outside of the FRP anchor splay.

Figure 4.49 — Specimen B-W-2-5-4 Strain vs. Width

Figure 4.50 illustrates the FRP strain at loads ranging from 9-kips (40.0 kN) to specimen failure. Consistent with Figure 4.49 maximum strain was recorded in strain gage row B, while decreased strain values were recorded along strain gage row A and C, which were located on the edge regions outside of the FRP anchor splay.
Consistent with the failure mode of Specimen B-W-2-5-4 described in section 4.6.2.4, at \( P=9.42 \)-kips (41.9 kN) a large strain and load drop was recorded in strain gage B9 indicating that the leading FRP anchor had failed. Following the load and strain drop as loading increased and debonding initiated in the bonded FRP laminate strain continued to increase in strain gages A9 and C9 to specimen failure.

![Figure 4.50 — Specimen B-W-2-5-4 Strain vs. Width](image)

**4.7.2.7 Specimen B-Z-4-5-6**

Typical strain profiles across the width of the FRP sheet width for Specimen B-Z-4-5-6 at column 8 are presented in Figure 4.51. Column 8 was located 2.5-inches (6.4 cm) from the loaded end in front of the leading FRP anchor. Failure of Specimen B-Z-4-5-6 occurred at 13.08-kips (58.2 kN).
Similar to previous specimen strain profiles, column 8 strain gages exhibited incremental strain increase with applied loading. Unlike previous specimens the strain profiles of Specimen B-Z-4-5-6 illustrate that the maximum strain occurred in strain gage A8, which was located on the North side of the FRP sheet within a region of bonded FRP that failed due to FRP rupture. Strain gage C8, which was located on the South side of the FRP sheet within a region of bonded FRP that failed due to a splay delamination and debonding as discussed in section 4.6.2.5, recorded reduced strain values. The reduced recordings of strain gage C8 are consistent with the observed failure mode and the inability of the FRP anchor splay region to remain bonded to the FRP laminate sheet. It is assumed that localized FRP anchor, bond, or load effects caused increased strain to occur in gage A8 compared to B8 and C8.
4.7.2.8 Specimen B-Y-2-5-4

Typical transverse strain profiles on the FRP sheet for Specimen B-Y-2-5-4 at column 7 are presented in Figure 4.52. Column 7 was located 3-inches (7.6 cm) from the loaded end in front of the leading FRP anchor. Failure of Specimen B-Y-2-5-4 occurred at 12.42-kips (55.2 kN).

![Figure 4.52](image)

**Figure 4.52 — Specimen B-Y-2-5-4 Strain vs. Width**

Figure 4.52 illustrates that the maximum strain occurred in strain gage A7 with linearly decreasing strain across the FRP laminate sheet. The strain profiles are consistent with a skewed debonding crack front across the width of FRP sheet.

A large strain increase occurred at approximately 6.96-kips (31.0 kN) indicating the initiation debonding at the strain gage column. It is assumed that localized FRP anchor, bond, or load effects caused increased strain to occur in gage A7 compared to
B7 and C7. Following debonding a constant increase in strain occurs across the FRP sheet width to failure.

4.7.2.9 Specimen B-X-2-5-4

Typical transverse strain profiles on the FRP sheet for Specimen B-X-2-5-4 at column 4 are presented in Figure 4.53. Column 4 was located 3.5-inches (8.9 cm) from the loaded end in front of the leading FRP anchor. Failure of Specimen B-X-2-5-4 occurred at 13.65-kips (60.5 kN).

The strain profiles illustrated in Figure 4.53 are approximately horizontal illustrating uniform strain distribution across the FRP sheet width. The slightly increased value of strain in gages A4 and C4 compared to gage B4 illustrates the very localized effect that FRP anchors have since gages A4 and C4 were located closer to the
center of the FRP anchor than gage B4, which was located at the edge of the anchor splay region.

A large strain increase occurred at approximately 6.15-kips (27.4 kN) indicating the initiation debonding at the strain gage column. Following debonding a constant increase in strain occurs across the FRP sheet width to failure.

4.7.2.10 Specimen C-X-4-10-6

Typical transverse strain profiles on the FRP sheet for Specimen C-X-4-10-6 at column 6 are presented in Figure 4.54. Column 6 was located 2.5-inches (6.4 cm) from the loaded end in front of the leading FRP anchor. Failure of Specimen C-X-4-10-6 occurred at 19.69-kips (87.6 kN).

![Figure 4.54 — Specimen C-X-4-10-6 Strain vs. Width](image)
Figure 4.54 illustrates that the maximum strain occurred in strain gage C6, with approximately linearly decreasing strain across the FRP laminate sheet. It is believed that localized FRP anchor, bond, or load effects and a skewed debonding crack front caused maximum strains to occur in gage C6 compared to gages A6 and B6.

A large strain decrease occurred at approximately 8.04-kips (35.8 kN) believed to occur do to the initiation debonding at the strain gage column. Following debonding a constant increase in strain occurs across the FRP sheet width to failure.

4.7.2.11 Specimen C-Y-4-10-6

Typical transverse strain profiles on the FRP sheet for Specimen C-Y-4-10-6 at column 9 are presented in Figure 4.55. Column 9 was located 2.5-inches (6.4 cm) from the loaded end in front of the leading FRP anchor. Failure of Specimen C-Y-4-10-6 occurred at 21.71-kips (96.6 kN).
The strain profiles illustrated in Figure 4.55 are approximately horizontal illustrating uniform strain distribution across the FRP sheet width. It is believed that localized FRP anchor, bond, or load effects and a skewed debonding crack front caused maximum strains to occur in gage A9 compared to gages B9 and C9.

A large strain increase occurred at approximately 11.97-kips (53.2 kN) indicating the initiation debonding at the strain gage column. Following debonding a constant increase in strain occurs across the FRP sheet width to failure.

4.7.2.12 Specimen C-U-2-10-4

Typical transverse strain profiles on the FRP sheet for Specimen C-U-2-10-4 at column 9 are presented in Figure 4.56. Column 9 was located 2.5-inches (6.4 cm) from
the loaded end in front of the leading FRP anchor. Failure of Specimen C-U-2-10-4 occurred at 29.00-kips (129.0 kN).

A large strain increase occurred at approximately 12.03-kips (53.5 kN) believed to occur due to the initiation debonding at the strain gage column. Following debonding, a constant increase in strain occurs across the FRP sheet width to failure.

The strain profiles illustrate that the maximum strain occurred in strain gage C9, which was located on the South side of the FRP sheet. As seen in Figure 4.34 failure of Specimen C-U-2-10-4 occurred with minor debonding of a thin strip of FRP on the South side of the FRP sheet. Due to the increased strain values recorded by strain gage C9, it is believed that localized FRP rupture initiated in this region, followed by force redistribution to the remaining FRP sheet inducing global FRP rupture.
4.7.3 FRP Strain along FRP Laminate

In this section strain profiles along the FRP composite sheet length are presented. Strain profiles along the FRP sheet length are provided to provide insight into the effects of FRP anchor longitudinal placement and effects of bonded FRP sheet length behind the anchors on strain development and failure modes. A strain gage column and row is described according to Figure 4.35.

4.7.3.1 Specimen A-0-0-10-0

The strain profile of Specimen A-0-0-10-0 along the FRP laminate length is presented in Figure 4.57, which was recorded by strain gage row B at 5-inch (12.7 cm) intervals. Three distinct regions are visible in the strain profile designated as A, B, and C. Region A occurred during initial loading and corresponds to exponentially decreasing values of axial strain towards the unloaded end between strain gages B5 and B6. The distance between the gages B5 and B6 is 5-inches (12.7 cm), which is termed the initial transfer length by Bizindavyi and Neale (1999). The maximum load corresponding to this strain distribution is the load to initiate a crack front that propagates toward the back end of the FRP sheet. Region B occurred with an additional increase in load moving the point of stress transfer 5-inches (12.7 cm) towards the unloaded end of the FRP to strain gage B5. The final region C illustrates a relatively uniform strain distribution over the debonded length between gages B6 and B3, a distance of 15-inches (38.1 cm), as debonding propagates towards the unloaded end of the FRP. All of the strain transfer in region C occurs between gages B1 and B3 over a distance of approximately 10-inches (25.4 cm) to 15-inches (38.1 cm).
4.7.3.2 Specimen A-0-0-5-0

The strain profile of Specimen A-0-0-5-0 along the FRP laminate length is presented in Figure 4.58, which was recorded by strain gage row B at 5-inch (12.7 cm) intervals. Similar to Specimen A-0-0-10-0 three distinct regions can be seen in the strain profile designated as A, B, and C. The distance between gages B4 and B5 is 5-inches (12.7 cm), which is designated as the initial transfer length for this specimen. Region B occurred with an additional increase in load moving the point of stress transfer 5-inches (12.7 cm) towards the unloaded end of the FRP to strain gage B4. The final region C occurs during the propagation of debonding and illustrates a constant 5-inch (12.7 cm) strain transfer length between gages B3 and B2.
4.7.3.3 Specimen B-Z-2-5-2

Figure 4.59 presents the strain profile for Specimen B-Z-2-5-2, recorded along the sheet centerline by strain gages in row B at 5-inch (12.7 cm) intervals. Similar to Specimens A-0-0-5-0 and A-0-0-10-0 three distinct regions labeled A, B, and C can be recognized in the strain profile. The initial transfer length occurred between gages B4 and B5 over a distance of 3.5-inches (8.9 cm), which is approximately the same length as the control specimens. Region B is observed to be approximately 10-inches (25.4 cm) in length between gages B5 and B3.

In region C during the propagation of debonding the strain profile appears to be nearly linear with a peak strain occurring at the leading FRP anchor. As described in section 4.6.2.1 the failure of Specimen B-Z-2-5-2 occurred due to FRP debonding and anchor rupture, which is consistent with the strain profile since increased values of
strain associated with debonding can be observed along the entire FRP sheet length. A maximum strain of 10.6 mε occurred in strain gage B4, located in front of the leading FRP anchor, during the final phase of debonding.

The strain profile indicates that the leading FRP anchor developed larger maximum strains compared the control specimen before failure. However, the trailing FRP anchor had no influence on the strain profile and did not contribute to fastening the FRP sheet. The FRP anchors were effective in developing a larger maximum load and strain in the FRP sheet before specimen failure and did not prevent the propagation of debonding that eventually lead to specimen failure.

4.7.3.4 Specimen B-Z-2-5-4

In order to develop a closer strain gage array along the FRP sheet centerline to capture the longitudinal distribution of strains in better detail than previous specimens it
was decided to place strain gages in row B at 2.5-inch (6.4 cm) intervals for Specimen B-Z-2-5-4 and all subsequent specimens. Unlike previous specimens the initial transfer length for this specimen occurred between gages B11 and B9 over a distance of 3.6-inches (9.1 cm). However, it should be noted that the shorter transfer length of this specimen when compared to the controls specimens might have been a function of the closer spaced longitudinal strain gages.

Consistent with the observed failure mode of Specimen B-Z-2-5-4 described in section 4.6.2.2, region C indicates the transfer of force at the rear anchor closest to the unloaded end of the specimen. Unlike the control specimens, however, the point of maximum strain occurred at strain gage B10 in front of the leading FRP anchor. It was observed during failure of the specimen that debonding propagated to within approximately 9-inches (22.9 cm) of the sheet end before failure, similar to the recorded strain profile with strains observed up to approximately 10-inches (25.4 cm) of the sheet end to strain gage B3.

The peak values of strain recorded in gages B10 and B6 in region C are believed to have occurred due to the FRP anchors. The reason for higher recorded strain values in gage B10 compared with those in gage B9, however, could not be determined. It is also undetermined why strain gage B12 recorded higher values of strain compared to gage B11, as it is expected that these gages would have identical values once the debonding crack front propagated past the gages towards the unloaded end of the FRP sheet. A maximum strain of 11.4 mε occurred in strain gage B10, which was located in front of the leading FRP anchor where FRP rupture occurred during specimen failure. The FRP anchors were effective in developing a larger maximum load and strain.
compared with the control specimen before failure, but unlike the control specimens the FRP anchors prevented debonding to the FRP sheet end.

![Figure 4.60 — Specimen B-Z-2-5-4 Strain vs. Length](image)

4.7.3.5 Specimen B-Z-4-5-4

Figure 4.61 presents the strain profile for Specimen B-Z-4-5-4, recorded along the sheet centerline by strain gages in row B at 2.5-inch (6.4 cm) intervals. Similar to all other specimens three distinct regions labeled A, B, and C can be recognized in the strain profile. Similar to Specimen B-Z-2-5-4 the initial transfer length for the specimen occurred between gages B9 and B8 over a distance of 2.5-inches (6.4 cm).

The final region C occurred at higher load levels during debonding propagation towards the unloaded end of the FRP. As described in section 4.6.2.3 the failure of Specimen B-Z-4-5-4 occurred due to FRP debonding and anchor rupture, which is consistent with the strain profile since strain values greater than 4 mε, which can be
associated with debonding, are observed throughout region C. A maximum strain of $9.79 \, \mu \epsilon$ occurred in strain gage B8, located in front of the leading FRP anchor, during the final phase of debonding.

The strain profile indicates that the leading FRP anchor developed a larger maximum strain and ultimate load compared with the control specimen before failure. In region C strain gage B6 recorded a peak strain in front of the trailing FRP anchor indicating that the FRP anchor was contributing to force resistance. The FRP anchors were effective in developing a larger maximum load and strain in the FRP laminate before specimen failure. The approximately constant strain region in front of gage B6 indicates that the leading FRP anchor did not prevent the propagation of debonding past the anchor location and eventually lead to specimen failure.

