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New Carbon Fiber Anchor to Secure Carbon Fiber-Reinforced Polymer (CFRP) Flexural Strengthening Sheets

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Kansas State University Transportation Center

Kansas Department of Transportation

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This experimental program designed, built, and tested six T-beams in four-point bending. The first beam was tested as a control specimen, while the second beam was strengthened with one sheet of CFRP V-Wrap C200HM with no anchors. The third beam was strengthened similarly to the second beam with CFRP splay anchors added to each shear span. The fourth beam was strengthened with a CFRP sheet and four splay anchors per shear span, while the fifth beam contained five splay anchors per shear span. The sixth beam was strengthened with five anchors per shear span with smaller splay areas for each anchor. Test results showed that the use of six anchors per shear span led to full flexural capacity by attaining CFRP rupture. However, although the use of four and five anchors per shear span significantly delayed the debonding, the CFRP sheets debonded at loads close to the load needed to rupture the CFRP sheet when six anchors were used.

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PREFACE

The Kansas Department of Transportation's (KDOT) Kansas Transportation Research and New-Developments (K-TRAN) Research Program funded this research project. It is an ongoing, cooperative and comprehensive research program addressing transportation needs of the state of Kansas utilizing academic and research resources from KDOT, Kansas State University and the University of Kansas. Transportation professionals in KDOT and the universities jointly develop the projects included in the research program.

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This experimental program designed, built, and tested six T-beams in four-point bending. The first beam was tested as a control specimen, while the second beam was strengthened with one sheet of CFRP V-Wrap C200HM with no anchors. The third beam was strengthened similarly to the second beam with CFRP splay anchors added to each shear span. The fourth beam was strengthened with a CFRP sheet and four splay anchors per shear span, while the fifth beam contained five splay anchors per shear span. The sixth beam was strengthened with five anchors per shear span with smaller splay areas for each anchor. Test results showed that the use of six anchors per shear span led to full flexural capacity by attaining CFRP rupture. However, although the use of four and five anchors per shear span significantly delayed the debonding, the CFRP sheets debonded at loads close to the load needed to rupture the CFRP sheet when six anchors were used.

Acknowledgements

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Chapter 1: Introduction

1.1 Background

As infrastructure deteriorates, creative solutions have been proposed to repair and strengthen structures rather than tear them down. One common technique utilizes externally bonded fiber-reinforced polymers (FRPs) to strengthen concrete structures. Although FRP is an economical and environmentally friendly option, continued research is needed to address the disadvantages of its application. The primary disadvantage of FRP use is debonding, in which the bond between the epoxy and the concrete, or within the concrete substrate, fails before the full capacity of the FRP is reached, resulting in decreased FRP cost effectiveness because the full strength of the FRP is not utilized. One solution utilizes an anchorage system to maintain FRP attachment to the beam so that the full strength of the FRP can be reached, resulting in a rupture failure of the FRP rather than debonding. Many research projects have studied various anchorage systems to determine optimal anchoring of FRP sheets, but new research projects are needed to improve current anchorage systems.

1.2 Objectives

This study had three main objectives. The first objective sought to evaluate a carbon FRP (CFRP) fiber-splay anchorage system using fiber bundles inserted into the soffit and then splayed onto the CFRP sheet(s) in beams strengthened with CFRP sheets in flexure. Therefore, the behaviors of four beams with various splay anchorage configurations were compared to a control beam and a strengthened beam with no anchorage. The second objective sought to optimize cost versus gain in strength for splay anchorage by utilizing three different splay anchor spacings to correlate the amount of fiber used for the anchors with the increase in beam strength over the control and strengthened beam with no anchorage. The third main objective sought to qualify strain improvement due to splay anchors by calculating strain improvement ratios for each strengthened beam. This report includes the complete design, construction, and testing of the beam specimens, and the results of each objective are addressed in the conclusions section.

1.3 Scope

Following the first introductory chapter of this report, Chapter 2 is a literature review that focuses on externally bonded FRP, CFRP splay anchors, and U-wrap anchorage systems. Chapter 3 discusses the design, construction, and strengthening of beam specimens, while Chapter 4 describes material testing for the actual strength of the concrete and steel rebar. Chapter 5 clarifies the experimental set up and test results from each beam specimen, and Chapter 6 contains the summary and conclusions from this study as well as recommendations for future work.

Chapter 2: Literature Review

This literature review examines previous research on externally bonded FRP, carbon splay anchors, and U-wrap anchorage systems, including the debonding of FRP, in correlation to the work of this study.