![Figure 4.61 — Specimen B-Z-4-5-4 Strain vs. Length](image)

**Figure 4.61 — Specimen B-Z-4-5-4 Strain vs. Length**
4.7.3.6 Specimen B-W-2-5-4

The strain profile of Specimen B-W-2-5-4 presented in Figure 4.62, which setup and strain gage locations were presented in section 3.5, is unique compared to any other specimens. Two distinct regions can be seen in the strain profile designated as A and B. As increased load was applied strain increased in the FRP sheet with peak values occurring at strain gages B9 and B6 in front of the FRP anchors with strain readings also registered in gages B3 and B4 indicating that the applied force was being resisted by both FRP anchors and the tail bonded FRP laminate.

At a load of 9.28-kips (41.3 kN) the leading FRP anchor failed causing strain gage B9 to malfunction and record compression readings. Following the failure of the leading FRP anchor and associated load drop, increased load was applied illustrated in the strain profile by the B region curves. Region B demonstrates that strain increased in the FRP sheet causing debonding of the tail end of the FRP sheet. The failure of Specimen B-W-2-5-4 occurred due to FRP debonding of side regions on the bonded FRP and FRP rupture in front of the leading FRP anchor, which is consistent with the strain profiles since the maximum strain was recorded in strain gage B9 in front of the leading FRP anchor.

The similarities between Figure 4.62 and the longitudinal strain profile of Specimen B-Z-2-5-4 presented in Figure 4.60 should also be noted particularly in region C strain gages B6, B7, and B8. The shape of the strain profile curves recorded by the gages is similar, however Specimen B-W-2-5-4 recorded reduced strain values since this region of the FRP was unbonded.
4.7.3.7 Specimen B-Z-4-5-6

The behavior of Specimen B-Z-4-5-6 is similar to previous specimens exhibiting an initial transfer length and a shift in stress transfer as increased load was applied. Similar to Specimen B-Z-4-5-4 the initial transfer length for the specimen occurred between gages B9 and B8 over a distance of 2.5-inches (6.4 cm).

The failure mode of Specimen B-Z-4-5-6, which was described in section 4.6.2.5, was due to FRP rupture and debonding. It was observed that debonding propagated to within approximately 9-inches (22.9 cm) of the FRP sheet end before specimen failure, which is consistent with the strain profile, since strain was recorded to within approximately 10-inches (25.4 cm) of the FRP laminate end to strain gage B3.

The reason for negative strain values in gage B6 could not be determined, but is believed to have occurred due to a gage malfunction. A maximum strain of 10.3 mε.
was recorded in gage B9, located in front of the leading FRP anchor where FRP rupture occurred during specimen failure.

4.7.3.8 Specimen B-Y-2-5-4

The behavior of Specimen B-Y-2-5-4 is similar to previous specimens exhibiting an initial transfer length and a shift in stress transfer as increased load was applied. Similar to Specimens B-Z-2-5-4, B-Z-4-5-4, and B-Z-4-5-6 the initial transfer length for the specimen occurred between gages B9 and B7 over a distance of 3-inches (7.6 cm).

A maximum strain of 5.54 mε was recorded in gage B7, located in front of the leading FRP anchor where FRP rupture occurred during specimen failure. During the failure of Specimen B-Y-2-5-4 it was observed that debonding propagated approximately 9-inches (22.9 cm) from the loaded end of the FRP laminate, similar to...
the recorded strain profile with strains observed up to approximately 12.5-inches (31.8 cm) from the loaded specimen end to strain gage B4. The peak values of strain recorded in gages B7 in region C is believed to have occurred due to the FRP anchors.

Unlike the previous specimen strain profiles that had FRP anchor patterns with two anchors in the longitudinal direction, region C in Figure 4.64 exhibits a rapid drop in strain across the FRP anchors. The rapid drop in strains indicates that the FRP anchors were effective in transferring shear into the concrete substrate and arresting debonding propagation past strain gage B4 to within 17.5-inches (44.5 cm) of the FRP sheet end. The tailing end of the FRP sheet, past strain gage B4, although bonded to the concrete surface, did not contribute to force resistance.

The reason for a non-uniform strain distribution between strain gages B7 and B10 could not be determined, as it is theoretically expected that these gages should have identical values once the debonding crack front propagated past the gages toward the unloaded end of the FRP sheet. It can be concluded that the FRP anchors were effective in developing a larger maximum load and strain compared with the control specimen before failure. The FRP anchors prevented debonding of the FRP sheet from the concrete surface in this specimen as well.
4.7.3.9 Specimen B-X-2-5-4

The behavior of Specimen B-X-2-5-4 is similar to previous specimens exhibiting an initial transfer length and a shift in stress transfer as increased load was applied. Similar to other specimens with FRP anchors the initial transfer length for the specimen occurred between gages B9 and B7 over a distance of 3.5-inches (8.9 cm). A shorter length of bonded sheet was used in this specimen to determine if the length of the bonded sheet behind the FRP anchors contributed to the force resistance. In Specimen B-Y-2-5-4 longitudinal strain profiles it can be seen that the last 17.5-inches (44.5 cm) did not contribute to the force resistance since no strain was recorded.

A maximum strain of 7.54 \( \varepsilon \) was recorded in gage B4, located in front of the leading FRP anchor. As described in section 4.6.3.2 the failure of Specimen B-X-2-5-4 occurred due to FRP debonding and delamination, which is consistent with the strain
profile, since increased values of 4 mε that can be associated with debonding can be observed throughout region C.

It should be noted that the FRP anchor pattern, anchor diameter, and splay diameter of Specimen B-X-2-5-4 were identical to Specimen B-Y-2-5-4. The difference between the two specimens was the bonded length of FRP sheet behind the anchors; Specimen B-Y-2-5-4 had a bonded length of 30-inches (76.2 cm) and Specimen B-X-2-5-4 had a bonded length of 15-inches (38.1 cm). Similar to Specimen B-Y-2-5-4, Specimen B-X-2-5-4 exhibits a rapid strain drop in region C behind the FRP anchors. However, unlike Specimen B-Y-2-5-4, which recorded strain up to gage B4 17.5-inches (44.5 cm) from the unloaded end of the specimen, Specimen B-X-2-5-4 recorded strain in all gages along the entire sheet length. The reason why Specimen B-Y-2-5-4 recorded no strain past gage B4, which has the same gage location as B1 of Specimen B-X-2-5-4 that recorded substantial strain to the FRP sheet end, could not be determined.

It can be concluded that the FRP anchors were effective in developing a larger maximum load and strain compared with the control specimen before failure. However, region C indicates strain values associated with debonding indicating that the FRP anchors did not prevent the propagation of debonding to the FRP sheet end eventually leading to specimen failure.
4.7.3.10 Specimen C-X-4-10-6

The behavior of Specimen C-X-4-10-6 is similar to previous specimens exhibiting an initial transfer length and a shift in stress transfer as increased load was applied. Similar to other specimens with FRP anchors the initial transfer length for the specimen occurred between gages B5 and B6 over a distance of 2.5-inches (6.4 cm).

As described in section 4.6.3.3 the failure of Specimen C-X-4-10-6 occurred due to FRP rupture, debonding, anchor pullout, and delamination, which are consistent with the strain profile, since strain values greater than 4 m\text{e}, which can be associated with debonding, can be observed throughout region C. A maximum strain of 6.16 m\text{e} occurred in strain gage B5 located in front of the leading FRP anchor during the final phase of debonding.
In comparison to Specimen B-X-2-5-4, which had the identical FRP anchor pattern and bond length as Specimen C-X-4-10-6, region C in Figure 4.66 is much more uniform in front of the FRP anchors. Strain gage B5 in Specimen C-X-4-10-6 reached a maximum strain of 6.16 m\(\epsilon\), compared to a maximum strain of 7.54 m\(\epsilon\) in strain gage B4 of Specimen B-X-2-5-4, which had identical locations in the center of the FRP sheet. The difference in the strain profiles is believed to occur due to the placement of the strain gages. In Specimen B-X-2-5-4 strain gage B4 was placed in line with the edge the FRP anchor splay, compared to Specimen C-X-4-10-6 strain gage B5 that was located in the \(\frac{1}{2}\)-inch (1.3 cm) gap between the FRP anchors. This behavior exhibits the very localized effects of FRP anchors on the FRP sheet directly in front of the anchor splay region.

The FRP anchors were effective in developing a larger maximum load and strain in the FRP laminate before failure. However, region C indicates a relatively uniform strain distribution along the FRP sheet indicating that the FRP anchors did not prevent the propagation of debonding to the FRP sheet end eventually leading to specimen failure.
4.7.3.11 Specimen C-Y-4-10-6

The behavior of Specimen C-Y-4-10-6 is similar to previous specimens exhibiting an initial transfer length and a shift in stress transfer as increased load was applied. Similar to other specimens with FRP anchors the initial transfer length for the specimen occurred between gages B9 and B7 over a distance of 2.5-inches (6.4 cm). A longer length of bonded sheet was used in this specimen to determine if failure by FRP rupture could be obtained by allowing for a longer debonding length since the failure of Specimen C-X-4-10-6, which had a 15-inch (38.1 cm) bond length, occurred by FRP debonding.

At the failure load of 19.69-kips (87.6 kN) axial strain throughout the entire FRP sheet was greater than 4 mε indicating that the entire FRP sheet had debonded, which is consistent with the observed failure modes of debonding, FRP rupture, and
delamination. A maximum strain of $7.20 \, \varepsilon$ occurred in strain gage B9, located in front of the leading FRP anchor, during the final phase of debonding. It is undetermined why strain gage B11 recorded reduced values of strain compared to gages B10 and B9, as it is expected that these gages would have identical values once the debonding crack front propagated past the gages towards the unloaded end of the FRP sheet.

The reason for a peak recorded strain value in gage B9, which was located in the $\frac{1}{2}$-inch (1.3 cm) gap between the FRP anchors, could not be determined. Region B in Figure 4.67 exhibits a rapid strain drop behind the FRP anchors similar to Specimens B-Y-2-5-4 and B-X-2-5-4; however, as increased load was applied region C illustrates debonding to failure. The observed failure mode and longitudinal strain profile of Specimen C-Y-4-10-6 compared to Specimen B-Y-2-5-4, which had the same anchor pattern and bond length, indicate that smaller FRP anchors are less prevalent to premature failure modes and do a better job at preventing FRP debonding.

The FRP anchors were effective in developing a larger maximum load and strain in the FRP sheet before failure. However, the anchors did not prevent the propagation of debonding to the FRP sheet end eventually leading to specimen failure.
4.7.3.12 Specimen C-U-2-10-4

The behavior of Specimen C-U-2-10-4 is similar to previous specimens exhibiting an initial transfer length and a shift in stress transfer as increased load was applied. Similar to previous specimens the initial transfer length for the specimen occurred between gages B9 and B10 over a distance of 2.5-inches (6.4 cm).

A maximum strain of 10.03 m\(\varepsilon\) was recorded in gage B10, located in front of the leading FRP anchor where FRP rupture occurred during specimen failure. During the failure of Specimen C-U-2-10-4 it was observed that debonding propagated approximately 11-inches (27.9 cm) from the loaded end of the FRP laminate, similar to the recorded strain profile with strains observed up to approximately 15-inches (38.1 cm) from the loaded specimen end to strain gage B5.
The reason for higher recorded strain values in gage B10 compared with those in gage B9, could not be determined, since it is expected that these gages would have identical values once the debonding crack front propagated past the gages towards the unloaded end of the FRP sheet. It is believed that the strain drop occurred from localized FRP anchor, bond or load effects. It is believed that the FRP anchors were effective in shear force resistance and arresting debonding propagation past strain gage B5 to within 15-inches (38.1 cm) of the FRP sheet end.

Comparing the failure modes and longitudinal strain profiles between Specimens C-Y-4-10-6 and C-U-2-10-4, which had similar anchor patterns and identical bond length and width, illustrates that smaller FRP anchors are less prevalent to premature failure modes and do a better job at preventing FRP debonding.

It can be concluded that the FRP anchors were effective in developing a larger maximum load and strain compared to the control specimen before failure. The FRP anchors prevented debonding of the FRP sheet from the concrete surface in this specimen as well.
4.7.4 Strain Summary

Table 4.8 illustrates the maximum-recorded FRP axial strain for each specimen, the strain gage location of the maximum-recorded strain, and the percentage of maximum-recorded strain of the ultimate FRP strain defined as $14.9 \, \varepsilon$ as calculated in section 4.3 from FRP coupon testing.

The first major observation is the location of the strain gage with maximum-recorded strain. It is expected that axial strain would be the greatest in the center of the FRP laminate as described in section 2.3.1.1 for specimens with no FRP anchors. However, Table 4.8 illustrates that the location of the maximum strain occurred in row C for both control specimens. Specimens with FRP anchors centered about the sheet centerline, specifically specimens with anchor pattern Z, it is expected that the maximum strain would occur in the center of the FRP laminate directly in line
with the FRP anchor, which Table 4.8 illustrates is true. For specimens with FRP anchors that are not centered about the sheet centerline, particularly specimens with anchor patterns X, Y, or U, it is expected that the maximum-recorded strain would occur inline with the center of the FRP anchors and lower strains would occur in the centerline of the FRP sheet at the edge of an FRP splay region. As expected Table 4.8 illustrates that the location of maximum strain for specimens with non-centered FRP anchors occurred in either gage row A or C, which were located closer to the center of the FRP anchor diameter than gage row B that was located in the center of the FRP sheet outside of the anchor splay region.

The second observation is the variability in the recorded percentage of \( \frac{\varepsilon_{\text{max}}}{\varepsilon_{\text{ult}}} \) in Table 4.8 compared to \( \frac{P_{\text{test}}}{P_{\text{ult}}} \) in Table 4.5, which may have occurred due to a number of reasons. The first possibility is non-uniform load application, which would then introduce torque into the test inducing increased strain to one side of the FRP sheet. Strain variability could also be generated from variation in local material properties of the FRP laminate and concrete substrate, or from localized FRP anchor effects. Variability in behavior introduced through the properties of the concrete substrate is believed to be the major contributor strain variability during the propagation of debonding. The concrete surface strength is highly variable due to irregular aggregate and paste distribution and also because of irregularities introduced during surface preparation of the specimens.
The third observation to be noted is the inability to record FRP rupture strain, even in cases where FRP sheet rupture was observed. This phenomenon is believed to occur due to localized FRP material properties or the inability to record peak strain values using a discrete number of strain gages. As described in section 4.6 FRP rupture was observed for numerous specimens leading to the conclusion that the FRP rupture strain was reached, but not recorded. As described in the observed failure modes of the specimens it is believed that FRP rupture occurred in localized locations causing the redistribution of forces inducing global FRP rupture or debonding.

It should also be noted that in Specimen B-Z-2-5-2, B-Z-2-5-4, B-Z-4-5-4, B-W-2-5-4, B-Z-4-5-6, B-Y-2-5-4, and B-X-2-5-4 a strain peak can be observed in front of the leading FRP anchor once the debonding crack front had propagated past the leading anchor location. The strain peak is believed to have occurred due to a stress concentration at the FRP anchor. Once the debonding crack front had propagated past the anchors a flow of forces (Figure 4.69) into the FRP anchor may have caused a stress concentration at the anchor leading to a strain peak.

Figure 4.69 — Force Flow into FRP Anchor
4.8 Summary of Trends Observed in the Specimen Response

Several fundamental characteristics are evident from the observed failure modes and strain measurement results obtained in the investigation of fastening FRP laminates with FRP anchors. FRP anchors are effective in fastening FRP laminates to reinforced concrete elements allowing for the development of the full rupture strength of the composite sheet. FRP anchors provide an additional strength to the FRP composite allowing for the development of the ultimate strength of the FRP sheet.
The overall effectiveness of FRP anchors is dependent upon several factors, most noticeably the ratio of the FRP anchor splay diameter to FRP anchor diameter. This ratio governs the ability of the FRP anchor to engage a certain width of FRP composite sheet. Larger splay diameters were observed to engage wider regions of the FRP sheets. Therefore, providing a large anchor splay diameter with a small anchor diameter may cause failure in the anchor because the force being transferred by the sheet into the anchor exceeds the anchor capacity. Conversely, providing a small splay diameter with a large anchor diameter may not fail the anchor in shear because of the smaller force that the FRP anchor needs to transfer into the substrate. Small splay diameters, however, do not engage the entire FRP sheet width and lead to combined failure modes (localized sheet rupture and debonding of FRP sheet regions not engaged by the FRP anchor).

Also very important to the overall effectiveness of FRP anchors is the selected anchor spacing. Spacing anchors longitudinally along the composite sheets is not necessary to develop higher sheet forces. The ability to achieve FRP rupture can be directly attributed to the effectiveness of the leading FRP anchor closest to the applied load. Anchors placed across the width of the composite sheet are more efficient in developing higher forces in the anchored FRP sheet. Spacing the anchors such that their splays nearly overlap is very important as each anchor can engage a specific width of FRP sheet equal to the splay diameter of the FRP anchor. Spacing the anchors with a gap in-between anchor splays did not engage the composite sheet between the anchors resulting in composite sheet failure before development of the ultimate strength of the FRP anchors.
The length of FRP sheet bonded directly to the concrete surface behind the anchors also affects the effectiveness of the FRP anchor system. Providing a longer bond length of composite sheet develops a more ductile debonding process and increases the total force at specimen failure. The effectiveness of FRP anchors is dependent upon their ability to prevent FRP laminate debonding throughout the entire FRP sheet length. Table 4.9 illustrates those specimens in which the predominant failure mode was due to FRP rupture (highlighted in italics). It should be noted that the same specimens that failed mainly due to FRP rupture (B-Z-2-5-4, B-Z-4-5-6, B-Y-2-5-4, B-X-2-5-4, C-U-2-10-4), only partially debonded along the FRP sheet length. The debonded length in Table 4.9, described as the final unbonded length, was measured from the longitudinal strain profiles of the specimens described in section 4.7.3. Only the anchored specimens that were able to prevent complete FRP sheet debonding were able obtain approximately 70% or more of the ultimate strength of the FRP, with the exception of Specimen B-X-2-5-4 that failed due to FRP debonding and reached 77.3% of the FRP ultimate strength. The ability of FRP anchors to prevent debonding is affected by several parameters that caused premature failure modes associated to the anchorage system including anchor delamination, anchor pullout, and splitting failure between the FRP anchor and laminate sheet.
Table 4.9 — Specimen Debonded Length

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Debonded Length [in]</th>
<th>Bond Length [in]</th>
<th>$P_{\text{test}}/P_{\text{ult},s}$ [%]</th>
<th>Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-0-0-10-0</td>
<td>30</td>
<td>30</td>
<td>32.6%</td>
<td>Debonding</td>
</tr>
<tr>
<td>A-0-0-5-0</td>
<td>30</td>
<td>30</td>
<td>45.5%</td>
<td>Debonding</td>
</tr>
<tr>
<td>B-Z-2-5-2</td>
<td>30</td>
<td>30</td>
<td>57.8%</td>
<td>Anchor Shear, Debonding</td>
</tr>
<tr>
<td>B-Z-2-5-4</td>
<td>22.5</td>
<td>30</td>
<td>67.7%</td>
<td>FRP Rupture, Debonding</td>
</tr>
<tr>
<td>B-Z-4-5-4</td>
<td>30</td>
<td>30</td>
<td>62.6%</td>
<td>Anchor Shear, Debonding</td>
</tr>
<tr>
<td>B-Z-4-5-6</td>
<td>20</td>
<td>30</td>
<td>74.3%</td>
<td>FRP Rupture, Debonding</td>
</tr>
<tr>
<td>B-Y-2-5-4</td>
<td>12.5</td>
<td>30</td>
<td>70.6%</td>
<td>FRP Rupture, Debonding</td>
</tr>
<tr>
<td>B-X-2-5-4</td>
<td>15</td>
<td>15</td>
<td>77.3%</td>
<td>FRP Rupture, Delamination,</td>
</tr>
<tr>
<td>C-X-4-10-6</td>
<td>15</td>
<td>15</td>
<td>56.1%</td>
<td>FRP Rupture, Debonding,</td>
</tr>
<tr>
<td>C-Y-4-10-6</td>
<td>30</td>
<td>30</td>
<td>61.9%</td>
<td>FRP Rupture, Debonding,</td>
</tr>
<tr>
<td>C-U-2-10-4</td>
<td>15</td>
<td>30</td>
<td>82.6%</td>
<td>FRP Rupture</td>
</tr>
</tbody>
</table>

4.9 Summary

This chapter presented material testing results from compression tests performed on concrete cylinders and tensile tests performed on FRP coupons. Also presented were both the observed specimen response and strain measurement results from the experimental testing program described in Chapter 3. The observed failure modes of the specimens were discussed and illustrated using photographs. Strain readings
captured during testing and summaries of the trends observed in the specimen response were presented.
CHAPTER 5
EVALUATION OF TEST RESULTS

5.1 Introduction

In this chapter the parameters of the experimental program are discussed based upon the global response observed during the failure of each specimen. An FRP anchor shear strength is developed that can be used to predict the response of anchored FRP sheets to reinforced concrete subjected to direct shear. Due to the variability observed in strain measurements strains were not used for the purpose of evaluating the efficiency of an anchoring scheme on behavior of the FRP sheets.

5.2 Effect of Anchor Geometry

The effect of the FRP anchor geometry is an important parameter that governs the shear strength of FRP anchors. The two geometrical parameters studied during this experimental program were FRP anchor splay diameter and FRP anchor diameter. It was determined that FRP anchor length was not a governing parameter for the anchor length used in this research project based upon the failure modes of the initial Specimens B-Z-2-5-2 and B-Z-4-5-4. The failure modes of these specimens consisted of FRP anchor shear, which illustrated that FRP anchor pullout was not a dominant failure mode; therefore, it was decided to hold this geometrical parameter constant throughout all tests and vary the parameters that governed the effectiveness of FRP anchors. During the failure of Specimen C-X-4-10-6 FRP anchor pullout was identified as one of the failure modes, however, it is believed that this failure mode occurred because the ¾-inch (1.9 cm) FRP anchors that were used in this specimen were not
completely impregnated during the installation procedure. Discussion of the prevention of premature failure modes is presented in Chapter 7.

5.2.1 Effect of Splay Diameter

An important geometrical parameter in determining the strength and effectiveness of FRP anchors is the diameter of the anchor splay. The splay diameter determines the effective width of FRP sheet engaged by an individual anchor and determines the required diameter of the FRP anchor to transfer the force generated on the FRP sheet into the concrete substrate.

During failure of Specimen B-Z-2-5-4, discussed in section 4.6.2.2, it was observed that an FRP anchor with a splay diameter of 2-inches (5.1 cm) and an anchor diameter of ½-inch (1.3 cm) was effective in rupturing a 2-inch (5.1 cm) width of FRP sheet leading to the concept that the effective width of FRP sheet that an FRP anchor can engage equals the splay diameter. However, during the failure of Specimen B-Z-4-5-4, which had a splay diameter of 4-inches (10.2 cm) and an anchor diameter of ½-inch (1.3 cm), failure of the FRP anchors occurred due to FRP anchor shear. Failure of Specimen B-Z-4-5-4 illustrated that providing a 4-inch (10.2 cm) splay diameter with a ½-inch (1.3 cm) anchor diameter engaged of a width of FRP sheet that was too large for the FRP anchor shear strength. The diameter of the splay establishes the width of engaged FRP sheet, which then determines the required diameter of the FRP anchor so as not to fail the anchor due to shear.
5.2.2 Effect of Anchor Diameter

Equally as important as the diameter of the FRP splay region is the diameter of the FRP anchor. The FRP anchor diameter is directly related to the force being transferred at the anchor from the FRP sheet into the concrete substrate. As discussed earlier, the splay diameter is the main factor affecting the effective width of FRP laminate engaged. The FRP anchor diameter, therefore, has to be determined in accordance with the width of FRP laminate being engaged by each anchor splay.

The failure of FRP anchors in Specimen B-Z-2-5-2 (section 4.6.2.1) was caused by FRP anchor shear. The FRP anchors in this specimen had a splay diameter of 2-inches (5.1 cm) and a diameter of ¼-inch (0.64 cm). This failure mode indicated that the force developed within the 2-inch (5.1 cm) splay diameter was too large for the strength developed by a ¼-inch (0.64 cm)-diameter anchor causing FRP anchor shear failure. However, failure of the subsequent test Specimen B-Z-2-5-4, which had a splay diameter of 2-inches (5.1 cm) and an anchor diameter of ½-inch (1.3 cm), occurred due to FRP rupture indicating that a ½-inch (1.3 cm) anchor diameter was large enough to resist the force developed in the 2-inch (5.1 cm) width of splay region.

The same concept was demonstrated in the testing of Specimens B-Z-4-5-4 and B-Z-4-5-6. Specimen B-Z-4-5-4, with FRP anchors with a splay diameter of 4-inches (10.2 cm) and an anchor diameter of ½-inches (1.3 cm), failed due to FRP anchor shear. The failure of Specimen B-Z-4-5-6, which had a splay diameter of 4-inches (10.2 cm) and an anchor diameter of ¾-inch (1.9 cm) failed due to FRP rupture indicating that for a 4-inch (10.2 cm) splay diameter an anchor diameter of at least ¾-inch (1.9 cm) is required.
5.3 Effect of Anchor Arrangement

The spacing of FRP anchors both longitudinally and transversely within the bonded FRP laminate area is an important factor that affected specimen failure. The failure of Specimen B-Z-2-5-4, had an FPR anchor spacing shown in Figure 3.27, illustrated that the leading FRP anchor closest to the applied load provided the necessary strength to rupture 2-inches (5.2 cm) of FRP sheet directly in front of the anchor. The gain in ultimate capacity of Specimen B-Z-2-5-4 over the control specimen can be directly attributed to the effectiveness of the leading FRP anchor. It is believed that the trailing FRP anchor in Specimen B-Z-2-5-4 provided no additional strength in fastening the FRP sheet to the concrete substrate since the maximum FRP strain was recorded in front of the leading FRP anchor and sheet rupture occurred in the identical location. The failure of Specimen B-Y-2-5-4, with an anchor arrangement as shown in Figure 3.29, further corroborated this observation. Since two FRP anchors with 2-inch (5.2 cm) splays were placed transversely adjacent to one another and failure of the specimen occurred by rupturing 4-inches (10.2 cm) of FRP laminate directly in front of the FRP anchors as described in section 4.6.8.