2.1 Externally Bonded FRP

Ali-Ahmad, Subramaniam, and Ghosn (2006) performed experimental research to investigate debonding between concrete and FRP sheets. Concrete blocks measuring 330 mm (13 inches) in length, 125 mm (4.92 inches) in width, and 125 mm (4.92 inches) in height were cast, and an FRP composite was applied to one side of each block. A testing apparatus was developed that loaded the specimens in direct shear by applying a load directly to the FRP attached to the concrete while supports held the concrete block in place. A quasi-static monotonic and a quasi-static cyclic direct shear test were run on various samples, and surface strains in the FRP and the concrete were obtained using digital image correlation. This technique, which utilizes the mathematical correlation method, analyzes digital images of a specimen undergoing deformation, and outputs strain values. Results from all specimens yielded a debonding failure, leading to the conclusion that an interfacial crack causes FRP debonding from a concrete surface. The crack grew as the load increased. Once the crack reached a critical length, it continued to propagate, but the load on the specimen remained constant, causing increased slippage of the FRP sheet. This study also used direct tensile tests to compare debonded FRP sheets with control FRP coupons. Results showed that the two had the same load response, meaning the debonded sheet did not reach any level of damage in the FRP.

2.2 CFRP Splay Anchors

Orton, Jirsa, and Bayrak (2008) studied the effect of CFRP anchors on the overall tensile strength of an externally bonded CFRP sheet. The research utilized 40 specimens, each constructed of two concrete blocks measuring 20 cm (7.87 inches) wide and 81 cm (31.89 inches) long. The blocks were attached with an externally bonded sheet of CFRP to simulate a concrete beam with a crack at the midspan. The blocks were simply supported and loaded at the midspan to subject the

CFRP to tension forces. The design parameters included size, number, and spacing of CFRP anchors, as well as offset height and angle between the two blocks, the type of CFRP material used, and the surface preparation technique. All design parameters varied throughout the samples to determine correlations. First, seven beams were tested to examine the number, size, and spacing of anchors. From the results of these beams, the authors concluded that the total cross-section of the anchor should be at least two times greater than the area of the CFRP sheet and that a larger number of smaller size anchors is more effective than fewer of a larger anchor size. The second test investigated how the offset height of the two blocks and the transition slope between them affected capacity of the CFRP sheet. Results showed that the beams reached loads with full capacities of CFRP sheets when a 1:4 transition slope was used and the CFRP sheet was properly anchored with a splay anchor. The third test changed the type of CFRP material. One material had decreased ultimate strength, which resulted in decreased beam strength, thereby requiring more anchors to reach full strength. Finally, to study surface preparation, the researchers created two specimens that had plastic wrap between the concrete and the CFRP sheet so only the anchors held the sheet onto the beam. Results showed that CFRP reached full capacity if the sheet was properly anchored to the beam with a splay anchor and the anchors had enough capacity. The authors concluded that surface preparation is inconsequential if adequate anchorage is applied because the anchors hold the CFRP sheet in place.

Ali, Abdalla, Hawileh, and Galal (2014) conducted an experimental study to investigate the behavior of concrete beams strengthened with CFRP sheets and plates and CFRP anchors. Although the experiment utilized 16 beam specimens, the literature only contained information regarding five of the specimens. The beams, which measured 120 mm (4.72 inches) wide, 240 mm (9.45 inches) tall, and 1,840 mm (72.44 inches) long with a clear span of 1,690 mm (66.54 inches), were loaded in four-point bending. The first beam was a control beam that failed at a load of 67.98 kN (15.28 kips). The second beam was strengthened with a CFRP sheet that measured 1,000 mm (39.37 inches) long; this beam failed at a load of 73.01 kN (16.41 kips). The third beam was strengthened with the same CFRP sheet layout as the second beam with the addition of one CFRP anchor on each end of the sheet. The anchor holes were 10 mm (0.39 inches) in diameter and 40 mm (1.57 inches) deep. This beam failed at a load of 80.15 kN (18.01 kips). The fourth beam was strengthened with a CFRP plate measuring 1,000 mm (39.37 inches) long; this beam failed at a load of 65.02 kN (14.61 kips). The fifth beam was strengthened with a CFRP plate that measured 1,000 mm (39.37 inches) long and two CFRP anchors, one at each end of the plate. The anchor holes were 10 mm (0.39 inches) in diameter and 80 mm deep (3.15 inches). This beam failed at a load of 78.28 kN (17.59). Results from this experiment showed an increase in the ultimate load for beams with CFRP anchors. The authors concluded that the control beam had the most ductility, while the strengthened beams had decreased ductility. They also observed that the anchors did not significantly contribute to the flexural stiffness of the beams.