5.4 Effect of Composite Bond Length

The length of bonded composite laminate behind the FRP anchors was another parameter studied during the experimental program. Providing a longer bond length behind the FRP anchors allowed for a more ductile debonding process and increased the total force at specimen failure. The effectiveness of FRP anchors is dependent upon their ability to prevent FRP laminate debonding throughout the entire FRP sheet length.
The failure of Specimen B-Y-2-5-4, which had a pair FRP anchors spaced transversely across the sheet width and sheet bond length of 30-inches (76.2 cm) (Figure 3.29), was due to FRP rupture in front of the leading FRP anchors. As discussed in section 4.7.3.8 the FRP anchors were effective in preventing debonding to within 17.5-inches (44.5 cm) from the unloaded end of the FRP sheet end. The failure of Specimen B-X-2-5-4, with only half of the bond length of Specimen B-Y-2-5-4, failed from a combination of modes including FRP rupture, delamination, and debonding. The FRP anchors in Specimen B-X-2-5-4 were not effective in preventing debonding to propagate to the sheet end, so failure modes other than FRP rupture occurred causing premature failure of the system. It is evident that bond of the FRP sheet behind the leading anchor allowed higher strains to be developed in the FRP sheet region in front of the anchor section leading to FRP rupture.

5.5 Effect of Composite Width

Composite sheet widths of 5-inches (12.7 cm) and 10-inches (25.4 cm) were studied during the experimental program. As discussed in section 2.3.1.1 increasing the nominal bond width increases the bond strength. In order to anchor the entire bond width to achieve FRP sheet rupture it is necessary to place FRP anchors that have splay diameters that cover the entire bonded width.

Failure of Specimen B-Y-2-5-4, which had an anchor placement according to Figure 3.29, was characterized by rupture of a 4-inch (10.2 cm) width of FRP sheet in front of the FRP anchors and FRP debonding of ½-inch (1.3 cm) side regions adjacent to the FRP anchors. Similar results occurred during the failure of Specimen C-U-2-10-
4, which had twice the number of anchors and bond width compared with Specimen B-Y-2-5-4. Failure in Specimen C-U-2-10-4 occurred by FRP rupture in the 8-inch (20.4 cm) region of FRP sheet in front of the FRP anchors with practically no edge debonding observed on the side regions not anchored by the FRP anchor splay regions. In conclusion the concepts of FRP anchor design work in developing the force necessary to rupture narrow and wide FRP sheets.

5.6 Formulation of FRP Anchor Shear Strength Equation

Based on the failure modes of multiple specimens a behavioral model was developed allowing the design of additional anchor geometries for various FRP sheet-strengthening conditions.

Based on the observed failure modes of Specimens B-Z-2-5-4 and B-W-2-5-4, as discussed in section 4.6.2.2 and 4.6.2.4, respectively, it was assumed that an FRP anchor with a 2-inch (5.1 cm) splay diameter and an anchor diameter of ½-inch (1.3 cm) was strong enough to develop the force required to rupture a 2-inch (5.1 cm) width of FRP laminate. The failure mode of Specimen B-Z-4-5-4, which exhibited anchor shear, demonstrated that an FRP anchor with a 4-inch (10.2 cm) splay diameter and an anchor diameter of ½-inch (1.3 cm) was not strong enough to develop the force required to rupture 4-inches (10.2 cm) of FRP laminate. Using the FRP coupon tests discussed in section 4.3 it was determined that the FRP used in this experimental program had an ultimate tensile strength of 3.51 kips/inch. Dividing the force necessary to rupture a 2-inch (5.1 cm) width of FRP sheet by the nominal area of a ½-inch (1.3 cm) FRP anchor yielded an FRP anchor average shear strength of 35.8 k/in² (246.8 MPa).
To determine the necessary anchor diameter for a 4-inch (10.2 cm) splay diameter, as used in Specimens B-Z-4-5-6, C-X-4-10-6, and C-Y-4-10-6, the average shear strength of the anchor of 35.8 k/in² (246.8 MPa) was used. The strength of the FRP sheet engaged by a 4-inch (10.2 cm) splay diameter was 14.04-kips (62.5 kN) based upon an ultimate tensile strength of 3.51 kips/inch as discussed above. Dividing the required FRP rupture force by the estimated average anchor shear strength and solving for the anchor diameter yielded an FRP anchor diameter of 0.707-inches (1.8 cm); therefore, it was decided to use an anchor diameter of ¾-inch (1.91 cm) for all tests involving a 4-inch (10.2 cm) anchor splay diameter.

The proposed anchor diameter formulation, which is based on an experimentally derived anchor shear strength previously discussed, has the following parameters (see Figure 5.1):

\[
D_A = \sqrt[4]{\frac{4(S_a)(f_{fu})(t_p)(n_p)}{\pi(35.8)}} \tag{5.1}
\]

where:

\(D_A\) = FRP anchor diameter [in]

\(S_a\) = anchor splay diameter [in]

\(f_{fu}\) = FRP ultimate tensile strength [ksi]

\(t_p\) = nominal thickness of FRP sheet [in]

\(n_p\) = number of FRP plies
To incorporate the shear strength equation it is necessary to select an anchor splay diameter, which will determine the effective width of FRP sheet engaged by an individual anchor. Once the force that each anchor needs to transfer into the substrate is determined, the required FRP anchor diameter is then calculated using equation 5.1. It should be noted however that equation 5.1 is only valid for FRP anchors ranging from \( \frac{1}{4} \)-inch (0.64 cm) to \( \frac{3}{4} \)-inch (1.91 cm) in diameter, as these were the FRP anchor diameters used in this research program. A discussion of the prevention of premature failure modes is presented in Chapter 7.
CHAPTER 6
FINITE ELEMENT ANALYSIS

6.1 Introduction

This chapter presents the formulation and results of a 2-D plane stress finite element model developed using ADINA 8.3.3 to study the bond behavior between FRP sheets attached to the surface of reinforced concrete elements. In particular, two dimensional finite element models of Specimens A-0-0-10-0, A-0-0-5-0, and B-Y-2-5-4, are used to analyze the interfacial debonding behavior between FRP sheets bonded to the concrete surface. To validate the finite element models, comparisons are made between the finite element model and the test specimens regarding FRP strain profiles and ultimate load carrying capacity.

Nonlinear shear spring elements are utilized to properly model the behavior between the FRP sheets and concrete. The constitutive relations of these nonlinear shear-spring elements represented the interfacial behavior between the FRP sheet and concrete surface. The stress-strain relations of the nonlinear shear spring elements is calculated using the bond-slip model published by Lu et al. (2005), which accounts for the width of the concrete block, the width of the bonded FRP sheet, stiffness of the adhesive, and constitutive properties of the concrete. These interface elements, therefore, represent the properties of the interface between the FRP sheets and the concrete surface including the epoxy adhesive and surface characteristics of the concrete substrate.
The use of a 2-D finite element model to represent the behavior of the specimens tested in this research has several limitations. In performing a 2-D plane stress finite element analysis of the specimens the main assumption was that out-of-plane stresses were equal to zero. The ability to capture the full behavior of the debonding crack front as it propagated towards the unloaded end of the specimen was not captured due to the inability of a 2-D model to capture the behavior of the FRP edge regions at the FRP-concrete interface. These tests as well as results from previous investigations have shown that out of plane stresses can be important. As discussed in section 2.3.1.1 Subramaniam et al. (2007) found that during the propagation of debonding edge regions are subjected to high axial and shear strain gradients. Transverse strain profiles determined during testing of control Specimens A-0-0-5-0 and A-0-0-10-0 (section 4.7.2) showed reduced strain readings in strain gages near the edge of the FRP sheet during the propagation of debonding. This behavior indicated that the FRP central region is principally accountable for shear stress transfer between the FRP and reinforced concrete substrate. A 2-D plane stress finite element model is incapable of capturing the edge region behavior at the FRP-concrete interface since the transverse distribution of strains is not calculated. The 2-D models, however, do capture longitudinal behavior of the specimens and are computationally very efficient. For these reasons, 2-D finite element models were selected for this research and were only used to compare with the behavior of the finite element concrete system in the longitudinal direction.
6.2 Material Modeling

This section presents the constitutive definitions that were used to model the concrete, FRP sheet, and FRP-concrete interface elements.

6.2.1 Concrete Material Model

The constitutive formulations used to model the concrete elements were provided in the ADINA software. The material model had five basic features (ADINA R & D, Inc., 2005):

1. A nonlinear stress-strain relation to allow for the weakening of the material under increasing compressive stresses.
2. A failure envelope that defines failure in tension at a maximum relatively small principal stress.
3. A failure envelope that defines crushing in compression at high compression stresses.
4. A strategy to model the post-cracking and crushing behavior of the material.
5. Strain softening from compression crushing failure to an ultimate strain, at which the material totally fails.

The compressive and tensile material failure envelopes were employed to establish the uniaxial stress-strain law accounting for multiaxial stress conditions, and to identify whether tensile or crushing failure of the material occurred (ADINA R & D, Inc., 2005). The post failure material behaviors considered in the model include the post tensile cracking, post compression crushing, and strain-softening behaviors.
Tension stiffening was modeled as a linearly descending line after the post-peak point at which the concrete had cracked as seen in Figure 6.2. The biaxial concrete compressive failure envelope can be seen in Figure 6.1, where $'\sigma_{pi}$ is defined as the principal stress in direction $i$ and $\sigma_c$ is defined as the maximum uniaxial compressive stress. Compressive stresses and strains are taken as negative in Figure 6.1.

The parameters used to implement the concrete material model as illustrated in Figure 6.2 were defined as follows:

- $\sigma = $ uniaxial stress
- $e = $ uniaxial strain
- $'\sigma_t = $ uniaxial cut-off tensile strength
- $'\sigma_{yp} = $ post-cracking uniaxial cut-off tensile strength. Note that $'\sigma_{yp}=0$, therefore, ADINA set $'\sigma_{yp}='\sigma_t$. 

![Figure 6.1 — Biaxial Failure Envelope (ADINA R & D, Inc., 2005)]


$e_t$ = uniaxial cut-off tensile strain. Note that this was not accounted for since $\sigma_{tp} = 0$

$e_u$ = ultimate uniaxial compressive strain

$e_c$ = uniaxial strain corresponding to $\sigma_c$

$\sigma_u$ = ultimate uniaxial compressive stress

$\sigma_c$ = maximum uniaxial compressive stress

Figure 6.2 — Uniaxial Stress-Strain Relations (ADINA R & D, Inc., 2005)

In order to calculate the compressive input parameters for each finite element model from the concrete cylinder testing performed for each specimen, the Modified Hognestad stress-strain curve was utilized according to Figure 6.3. In this model, the stress-strain curve consists of two distinct regions: a region in which stress increases parabolically as a function of strain, followed by a region where the stresses decrease
The parabolic region extends from the point of zero stress to the point of maximum concrete stress, $f''_c$. The maximum concrete stress, $f''_c$, was calculated as 90% of the nominal concrete compressive stress, $f'_c$. The strain corresponding to the maximum concrete stress is defined as $\varepsilon_{p}$, which was calculated as a function of the maximum concrete stress and modulus of elasticity. The limiting concrete strain at the termination point of the linear descending branch of the stress-strain curve is called the maximum usable strain, $\varepsilon_{cu}$. The maximum limiting strain was set to 0.0038 as suggested by MacGregor (1997). In accordance with ACI 318-02, the modulus of elasticity, $E_c$, was calculated as $$E_c = 33w^{1.5} \sqrt{f'_c},$$ where $w$ is the unit weight of concrete $\left[ \frac{lb}{ft^3} \right]$ and $f'_c$ is the nominal concrete compressive strength (psi) (ACI Committee 318, 2002). The uniaxial tensile strength of concrete, $f'_t$, was calculated as $4\sqrt{f'_c}$. The slope of the tension portion of the stress-strain curve was set to the tangent modulus at the origin of the stress-strain plot.

![Modified Hognestad Stress-Strain Curve](image)

**Figure 6.3 — Modified Hognestad Stress-Strain Curve (MacGregor, 1997)**
6.2.2 FRP Constitutive Model

A linear elastic tensile constitutive model was used to model the FRP. A rupture point was defined as the ultimate tensile strain as calculated from the FRP coupon testing, which was discussed in section 4.3.

6.2.3 FRP-Concrete Interface Constitutive Model

In order to model the interfacial behavior between the concrete surface and FRP sheet a bilinear bond stress-slip model was incorporated into the finite element analysis as proposed by Lu et al. (2005). A typical shear stress-slip curve can be seen in Figure 6.4, which illustrates the definitions of each of the parameters that are incorporated into the model.

![Figure 6.4 — Typical Bond Stress-Slip Curve](image-url)
It is important to note that the bond-stress slip model defines the entire interfacial behavior of the FRP-concrete interface. This bond stress-slip model was based on a meso-scale finite element analysis and has the option of modifying the model based on the stiffness of the adhesive used for specific test specimens.

The initial ascending linear portion of the model was calculated as follows:

\[
\text{if } s \leq s_o, \text{ then } \tau = \tau_{\text{max}} \frac{s}{s_o} \quad [6.1]
\]

where:

\[
s = \text{local slip [mm]}
\]
\[
s_o = \text{local slip at } \tau_{\text{max}} \text{ [mm]}
\]
\[
\tau = \text{local bond stress [MPa]}
\]
\[
\tau_{\text{max}} = \text{maximum local bond stress [MPa]}
\]

\[
s_o = 0.0195 \beta_o f'_t + s_e \quad [6.2]
\]

\[
\tau_{\text{max}} = 1.5 \beta_w f'_t \quad [6.3]
\]

\[
\beta_o = \text{width ratio factor}
\]
\[
f'_t = \text{concrete tensile strength [MPa]}
\]
\[
s_e = \text{elastic component of local slip [mm]}
\]

where:

\[
f'_t = 0.3325 \sqrt{f'_c} \quad [6.4]
\]
\[ \beta_{\omega} = \frac{2.25 - \frac{b_f}{b_c}}{\sqrt{1.25 + \frac{b_f}{b_c}}} \]  

[6.5]

\[ s_c = \frac{\tau_{\text{max}}}{K_o} \]  

[6.6]

\( f_c' \) = concrete compressive strength [MPa]

\( b_f \) = width of FRP sheet [mm]

\( b_c \) = width of concrete prism [mm]

\( K_o \) = initial stiffness of the bond-slip model [MPa/mm]

where:

\[ K_o = \frac{K_a K_c}{K_a + K_c} \]  

[6.7]

\( K_a \) = shear stiffness of the adhesive layer [MPa/mm]. For the finite element analyses of this experimental program a normal adhesive was assumed, therefore, \( K_a = 5 \) MPa/mm (Lu et al., 2005).

\( K_c \) = shear stiffness of the concrete [MPa/mm]

where:

\[ K_c = \frac{G_c}{t_c} \]  

[6.8]

\( G_c \) = elastic shear modulus of concrete [MPa]

\( t_c \) = effective thickness of the concrete [mm]

where:
\[ G_c = \frac{E_c}{2(1 + \nu)} \]  

\[ E_c = \text{modulus of elasticity of concrete [MPa]} \]

\[ \nu = \text{poisson's ratio}. \quad \text{It was assumed that } \nu = 0.15. \]

where:

\[ E_c = 4732.93 \sqrt{f'_c} \]  

The descending linear portion of the curve was calculated as follows:

\[
\text{if } s_o < s < s_f, \text{ then } \tau = \tau_{\text{max}} \frac{s_f - s}{s_f - s_o} \]  

\[ s_f = \text{local slip when bond stress equals zero [mm]} \]

\[ G_f = \text{interfacial fracture energy [MPa/mm]} \]

where:

\[ G_f = 0.308 \beta_o^2 f'_t \]  

The final portion of the curve is as follows:

\[ \tau = 0 \quad \text{if } s > s_f \]

6.3 Geometrical Modeling

Failure by FRP debonding has been shown to occur due to a shear failure of the concrete within a 5-mm (0.2 in.) depth of the FRP-concrete interface; therefore, for the analyses the height of the concrete block was chosen to be 50-mm (2.0 in.) as seen in
Figure 6.5 to reduce computational effort. The length of the concrete block was chosen to be 1-m (39.4 in.) (Figure 6.5), the nominal length of all test specimen concrete blocks as seen in Figure 3.1. The height and width of the concrete elements was kept constant at 10-mm (0.39 in.) and 0.61-m (24-in.), respectively. The bonded length and width of the FRP sheet varied according to the modeled specimen. The relative widths of the FRP and concrete elements varied in the bilinear shear stress-slip model $\beta_\omega$ parameter according to the modeled specimen. A 0.12-m (4.7 in) length of concrete elements was kept free of FRP and interface elements at the leading edge of the concrete block to simulate the debonded region at the lead end of the test specimens.

Quadrilateral nine-node plane stress elements 10-mm (0.4 in.) square were utilized to represent the concrete as seen in Figure 6.6 after an element convergence study was performed, which is discussed in section 6.3.1. The FRP sheet was modeled using two-node general 2-D isobeam elements. The thickness of the isobeam element was kept constant at 0.165-mm (0.0065 in.), the nominal thickness of 1-ply of FRP
sheet as provided by the manufacturer. A general 2-D isobeam element, which includes six degrees of freedom per node, was utilized because the element model included a “death upon rupture” criterion. The death upon rupture criterion allowed for the modeling of FRP rupture at that rupture strain of 1.49% according to the FRP coupon testing described in section 4.3. The isobeam element is an appropriate representation of the FRP sheet since the laminate has a small bending stiffness that would not be captured if a truss element were utilized. The length of each FRP isobeam element was kept constant at 10-mm (0.4 in). The two nodes of the isobeam element, however, were offset slightly from corner nodes of the concrete elements while keeping the nodes coplanar as seen in Figure 6.6. Offsetting the FRP isobeam element nodes from the concrete nodes allowed for the introduction of two-node bilinear shear spring elements located between the concrete element top surface and FRP sheet nodes as seen in Figure 6.6. It should be emphasized that there is no physical separation between the bilinear shear-spring element nodes, FRP isobeam element nodes, and concrete element nodes at the FRP interface as depicted in Figure 6.6. All of the element nodes at the FRP interface lie within the same XZ-plane as seen in Figure 6.6.

The stiffness of the two-node bilinear shear spring elements represented the FRP-concrete interface constitutive model. It is very important to note that the bilinear shear spring elements defined the entire interfacial behavior of the FRP-concrete interface. The stiffness of each shear spring element represented an interfacial shear area between the FRP sheet and the concrete. In order to implement the bilinear bond stress-slip model within ADINA 8.3.3, it was required to define a force-displacement relationship for the interface elements. The force-displacement relationship was
calculated from the bond-stress slip model by multiplying the calculated bond stress of each element by its tributary area. Each element represented a tributary area of:

\[
A_i = L_i b_f
\]  

where \( A_i \) is the tributary area of one interface element in \( \text{mm}^2 \), \( b_f \) is the width of the FRP sheet in millimeters, and \( L_i \) is the length between FRP isobeam element nodes as seen in Figure 6.6 (Ebead and Neale, 2007). Rupture of the bilinear shear spring element was also included in the element formulation. The force in the spring element was set to zero when the displacement was larger than the maximum slip on the force-displacement curve.

6.3.1 Element Convergence Study

To determine the correct element size within the finite element models a convergence study was performed utilizing four element sizes. Four finite element models of Specimen A-0-0-10-0 were formulated utilizing element sizes of 1-mm (0.039 in.), 4-mm (0.16 in.), 8-mm (0.31 in.), and 10-mm (0.39 in.), respectively. The results of the models (Table 6.1) were compared based upon the maximum load...
achieved in the FRP isobeam elements and the maximum principal stress attained in the concrete elements.

<table>
<thead>
<tr>
<th>Mesh Size [mm]</th>
<th>Ultimate Load [k]</th>
<th>Maximum Principal Stress in Concrete Elements [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>11.62</td>
<td>2.717</td>
</tr>
<tr>
<td>4</td>
<td>11.62</td>
<td>2.714</td>
</tr>
<tr>
<td>8</td>
<td>11.63</td>
<td>2.592</td>
</tr>
<tr>
<td>10</td>
<td>11.63</td>
<td>2.542</td>
</tr>
</tbody>
</table>

Table 6.1 illustrates that load convergence occurred with a mesh size of 10-mm (0.39 in.). The convergence of the principal stresses in the concrete elements, however, did not occur until a mesh size of 1-mm (0.039 in.) was utilized. Therefore it was decided that a mesh size of 10-mm (0.39 in.) would be utilized for all subsequent finite element models since comparison between the laboratory specimens and the finite element specimens was based strictly on the ultimate load obtained. However, in the discussion of concrete principal stresses a 1-mm (0.039 in.) mesh size was utilized since this was observed to be the converged state.

6.4 Finite Element Results

This section presents the results of three finite element models constructed of Specimens A-0-0-10-0, A-0-0-5-0, and B-Y-2-5-4. Finite element models of Specimen A-0-0-10-0 and A-0-0-5-0 were modeled since they were control specimens with no FRP anchors. The geometry of the specimens can be seen in section 3.4. A Finite element model of Specimens A-0-0-10-0 was first constructed using the previously discussed published shear stress-slip model. A calibration formulation was calculated based on the finite element results of Specimen A-0-0-10-0, which was then used to
calibrate Specimen A-0-0-5-0. Using the calibrated finite element model of Specimen A-0-0-5-0 as a baseline, a finite element model of Specimen B-Y-2-5-4 was then constructed incorporating FRP anchors. It was decided to model Specimen B-Y-2-5-4 due to the specimens observed failure modes. The failure of the specimen was primarily due to FRP rupture with no premature failure modes observed. Comparisons are made between the finite element model and the test specimen regarding FRP strain profiles and ultimate load carrying capacity.

6.4.1 Specimen A-0-0-10-0

The width of the FRP isobeam and bilinear shear spring elements in the Specimen A-0-0-10-0 finite element model were 254-mm (10 in.), equal to the width of the FRP sheet. The compressive strength of the concrete used in the model was 39.9 MPa (5784 psi), equivalent to the average compressive strength of all specimens with a 10-inch (25.4 cm) FRP bond width. The ultimate tensile strength, ultimate rupture strain, and tensile modulus were input according to the values obtained during the FRP coupon testing as discussed in section 4.3, presented in Table 6.2.

<table>
<thead>
<tr>
<th></th>
<th>Tensile Modulus</th>
<th>Ultimate Tensile Strength, ( f_u )</th>
<th>Ultimate Rupture Strain, ( \varepsilon_{fu} )</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Coupon Average [ksi]</strong></td>
<td>36684.0</td>
<td>539.6</td>
<td>1.49%</td>
</tr>
<tr>
<td><strong>Coupon Average [MPa]</strong></td>
<td>252927.3</td>
<td>3720.7</td>
<td></td>
</tr>
</tbody>
</table>

The stiffness of the nonlinear shear spring elements were input according to the published force-displacement curve (Lu et al., 2005) described in Figure 6.7, which are tabulated in Table 6.3.
The finite element FRP strain profile of Specimen A-0-0-10-0, which utilized the published shear stress-slip model described in Figure 6.7, can be seen in Figure 6.8. Figure 6.8 illustrates the expected behavior during the propagation of debonding of the FRP sheet. As increased load was applied, once the stress transfer zone was fully established, the stress transfer zone translated along the length of the FRP laminate away from the applied load towards the free end of the FRP laminate with its shape remaining constant in agreement with Subramaniam et al., 2007. At the ultimate load of 11.63-kips (51.7 kN) translation of the stress transfer zone along the length of the

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Local Slip at $F_{\text{max}}$, $s_0$ [m]</td>
<td>0.000045</td>
</tr>
<tr>
<td>Max. Local Slip, $s_f$ [m]</td>
<td>0.000297</td>
</tr>
<tr>
<td>Maximum Force, $F_{\text{max}}$ [N]</td>
<td>8446.75</td>
</tr>
</tbody>
</table>
FRP towards the free end of the FRP was observed. As debonding propagated towards the free end of the FRP sheet a constant maximum strain of 4.87 m£ was observed in the debonded FRP.

The finite element model was analyzed using a prescribed time displacement function. As increased displacement was applied to the FRP sheet, FRP strain increased in the sheet until the maximum displacement occurred in the interfacial bilinear shear spring elements at which time the elements ruptured causing debonding propagation towards the unloaded end of the FRP. Failure by debonding in the finite element model occurred due to interface shear spring rupture. The experimental FRP longitudinal strain profile as presented in section 4.7.3.1 can be seen in Figure 6.9.
The maximum obtained load of 11.63-kips (51.7 kN) in the finite element simulation of this specimen is nearly identical to the experimental ultimate load of 11.43-kips (50.8 kN). In the formulation of the bilinear shear stress-slip model it was assumed that the shear stiffness of the adhesive used in the experiments was a normal adhesive that had a shear stiffness of 5 MPa/mm. This assumption may be marginally incorrect causing the difference in strain profile results.

It was noticed that the shape of the strain profiles in Figure 6.8 and Figure 6.9 are very similar up to a load of approximately 10.5-kips (46.7 kN). Region B of Figure 6.9 follows an S-shaped curve, similar with the curves illustrated in Figure 6.8 obtained in the finite element model during propagation of debonding. The strain gage array used in the experiments (5-inch (12.7 cm) strain gage spacing) and the data recording rate of one reading every three seconds, may have affected the shape of measured
strains after debonding propagated suddenly at loads approximating failure of the specimen (region C in Figure 6.9).

The length of FRP over which force transfer during debonding propagation occurred, which is the distance from the point of maximum strain to the point of zero strain, between the finite element and experimental strain profiles differs slightly. In Figure 6.8 the length of the force transfer was approximately 6-inches (15.2 cm), but the force transfer zone in Figure 6.9 was approximately 5-inches (12.7 cm) during the laboratory test. The difference between the experimental and finite element results occurred due to the strain gage spacing during the experimental test. Had the gages been spaced closer it is believed that the longitudinal strain profiles would have matched the finite element profiles.

Due to the nearly identical maximum loads obtained in the finite element model and experimental test of Specimen A-0-0-10-0, it was decided that no calibration of the finite element model was necessary. The maximum load of 11.63-kips (51.7 kN) obtained in the finite element model was 1.75% greater than the maximum load of 11.43-kips (50.8 kN) obtained in the experimental test, therefore, it was decided that no further action was necessary.