2.3 U-wrap Anchorage Systems

Pham and Al-Mahaidi (2006) evaluated beams retrofitted with FRP and associated debonding failure loads. Experimental program number two, which focused on U-wrap anchorage systems, tested eight rectangular concrete beams with widths of 140 mm (5.51 inches) and heights of 260 mm (10.24 inches). The tension steel consisted of three bars with diameters of 12 mm (0.5 inches) each, and the compression steel was comprised of two bars with diameters of 12 mm (0.5 inches) each. All the beams were strengthened with CFRP, but the first beam had no anchorage. Two of the beams had one non-prestressed U-strap at the end of the CFRP sheet, while the two other beams had one prestressed U-strap at the end of the CFRP sheet. The remaining three beams had three U-straps spaced 180 mm (7.09 inches) apart; two of the beams had prestressed U-straps, and the third beam had non-prestressed U-straps. The beams were tested in three-point bending with a clear span of 1,600 mm (63 inches). Improvement in beam strength for the four beams with one U-strap ranged from 15% to 44%, while beams with multiple U-straps per shear span increased the ultimate capacity up to 79% more than beams with just one U-strap. The authors concluded that use of U-wraps effectively limits the debonding of externally bonded FRP. In addition, they found that placing multiple U-straps within the shear span limited the debonding because the openings of flexure-shear cracks were restricted. Study results proved that prestressed U-straps performed only slightly better than non-prestressed U-straps.

Yalim, Kalayci, and Mirmiran (2008) performed an experimental study that examined the performance of FRP-strengthened reinforced concrete beams in flexure based on the amount of surface preparation and U-wrap anchorage. The overall study consisted of 26 specimens with two CFRP systems, wet layup and precured, as well as three different levels of surface preparation. The surface preparation was classified based on the roughness of the surface. These beams also had unique U-wrap layouts, including no U-wraps, 4 U-wraps, 7 U-wraps, 11 U-wraps, and a full-length U-wrap. The flexural beams were T-beams with a web width of 152 mm (6 inches) and a web depth of 305 mm (12 inches), and the flange was 305 mm (12 inches) wide and 76 mm (3 inches) thick. Tension steel reinforcement was two No. 16M (#5) bars, and compression steel reinforcement was two No. 10M (#3) bars. The beams were 2.1 meters (82.68 inches) long with a clear span of 2 meters (78.74 inches). All the beams were tested in three-point bending. Study results showed that the amount of surface roughness did not significantly affect beam performance regardless of the FRP system, whether or not U-wrap anchorage was used, or if the failure load of the beam was debonding or FRP rupture. Although the ultimate load of the beam increased as the amount of anchorage increased, the most significant difference between the beams was ductility, which was greatly affected as the anchorage increased. Study results also showed that decreased amounts of anchorage resulted in FRP debonding as the failure mode. For beams with four and seven straps, FRP debonding was the failure mode after the straps themselves ruptured; for beams with 11 straps and a full-length continuous strap, FRP rupture was reached as the failure mode.

Rasheed, Decker, Esmaeily, Peterman, and Melhem (2015) studied the impact of CFRP U-wraps on the flexural capacity of concrete beams. Six beams were constructed for this experiment, and the beams were divided into two series. The first series consisted of three beams with rectangular cross sections, while the second series contained the remaining three beams, that had a T-shaped cross section. Each beam measured 4,877 mm (192 inches) in length with a clear span of 4,724 mm (186 inches). The cross sections of the rectangular beams measured 152 mm (6 inches) by 305 mm (12 inches). The tension steel consisted of two (No. 5) bars that were each 16 mm in diameter, while the compression steel was comprised of two (No. 3) bars that were each 10 mm in diameter. The T-beams, which had web dimensions identical to the rectangular beams, also had flange dimensions of 406 mm (16 inches) wide by 102 mm (4 inches) thick. The rebar was also the same diameter and number as the rectangular beams, but instead of two compression bars, the rebar contained four compression bars. For each series of beams, the first beam was a control beam, the second was strengthened with five layers of CFRP, and the third beam was strengthened with five layers of CFRP and additional transverse CFRP U-wraps. All the beams were tested in four-point bending with initial load control until steel yielding and then displacement control throughout the remainder of the test. The T-beam with U-wraps showed a strength increase of approximately 130% compared to the strengthened T-beam with no U-wraps and a strength increase of approximately 210% compared to the control beam. For the rectangular beams, the beam with U-wraps showed a strength increase of approximately 110% compared to the strengthened beam with no U-wraps and a strength increase of approximately 220% compared to the control beam. Study results proved that the use of U-wraps as an anchorage system effectively increases the flexural strengthening level of a beam. In addition, the study proposed a design method to determine the amount of U-wrap anchorage needed based on an adapted shear friction model.