6.4.1.1 Concrete Principal Stresses

A band plot at one time step illustrating the unaugmented principal stresses in the concrete elements throughout the concrete block during the propagation of debonding for Specimen A-0-0-10-0 is presented in Figure 6.10. As discussed in section 6.3.1. the principal stresses in the concrete elements were observed to converge when a concrete
element size of 1-mm (0.039 in.) was utilized, therefore, Figure 6.10 was formulated using a 1-mm (0.039 in.) mesh size. The band table illustrates the concrete principal stresses attributed to each color. The band plot illustrates a stress transfer zone that propagates towards the unloaded end of the FRP sheet during debonding propagation. The stress transfer zone has a fixed length and affects a localized region of concrete elements closest to the FRP-concrete interface.

A concrete failure plane was determined utilizing an approximate tensile strength of concrete, estimated as $f'_t = 0.3325 \sqrt{f'_c}$ (MPa), where $f'_c$ (MPa) is the concrete compressive strength of Specimen A-0-0-10-0. The concrete tensile strength was calculated to be 2.1 MPa for Specimen A-0-0-10-0. Utilizing the band table color for a concrete maximum principal stress of 2.1 MPa a failure plane was determined from Figure 6.10. The failure plane has a length of approximately 45-mm (1.77 in.) and a depth of approximately 2-mm (0.079 in.). The depth of the concrete failure plane illustrates the extreme failure localization near the surface of the concrete as FRP debonding occurs. The depth of the failure plane approximately agrees well with the observed laboratory results discussed in section 4.6.1.1 for Specimen A-0-0-10-0, where debonding was identical within a 2-3 mm layer of concrete as it remained attached to the FRP composite sheet.
6.4.2 Specimen A-0-0-5-0

The width of the FRP isobeam and bilinear shear spring elements in the Specimen A-0-0-5-0 finite element model were 127-mm (5 in.), equal to the width of the FRP sheet. The compressive strength of the concrete used in the model was 36.1 MPa (5231 psi), equivalent to the average compressive strength of all specimens with a 5-inch (12.7 cm) FRP bond width.

In the initial formulation of the nonlinear shear spring elements the stiffness of the elements was input according to the published force-displacement parameters described in Table 6.4. Analyzing the Specimen A-0-0-5-0 finite element model utilizing the published shear stress-slip parameters resulted in a maximum load of 6.43-kips (28.6 kN) (see Figure 6.11), which is 17.9% lower than the experimental maximum load of 7.83-kips (34.8 kN). Due to the large difference in the maximum loads between the finite element model and experimental test it was decided that further calibration of the finite element model was necessary. The uncalibrated finite element model and
experimental strain profiles for Specimen A-0-0-5-0 can be seen in Figure 6.11 and Figure 6.12, respectively.

**Table 6.4 — Specimen A-0-0-5-0 Uncalibrated Force-Displacement Parameters**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Local Slip at $F_{\text{max}}$, $s_0$ [m]</td>
<td>0.000048</td>
</tr>
<tr>
<td>Max. Local Slip, $s_f$ [m]</td>
<td>0.000344</td>
</tr>
<tr>
<td>Maximum Force, $F_{\text{max}}$ [N]</td>
<td>4480.56</td>
</tr>
</tbody>
</table>

**Figure 6.11 — Specimen A-0-0-5-0 Finite Element Uncalibrated Strain Profile**
Due to the variability observed in strain measurements, as discussed in section 4.7.4, it was decided to calibrate the finite element model based on the maximum load obtained during the laboratory test.

The large difference in the maximum load obtained in the uncalibrated finite element model and the laboratory test is believed to be influenced by the width of FRP sheet. The narrower 5-inch (12.7 cm) wide sheet tested in Specimen A-0-0-5-0 is believed to have exhibited larger maximum local slip, $s_f$, than that obtained in Specimen A-0-0-10-0. The calculated maximum slip using the published shear-stress slip model gave a good estimate of peak load in the 10-inch (25.4 cm) specimen. Holding the maximum force, $F_{max}$, and the local slip at $\tau_{max}$ ($s_o$) constant, but increasing the
maximum local slip, \( s_r \), in the interfacial bilinear shear stress-slip model of Specimen A-0-0-5-0, allowed for a larger maximum load to be obtained in the FRP sheet.

Due to the nearly identical maximum loads obtained in the finite element model and experimental test of Specimen A-0-0-10-0, it was decided that a calibration factor according to equation 6.16 would properly calibrate the Specimen A-0-0-5-0 finite element model.

\[
s_{f_e} = s_{f_{0a}} \sqrt{\frac{0.254}{b_f}} \tag{6.16}
\]

where:

- \( s_{f_e} \) = calibrated maximum local slip [m]
- \( s_{f_{0a}} \) = published maximum local slip of Specimen A-0-0-10-0 (Table 6.3) [m]
- \( b_f \) = width of FRP sheet [m]

A graph of equation 6.16 (Figure 6.13) illustrates that as the width of the FRP sheet increases the calibrated maximum local slip, \( s_{f_e} \), decreases exponentially and as the width of the FRP sheet decreases the calibrated maximum local slip increases exponentially. Equation 6.16 is consistent with the width ratio factor, \( \beta_{\infty} \), described in equation 6.5 of the bond stress-slip model proposed by Lu et al. (2005), which illustrates that as the width of the FRP sheet increases the width ratio factor decreases causing the maximum local slip when the bond stress equals zero to decrease exponentially.
Utilizing equation 6.16 the calibrated force-displacement parameters for Specimen A-0-0-5-0 are presented in Table 6.5. The calibrated finite element FRP strain profile of Specimen A-0-0-5-0 is presented in Figure 6.14. Utilizing equation 6.16 to calibrate the finite element model of Specimen A-0-0-5-0 resulted in a maximum load of 7.64-kips (34.0 kN), which is 2.4% smaller than the experimental maximum load of 7.83-kips (34.8 kN). The shapes of the strain profile curves in the finite element model during initial debonding are consistent with the experimental strain profile curves, both exhibiting a length of force transfer approximately equal to 5-inches (12.7 cm). At the ultimate load of 7.64-kips (34.0 kN) translation of the stress transfer zone along the length of the FRP towards the free end of the FRP was observed in the finite element model.
6.4.3 Specimen B-Y-2-5-4

The width of the FRP isobeam and bilinear shear spring elements in the Specimen B-Y-2-5-4 finite element model were 127-mm (5 in.), equal to the width of the FRP sheet. The compressive strength of the concrete used in the model was 36.1 MPa (5231 psi), equivalent to the average compressive strength of all specimens with a 5-inch (12.7 cm) FRP bond width.
Using the calibrated finite element model of Specimen A-0-0-5-0 as a baseline, a finite element model of Specimen B-Y-2-5-4 was constructed incorporating FRP anchors. The FRP anchor and sheet geometry can be seen in section 3.4. The failure of Specimen B-Y-2-5-4 occurred due to FRP rupture in front of the FRP anchors as discussed in section 4.6.3.1. As discussed in section 4.7.3.8, the longitudinal strain profile of Specimen B-Y-2-5-4 (Figure 6.15) exhibited an initial transfer length, then a shift in stress transfer as increased load was applied, and a rapid drop in strain behind the FRP anchors. The rapid drop in strains indicated that the FRP anchors were effective in arresting debonding propagation and a minimum amount of force resistance was developed behind the FRP anchors to strain gage B4. Although FRP rupture was observed during the physical test, it should be noted that the FRP rupture strain was not recorded in the strain gages in front of the FRP anchors during the test.

![Figure 6.15 — Specimen B-Y-2-5-4 Experimental Strain Profile](image-url)
To account for the FRP anchors in the finite element model, the shear-spring elements within the splay region of the FRP anchor were modeled as shear-spring elements with their properties modified from those used in Specimen A-0-0-5-0. Three simple finite element models utilizing linear-elastic, bilinear, and elastic-plastic force-displacement relationships within the FRP anchor shear-spring elements were created so the model results could be easily interpreted to determine if FRP anchor behavior was properly captured. To set an upper bound for the anchor shear-spring elements the first finite element model of Specimen B-Y-2-5-4 utilized linear-elastic behavior for the FRP anchor interfacial shear-spring elements. A second finite element model was constructed using a bilinear relationship similar to the shear-spring elements used to represent the FRP-concrete interface, which exhibited loading and unloading branches. Lastly, a finite element model of Specimen B-Y-2-5-4 was constructed utilizing elastic-plastic behavior for the FRP anchor interfacial shear-spring elements. A comparison of the behavior of the three models is presented below.

6.4.3.1 Linear-Elastic FRP Anchor Elements

The initial finite element model of Specimen B-Y-2-5-4 utilized linear-elastic behavior for the interfacial shear spring elements within the 2-inch (5.1 cm) FRP anchor splay diameter. The slope of the force-displacement curve was kept consistent with the calibrated FRP-concrete interface elements utilized in the finite element model of Specimen A-0-0-5-0. The stiffness of the FRP anchor interfacial shear spring elements was input according to Figure 6.7, which is tabulated in Table 6.6. The stiffness of the elements was calculated to be 93345000 N/m, the slope of the force-displacement
curve. The stiffness of the interfacial shear-spring elements outside of the FRP anchor
splay region were kept identical to the calibrated force-displacement parameters of
Specimen A-0-0-5-0 described in section 6.4.2.

Table 6.6 — Specimen B-Y-2-5-4 Linear-Elastic FRP Anchor Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slip [m]</td>
<td>0.000048</td>
</tr>
<tr>
<td>Force [N]</td>
<td>4480.56</td>
</tr>
</tbody>
</table>

![Figure 6.16 — Specimen B-Y-2-5-4 Linear-Elastic FRP Anchor Curve](image)

The finite element model was analyzed utilizing a prescribed time displacement
function. The linear elastic FRP anchor strain profile of the finite element model of
Specimen B-Y-2-5-4 is presented in Figure 6.17.
The strain profiles in Figure 6.17 exhibited similar behavior to the experimental strain profiles displayed in Figure 6.7. As increased displacement was applied, once the stress transfer zone was fully established, the stress transfer zone translated along the length of the FRP laminate away from the applied load towards the FRP anchors. As increased displacement was applied to the FRP sheet, FRP strain increased in the sheet until the maximum displacement occurred in the interfacial bilinear shear-spring elements causing element rupture, which lead to debonding propagation towards the FRP anchors. Once the debonding front reached the FRP anchors, strain increased linearly to failure in the interfacial shear-spring FRP elements in front of the FRP anchor. The FRP elements behind the FRP anchors, towards the unloaded end of the FRP sheet, displayed a rapid drop in strain indicating that the FRP anchors were effective in arresting debonding propagation. The rapid drop in strain behind the FRP
anchors decreased exponentially to zero within 5-inches (12.7 cm) behind the FRP anchors similar to the experimental strain profile indicating that the sheet region behind the FRP anchors was contributing minimally to force resistance.

Despite the similarities in the experimental and linear-elastic finite element strain profile shapes, the corresponding maximum load and strain values varied significantly. Figure 6.17 demonstrates that as increased displacement was applied to the end of the FRP sheet, the load and strain in the FRP sheet increased until the FRP rupture strain of 1.49 mε was obtained causing sheet rupture. The maximum load obtained in the linear-elastic finite element model was 17.12-kips (76.2 kN), which is 37.8% higher than the experimental maximum load of 12.42-kips (55.2 kN).

This finite element formulation illustrates that the strain profile behavior utilizing linear-elastic FRP anchors is similar to the experimental strain profile results. However, the load at failure is much greater in the finite element model illustrating that linear-elastic shear-spring FRP anchor elements cannot replicate the extreme localized behavior of FRP anchors. The laboratory test reached a maximum load of 12.42-kips (55.2 kN), which is significantly lower than the rupture strength of 17.55-kips (78.1 kN) for a 5-inch (12.7 cm) width of FRP sheet. This behavior cannot be modeled utilizing linear-elastic FRP anchor elements since the force and displacement in the linear-elastic FRP anchor elements will continuously increase until the rupture strain of the FRP sheet is obtained.
6.4.3.2 Bilinear FRP Anchor Elements

To limit the force that FRP anchors can develop, the second finite element model was constructed using a bilinear relationship for the FRP anchor shear-spring elements similar to the shear-spring elements used to represent the FRP-concrete interface, which exhibited loading and unloading branches. Holding the local slip at \( \tau_{\text{max}} (s_o) \) and the maximum local slip, \( s_f \), equal to values obtained during the calibration of Specimen A-0-0-5-0, a trial and error procedure was utilized to determine the increase in the maximum force, \( F_{\text{max}} \), in the FRP anchor shear-spring elements necessary to obtain the ultimate force during the laboratory test of Specimen B-Y-2-5-4. The results of the trial and error procedure resulted in stiffness parameters according to Figure 6.18, which are tabulated in Table 6.7. In order to reach the experimental maximum load of 12.42-kips (55.2 kN), it was required to quadruple the maximum force, \( F_{\text{max}} \), in the bilinear anchor shear springs to 17.5-kN (3.93 kips). The bilinear FRP anchor strain profile of the finite element model of Specimen B-Y-2-5-4 is presented in Figure 6.19.

<table>
<thead>
<tr>
<th>Table 6.7 — Specimen B-Y-2-5-4 Bilinear FRP Anchor Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Local Slip at ( F_{\text{max}} ), ( s_0 ) [m]</td>
</tr>
<tr>
<td>Maximum Force, ( F_{\text{max}} ) [N]</td>
</tr>
<tr>
<td>Max. Local Slip, ( s_f ) [m]</td>
</tr>
</tbody>
</table>
Figure 6.18 — Specimen B-Y-2-5-4 Bilinear FRP Anchor Curve

Figure 6.19 — Specimen B-Y-2-5-4 Bilinear FRP Anchor Strain Profile
The strain profiles in Figure 6.19 exhibit similar behavior to the experimental strain profile. As increased displacement was applied to the FRP sheet debonding occurred in the interfacial shear-spring elements in front of the FRP anchor elements. Once the debonding front reached the FRP anchor shear-spring elements strain increased linearly in the FRP anchor and FRP sheet shear-spring elements. The maximum load of 12.54-kips (55.8 kN) is similar to the maximum load obtained during the laboratory test of Specimen B-Y-2-5-4 (12.42-kips), but the maximum strain in the finite element model is greater than the experimental strain by a factor of approximately two.

The shape of the strain profile curves behind the FRP anchors towards the unloaded end of the FRP sheet in the finite element model differ slightly from the laboratory test. The strain profiles in Figure 6.19 display a rapid drop in strain behind the FRP anchors that is much steeper (almost vertical) than the experimental strain profile. This indicates that this region is not contributing to force resistance. It is believed that the stiffness (slope) of the loading portion of the FRP anchor shear-spring elements is so steep that it is not allowing force resistance to develop in the region behind the FRP anchors. Furthermore, the fourfold increase in spring peak force, $F_{\text{max}}$, effectively locks the majority of the force transfer in the section corresponding to the FRP anchor springs and allows very little development of force in the FRP sheet behind the anchor section.
6.4.3.3 Elastic-Plastic FRP Anchor Elements

The last finite element model of Specimen B-Y-2-5-4 was constructed utilizing elastic-plastic behavior for the FRP anchor interfacial shear-spring elements. Holding the local slip at $\tau_{\text{max}}$ ($s_0$) equal to value obtained during the calibration of Specimen A-0-0-5-0 and the maximum force, $F_{\text{max}}$, equal to 17.5-kN (3.93 kips), the value obtained during the bilinear modeling of Specimen B-Y-2-5-4, the maximum local slip, $s_f$, was set equal to 1-m (39.4 in.) to properly model elastic-plastic behavior. The stiffness of the FRP anchors interfacial shear spring elements was input according to Figure 6.20, which is tabulated in Table 6.8. The bilinear FRP anchor strain profile of the finite element model of Specimen B-Y-2-5-4 is presented in Figure 6.21.

<table>
<thead>
<tr>
<th>Table 6.8 — Specimen B-Y-2-5-4 Elastic-Plastic FRP Anchor Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Local Slip at $F_{\text{max}}$, $s_0$ [m]</td>
</tr>
<tr>
<td>Maximum Force, $F_{\text{max}}$ [N]</td>
</tr>
</tbody>
</table>
Figure 6.20 — Specimen B-Y-2-5-4 Elastic-Plastic FRP Anchor Curve

Figure 6.21 — Specimen B-Y-2-5-4 Elastic-Plastic FRP Anchor Strain Profile
The shape of the strain profile in Figure 6.21 is similar to the bilinear strain profile shown in Figure 6.19. The behavior of the bilinear and elastic-plastic FRP anchor shear-spring elements is almost identical. The maximum load of 13.29-kips (59.1 kN) obtained in the finite element model is similar to the maximum load obtained during the laboratory test of Specimen B-Y-2-5-4 (12.42-kips), however, the maximum strain in the finite element model is greater than the experimental strains by a factor of two. Similar to Figure 6.19 the shape of the strain profile curves behind the FRP anchors is nearly vertical indicating that the stiffness (slope) of the loading portion of the FRP anchor shear-spring elements is so steep that it is not allowing force resistance to develop in the region behind the FRP anchors.

### 6.4.3.4 Specimen B-Y-2-5-4 Summary

Three finite element models of Specimen B-Y-2-5-4 were constructed each utilizing different constitutive behaviors to represent the FRP anchor interfacial shear-spring elements. The first model utilized linear-elastic behavior, the second model employed a bilinear behavior, and elastic-plastic behavior was applied to the last model. None of the finite element models were able to accurately formulate the experimental test strain profiles, maximum strain, or ultimate load. It is believed that the behavior of an FRP anchor is a combination of the three models with an initial linear-elastic loading branch, followed by a large drop in force capacity to a reserve strength that has infinite slip to failure, as displayed in Figure 6.22.

The constitutive behavior of an FRP anchor is dependent on the anchor failure mode (anchor shear, delamination, or pullout). It is believed that the perfect FRP
anchor would function as a rigid link between the FRP sheet and concrete elements transferring all of the force from the FRP sheet into the concrete elements. Further research into the constitutive behavior of FRP anchors is needed to properly model an FRP anchor using interfacial shear-spring elements.

![Figure 6.22 — Proposed FRP Anchor Capacity](image)

### 6.5 Summary

This chapter presented the formulation and results of a 2-D plane stress finite element model developed to study the behavior between FRP sheets bonded to reinforced concrete elements. Single-shear finite element models of FRP sheets bonded to a concrete substrate were loaded in tension to model the behavior of Specimens A-0-0-10-0, A-0-0-5-0, and B-Y-2-5-4. To validate the finite element models, comparisons were made between the finite element models and the test specimens regarding FRP strain profiles and ultimate load carrying capacity. A calibration formulation was
proposed that was used to calibrate the finite element model of control Specimen A-0-0-5-0. The calibration factor calculates the maximum local slip, $s_f$, based on the bonded width of FRP sheet. A finite element model of Specimen B-Y-2-5-4 illustrated that the behavior of FRP anchors is extremely localized and cannot be represented by simple linear-elastic, bilinear, or elastic-plastic constitutive relations. Further research into the constitutive behavior of FRP anchors is needed to properly model the FRP anchor region using interfacial shear-spring elements.
CHAPTER 7
SUMMARY AND CONCLUSIONS

7.1 Summary

The objective of this research program was to study the effects of anchoring techniques of FRP sheets used to improve the performance of strengthened reinforced concrete members primarily in shear applications. Because the most common failure mode of FRP-strengthened reinforced concrete members is debonding, the goal of the research was confined to examine the effects of anchoring patterns to avoid or delay debonding of the FRP laminates from the concrete surface. No models characterizing the behavior of FRP sheets anchored to concrete were found in the literature. Tests were developed to study the gain in strengthening capacity of FRP sheets when anchored to the concrete surface using FRP anchors. The tests were also intended to provide an understanding of the various failure modes that occur when using this technique.

Six rectangular reinforced concrete blocks of a constant geometry and reinforcement were strengthened with carbon fiber reinforced composite sheets. Three series (A, B, and C) consisting of twelve total specimens were tested. Specimen group A, which was a control group, consisted of a total of 2 tests conducted on one block and had one ply of bonded FRP with no FRP anchors. The goal of specimen group A was to set a baseline for subsequent tests to establish the ultimate load when FRP debonding occurred and to establish the distribution of strains throughout the FRP bonded length and width during the debonding process. The goal of specimen group B, which
consisted of a total of 7 tests conducted on four concrete blocks, studied the effects of using one row of ¼-inch (0.64 cm), ½-inch (1.27 cm), and ¾-inch (1.9 cm) diameter FRP anchors to study the efficiency of individual anchors to engage a given width of FRP material. An FRP anchor length of 2-inches (5.1 cm) was kept constant throughout all tests in this specimen group as anchor length was determined to be a non-controlling factor because anchor pullout was not observed in any of the initial test specimens. The goal of specimen group C, which consisted of three tests conducted on two concrete blocks, studied the effects of using FRP anchors to fasten one ply of bonded FRP sheet having a 10-inches (25.4 cm) width. Concepts regarding FRP anchor diameter, FRP anchor splay diameter, FRP anchor length, FRP anchor spacing, and FRP bonded sheet length studied during the testing of specimen group B were applied to specimen group C to confirm that the theories developed in the experiments worked for a wider bonded FRP sheet.

The observed behavior was analyzed in terms of local response of the constituent materials and the global performance of the FRP anchorage system. The local performance was evaluated measuring strains developed in the FRP, load at the initiation of debonding, and ultimate load at specimen failure. The global performance of the FRP anchorage system was analyzed based on the observed failure modes. Based on the global performance of multiple specimens a behavioral model was developed allowing the design of additional anchor geometries for various FRP sheet-strengthening conditions.

A 2-D plane stress finite element model was developed using ADINA 8.3.3 to study the bond behavior between FRP sheets attached to the surface of reinforced
concrete elements. In particular, two dimensional finite element models of Specimens A-0-0-10-0, A-0-0-5-0, and B-Y-2-5-4, were used to analyze the interfacial debonding behavior between FRP sheets bonded to the concrete surface. To validate the finite element models, comparisons were made between the finite element model and the test specimens regarding FRP strain profiles and ultimate load carrying capacity. A calibration formulation was proposed that calculated the maximum local slip, $s_f$, based on the bonded width of FRP sheet.

7.2 Global Response

Several fundamental characteristics were evident from the global response observed during the experimental investigation of fastening FRP laminates with FRP anchors. The overall effectiveness of FRP anchors was found to depend upon several factors, most noticeably the ratio of the FRP anchor splay diameter to FRP anchor diameter. This ratio governs the ability of the FRP anchor to engage a certain width of FRP composite sheet. Larger splay diameters engaged wider regions of the FRP sheets. Therefore, providing a large anchor splay diameter with a small anchor diameter caused failure in the anchor because the force being transferred by the sheet into the anchor exceeded the anchor capacity. Conversely, providing a small splay diameter with a large anchor diameter was not efficient since the anchor shear strength was not fully mobilized. Small splay diameters, however, do not engage the entire FRP sheet width and lead to combined failure modes (localized sheet rupture and debonding of FRP sheet regions not engaged by the FRP anchor).
The overall effectiveness of FRP anchors was found also to be affected by the selected anchor spacing. Spacing anchors longitudinally along the composite sheet was not necessary to develop higher sheet forces, but rather was important to increase ductility of the system before failure. Anchors placed across the width of the composite sheet were more efficient in developing higher forces in the anchored FRP sheet. Spacing the anchors such that their splays nearly overlapped was very important as each anchor could engage a specific width of FRP sheet equal to the splay diameter of the FRP anchor. Spacing the anchors with a gap in-between anchor splays did not engage the composite sheet between the anchors resulting in composite sheet failure before development of the ultimate strength of the FRP anchors.

The length of FRP sheet bonded directly to the concrete surface behind the anchors also affected the effectiveness of the FRP anchor system. Providing a longer bond length of composite sheet developed a more ductile debonding process and increased the total force at specimen failure. The effectiveness of FRP anchors is dependent upon their ability to prevent FRP laminate debonding throughout the entire FRP sheet length. Only the anchored specimens that were able to prevent complete FRP sheet debonding were able obtain approximately 70% or more of the ultimate strength of the FRP, with the exception of one specimen. The ability of FRP anchors to prevent debonding was affected by several parameters that caused premature failure modes associated to the anchorage system including anchor delamination, anchor pullout, and splitting failure between the FRP anchor and laminate sheet.
7.3 Local Response

The first major observation noted after examination of measured strains at different positions on the FRP sheet was the location of the maximum-recorded strain. It was expected that axial strain would be the greatest in the middle of the FRP laminate for specimens with no FRP anchors. However, it was illustrated that the location of the maximum strain occurred in a strain gage row near the edge of the FRP sheet. For specimens with FRP anchors centered about the sheet centerline it was expected that the maximum strain would occur in the center of the FRP laminate directly in line with the FRP anchor, which was shown to be true. For specimens with FRP anchors that were not centered about the sheet centerline it was expected that the maximum-recorded strain would occur inline with the center of the FRP anchors and lower strains would occur in the centerline of the FRP sheet at the edge of an FRP splay region. As expected it was illustrated that the location of maximum strain for specimens with non-centered FRP anchors occurred in the edge strain gage rows, which were located closer to the center of the FRP anchor diameter than the center strain gage row that was located in the center of the FRP sheet outside of the anchor splay region.

The second observation was the variability in the recorded percentage of \( \frac{\varepsilon_{\text{max}}}{\varepsilon_{\text{ult}}} \) compared to \( \frac{P_{\text{test}}}{P_{\text{ult}}} \). The wide variability in the maximum-recorded local strains was believed to be generated from local variation in local material properties of the FRP laminate and concrete substrate, or from localized FRP anchor effects. Variability in behavior introduced through the properties of the concrete substrate was believed to be
the major contributor strain variability. The concrete surface strength was highly variable due to irregular aggregate and paste distribution and also because of irregularities introduced during surface preparation of the specimens.

The third observation noted in the local response of individual strain gages was the inability to record FRP rupture strain, even in cases where FRP sheet rupture was observed. This phenomenon was believed to occur due to localized FRP material properties or the inability to record peak strain values using a discrete number of strain gages. It was noted that FRP rupture was observed for numerous specimens leading to the conclusion that the FRP rupture strain was reached, but not recorded. It was illustrated that FRP rupture occurred in localized locations causing the redistribution of forces inducing global FRP rupture or debonding.

7.4 Conclusions

FRP anchors are effective in fastening FRP laminates to reinforced concrete elements allowing for the development of the full rupture strength of the composite sheet. FRP anchors provide an additional strength to the FRP composite allowing for the development of the ultimate strength of the FRP sheet.

The effect of the FRP anchor geometry is an important parameter that governs the shear strength of FRP anchors. It was determined that FRP anchor length was not a governing parameter for the anchor length that was used in this research project. The splay diameter determines the effective width of FRP sheet engaged by an individual anchor and determines the required diameter of the FRP anchor to transfer the force generated on the FRP sheet into the concrete substrate. The FRP anchor diameter is
directly related to the force being transferred from the FRP sheet into the concrete substrate. The FRP anchor diameter, therefore, has to be determined in accordance with the width of FRP laminate being engaged by each anchor splay. An FRP anchor shear strength of 35.8 k/in² (246.8 MPa) was calculated based on the global failure modes of multiple specimens, which can be utilized to select an anchor splay diameter, which will determine the effective width of FRP sheet engaged by an individual anchor.

7.5 Areas of Future Research

Given the variation in the recorded FRP sheet strain and the erratic propagation of the debonding crack front, it was difficult to evaluate the efficiency of an anchoring system based on the local response with a discrete number of strain gages. Therefore, further research into the development of design guidelines that is reflective of the global response of the specimen is warranted. Furthermore, to achieve this, a parametric study utilizing the percent of FRP rupture sheet strength obtained at failure as the dependent variable could be conducted. Further experiments utilizing the FRP anchor diameter and splay diameter as the independent variables could be investigated. The results of the parametric study would give extra insight into the cause of premature failure modes, the loads at which these premature failure modes occur, and would validate the derived FRP anchor shear strength.

Future tests consisting of unbonded FRP sheets fastened utilizing FRP anchors with a large splay to anchor diameter ratio forcing the anchor to fail in shear would allow for the direct formulation of the FRP anchor shear strength provided that premature failure modes do not occur.
Additional control specimen investigations utilizing specimens without FRP anchors, constant FRP sheet bond lengths, and varying FRP sheet bond widths could be used to further validate the proposed finite element model calibration formulation. Subsequent to the calibration of the control specimen’s finite element models, further research into the constitutive behavior of FRP anchors is needed to properly model the FRP anchor region using interfacial shear-spring elements. The constitutive behavior of FRP anchors could possibly be determined by testing specimens with unbonded FRP sheets fastened to the concrete substrate utilizing FRP anchors, therefore isolating the behavior of FRP anchors.

Perhaps the most interesting area of future research would be conducted on specimens utilizing FRP sheets fabricated with FRP bundles that are woven normal to one another, unlike the FRP sheets used in this experimental program that consisted of longitudinal fiber bundles only. Interwoven fiber bundles would drastically change the behavior of an FRP anchor and might not limit the effective width of an FRP anchor equal to the splay diameter. Interwoven fiber bundles would also help prevent the premature splitting failure mode, which is discussed in further detail in the next section.

7.5.1 Prevention of Failure Modes

Several factors were noted during the premature failure of specimens utilizing FRP anchors with a ¾-inch (1.91 cm) anchor diameter and 4-inch (10.2 cm) splay diameter that could be prevented.

During the installation of ¾-inch (1.91 cm) FRP anchors it was observed that large openings existed in the FRP sheet behind the FRP anchor due to the spreading of
the longitudinal FRP fiber bundles necessary to pass the FRP anchor through the FRP sheet (see Figure 7.1). It is believed that these sheet openings caused the splitting failure mode observed during the failure of Specimens C-X-4-10-6 and C-Y-4-10-6 as discussed in sections 4.6.3.4 and 4.6.3.5. It is thought that placing a transverse FRP sheet behind the FRP anchors during the installation of the FRP anchors subsequent to passing the FRP anchor through the FRP sheet and prior to impregnating the FRP anchor splay could help prevent the splitting failure mode by holding the longitudinal fiber bundles in this region behind the FRP anchors together.

![Figure 7.1 — FRP Sheet Openings behind FRP Anchors](image)

FRP anchors with a large splay diameter are more susceptible to the delamination failure mode since the anchor must transfer the force generated over a large width FRP sheet into the concrete substrate. In order to prevent the delamination
failure mode observed during the failure of specimens with 4-inch (10.2 cm) splay
diameters, it is believed that placing an additional transverse FRP sheet FRP over the
FRP anchor splay region following impregnation of the FRP anchor splay would help
prevent delamination between the FRP anchor splay and FRP sheet interface.

During the failure of Specimen C-X-4-10-6, as discussed in section 4.6.3.3, FRP
anchor pullout was observed to occur. It is believed that anchor pullout occurred due to
the inability of epoxy saturant to impregnate the entire anchor. Since the FRP anchor
had a ¾-inch (1.91 cm) anchor diameter it is believed that the FRP anchor only partially
impregnated with epoxy saturant causing the premature failure mode. In order to
prevent FRP anchor pullout it is believed that formulating a new method to construct
FRP anchors where the FRP sheet is impregnated before the FRP anchor is rolled and
tied would ensure that the entire FRP anchor is saturated with epoxy and would help
prevent FRP anchor pullout. Examination of the effects of different FRP anchor
fabrication techniques on behavior of FRP-strengthened elements would therefore seem
warranted.
APPENDIX A

MATERIALS TEST RESULTS

Figure A.1 — FRP_A Stress-Strain Diagram

Figure A.2 — FRP_B Stress-Strain Diagram
Figure A.3 — FRP_C Stress-Strain Diagram

Figure A.4 — FRP_D Stress-Strain Diagram
Figure A.5 — FRP_E Stress-Strain Diagram

Figure A.6 — FRP_F Stress-Strain Diagram
Figure A.7 — FRP_G Stress-Strain Diagram

Figure A.8 — FRP_H Stress-Strain Diagram
Figure A.9 — FRP_I Stress-Strain Diagram

Figure A.10 — FRP_J Stress-Strain Diagram
APPENDIX B

SPECIMEN A-0-0-5-0 MEASURED SLIP

Figure B.1 — Specimen A-0-0-5-0 Position Transducer Key

Figure B.2 — Position Transducer 01
Figure B.3 — Position Transducer 02

Figure B.4 — Position Transducer 03
Figure B.5 — Position Transducer 04

Figure B.6 — Position Transducer 05
Figure B.7 — Position Transducer 06

Figure B.8 — Position Transducer 07
APPENDIX C

SPECIMEN A-0-0-10-0 MEASURED SLIP

Figure C.1 — Specimen A-0-0-10-0 Position Transducer Key

Figure C.2 — Position Transducer 01
Figure C.3 — Position Transducer 02

Figure C.4 — Position Transducer 03
Figure C.5 — Position Transducer 04

Figure C.6 — Position Transducer 05
Figure C.7 — Position Transducer 06

Figure C.8 — Position Transducer 07
APPENDIX D

SPECIMEN A-0-0-5-0 MEASURED STRAIN

Figure D.1 — Specimen A-0-0-5-0 Strain Gage Key

Figure D.2 — Strain Gages A1, B1, C1
Figure D.3 — Strain Gages A2, B2, C2

Figure D.4 — Strain Gages A3, B3, C3
Figure D.5 — Strain Gages A4, B4, C4

Figure D.6 — Strain Gages A5, B5, C5
APPENDIX E

SPECIMEN A-0-0-10-0 MEASURED STRAIN

Figure E.1 — Specimen A-0-0-10-0 Strain Gage Key

Figure E.2 — Strain Gages A1, B1, C1
Figure E.3 — Strain Gages A2, B2, C2

Figure E.4 — Strain Gages A3, B3, C3
Figure E.5 — Strain Gages A4, B4, C4

Figure E.6 — Strain Gages A5, B5, C5
Figure E.7 — Strain Gages A6, B6, C6
APPENDIX F

SPECIMEN B-Z-2-5-2 MEASURED STRAIN

Figure F.1 — Specimen B-Z-2-5-2 Strain Gage Key

Figure F.2 — Strain Gages A1, B1, C1
Figure F.3 — Strain Gages A2, B2, C2

Figure F.4 — Strain Gages A3, B3, C3
Figure F.5 — Strain Gages A4, B4, C4

Figure F.6 — Strain Gages A5, B5, C5
Figure G.1 — Specimen B-Z-2-5-4 Strain Gage Key

Figure G.2 — Strain Gage B1
Figure G.3 — Strain Gage B2

Figure G.4 — Strain Gage B3
Figure G.5 — Strain Gage B4

Figure G.6 — Strain Gage B5
Figure G.7 — Strain Gage B6

Figure G.8 — Strain Gage B7
Figure G.9 — Strain Gage B8

Figure G.10 — Strain Gages A9, B9, C9
Figure G.13 — Strain Gage B12
APPENDIX H

SPECIMEN B-Z-4-5-4 MEASURED STRAIN

Figure H.1 — Specimen B-Z-4-5-4 Strain Gage Key

Figure H.2 — Strain Gage B1
Figure H.3 — Strain Gage B2

Figure H.4 — Strain Gage B3
Figure H.5 — Strain Gage B4

Figure H.6 — Strain Gage B5
Figure H.7 — Strain Gage B6

Figure H.8 — Strain Gage B7
Figure H.9 — Strain Gages A8, B8, C8

Figure H.10 — Strain Gage B9
Figure H.11 — Strain Gage B10
APPENDIX I

SPECIMEN B-W-2-5-4 MEASURED STRAIN

Figure I.1 — Specimen B-W-2-5-4 Strain Gage Key

Figure I.2 — Strain Gage B1
Figure I.3 — Strain Gage B2

Figure I.4 — Strain Gage B3
Figure I.5 — Strain Gage B4

Figure I.6 — Strain Gage B5
Figure I.7 — Strain Gage B6

Figure I.8 — Strain Gage B7
Figure I.9 — Strain Gage B8

Figure I.10 — Strain Gages A9, B9, C9
Figure I.11 — Strain Gage B10

Figure I.12 — Strain Gage B11
Figure I.13 — Strain Gage B12
APPENDIX J

SPECIMEN B-Z-4-5-6 MEASURED STRAIN

Figure J.1 — Specimen B-Z-4-5-6 Strain Gage key

Figure J.2 — Strain Gage B1
Figure J.3 — Strain Gage B2

Figure J.4 — Strain Gage B3
Figure J.5 — Strain Gage B4

Figure J.6 — Strain Gage B5
Figure J.7 — Strain Gage B6

Figure J.8 — Strain Gage B7
Figure J.9 — Strain Gages A8, B8, C8

Figure J.10 — Strain Gage B9
Figure J.11 — Strain Gage B10
APPENDIX K

SPECIMEN B-Y-2-5-4 MEASURED STRAIN

Figure K.1 — Specimen B-Y-2-5-4 Strain Gage Key

Figure K.2 — Strain Gage B1
Figure K.3 — Strain Gage B2

Figure K.4 — Strain Gage B3
Figure K.7 — Strain Gage B6

Figure K.8 — Strain Gages A7, B7, C7
Figure K.9 — Strain Gage B8

Figure K.10 — Strain Gage B9
Figure K.11 — Strain Gage B10
APPENDIX L

SPECIMEN B-X-2-5-4 MEASURED STRAIN

Figure L.1 — Specimen B-X-2-5-4 Strain Gage Key

Figure L.2 — Strain Gage B1
Figure L.3 — Strain Gage B2

Figure L.4 — Strain Gage B3
Figure L.5 — Strain Gages A4, B4, C4

Figure L.6 — Strain Gage B5
Figure L.7 — Strain Gage B6

Figure L.8 — Strain Gage B7
APPENDIX M

SPECIMEN C-X-4-10-6 MEASURED STRAIN

Figure M.1 — Specimen C-X-4-10-6 Strain Gage Key

Figure M.2 — Strain Gage B1
Figure M.3 — Strain Gages A2, B2, C2

Figure M.4 — Strain Gage B3
Figure M.5 — Strain Gage B4

Figure M.6 — Strain Gages A5, B5, C5
Figure M.7 — Strain Gages A6, B6, C6

Figure M.8 — Strain Gage B7
APPENDIX N

SPECIMEN C-Y-4-10-6 MEASURED STRAIN

Figure N.1 — Specimen C-Y-4-10-6 Strain Gage Key

Figure N.2 — Strain Gage B1
Figure N.3 — Strain Gage B2

Figure N.4 — Strain Gage B3
Figure N.5 — Strain Gage B4

Figure N.6 — Strain Gage B5
Figure N.7 — Strain Gage B6

Figure N.8 — Strain Gage B7
Figure N.9 — Strain Gage B8

Figure N.10 — Strain Gages A9, B9, C9
Figure N.11 — Strain Gage B10

Figure N.12 — Strain Gage B11
APPENDIX O

SPECIMEN C-U-2-10-4 MEASURED STRAIN

Figure O.1 — Specimen C-U-2-10-4 Strain Gage Key

Figure O.2 — Strain Gage B1
Figure O.3 — Strain Gage B2

Figure O.4 — Strain Gage B3
Figure O.7 — Strain Gage B6

Figure O.8 — Strain Gage B7
Figure O.9 — Strain Gage B8

Figure O.10 — Strain Gages A9, B9, C9
Figure O.11 — Strain Gage B10

Figure O.12 — Strain Gage B11
APPENDIX P

REINFORCEMENT PHOTOGRAPHS

Figure P.1 — Steel Reinforcement (Plan View)

Figure P.2 — Steel Reinforcement (Side View)
Figure P.3 — Steel Reinforcement in Formwork
APPENDIX Q

DIRECT SHEAR TEST SETUP PHOTOGRAPHS

Figure Q.1 — W8x35 Fastening Beams

Figure Q.2 — Post-Tensioned Concrete Specimen
Figure Q.3 — FRP Connection (Side View)

Figure Q.4 — FRP Connection (Isometric View)
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