Chapter 3: Design and Construction of Specimens

3.1 Design of T-Beams

The T-beam design for this study was based on the requirements of ACI 318-14 (ACI Committee 318, 2014) and ACI 440.2R-17 (ACI Committee 440, 2017), which are themselves based on the principles of strain compatibility and force/moment equilibrium. A set of design criteria was used to evaluate each beam design. The first criterion was that the beams had to have a minimum web depth of 18 inches and a minimum web width of 10 inches to mimic a bridge-scale beam. The second criterion was that, if the FRP reached rupture strain, the strengthened beams reached a failure load less than the capacity of the actuator used for testing. This study utilized a 150-kip capacity actuator. The third criterion for design was that the control beam had to have a failure load less than the FRP debonding and FRP rupture load. The fourth criterion required concrete crushing failure to occur after FRP rupture failure for the strengthened beams to fully utilize the FRP material. The final evaluation criterion was that all the beams must fail in flexure.

This study utilized an Excel spreadsheet to efficiently accommodate different design inputs, while recalculating to show failure loads for the various failure modes. To simplify the calculations in the spreadsheet, the compression steel was not included in the calculation of the flexural strength of the beam because the contribution from this steel typically is negligible. Figure 3.1 shows the Excel spreadsheet used for design.

Fiber Properties			Steel Properties			Concrete Properties			Section Properties		
ffu	462	ksi	Es	29000	ksi	f'c	5	ksi	Flange Width	20	in
efu	0.014		fy	70	ksi				Flange Depth	4	in
Ef	33000	ksi							Web Width	10	in
tf	0.013	in							Web Depth	14	in
									Total Depth	18	in

Reinforcement			Diameter	Number	Area	Inputs=
Steel	#6		0.75	4	0.44	
Stirrups	#3		0.375		0.11	
Cover	1	in				
Fiber Layers	1					
Fiber Wrapping	0	Wrapped (Y=2, N=0)		2		

d	16.25	in
As	1.76	in^2
df1	18	
df2	18	in
Af1	0.13	in^2

Shear Stirrup Design				
Vu=	46	kips	4phisqrt(f'c)bd	34.47145558
phiVc	17.236	kips		
Sreq'd	6.525	in	Use 32 stirrups @ 6" C-C	

Goal seek for c by setting the equilibrium cell to 0 by changing the cell with c

efd= 0.009

Rupture Calcuation			
c=	2.6112		
Beta 1	0.7659		
ebi	0		
efe1	0.014	ecf	0.002376
		e'c	0.002121
ffe1	462	alpha	0.701826
Force Equilibirum			
af'cbc	183.26		
Asfy	123.2		
Affe1	60.06		
Equilibirum	1E-05		
es	0.0124		
Mn	2899.8	kip*in	
Mn	241.65	kip*ft	
Pn	**60.41**	kips	

Concrete Crushing Calcuation			
c=	2.6586		
Beta 1	0.8217		
ebi	0		
efe1	0.0177	ecf	0.003067278
		e'c	0.00212132
ffe1	584.1	alpha	0.749025434
Force Equilibirum			
af'cbc	199.13		
Asfy	123.2		
Affe1	75.933		
Equilibirum	-3E-04		
es	0.0157		
Mn	3151.3	kip*in	
Mn	262.61	kip*ft	
Pn	**65.65**	kips	

Debonding Calcuation			
c=	2.8027		
Beta 1	0.7251		
ebi	0		
efe1	0.009	ecf	0.001653
		e'c	0.002121
ffe1	295.7	alpha	0.576726
Force Equilibirum			
af'cbc	161.64		
Asfy	123.2		
Affe1	38.441		
Affe2	0		
Equilibiru	-1E-06		
es	0.0079		
Mn	2529.7	kip*in	
Mn	210.81	kip*ft	
Pn	**52.7**	kips	

Control Beam Calculation		
c=	1.8118	
Beta 1	0.8	
ebi	0	
efe1	0	
efe2	0	
ffe1	0	
ffe2	0	
Force Equilibirum		
0.85B1f'c	123.2	
Asfy	123.2	
Affe1	0	
Affe2	0	
Equilibiru	1E-14	
es	0.0239	
Mn	1912.7	kip*in
Mn	159.39	kip*ft
Pn	**39.85**	kips

Figure 3.1: Spreadsheet for T-Beam Design

3.2 T-Beam Geometry

The final beam design included a T-beam that measured 16 ft long with a clear span of 15.5 ft. The beams had a web depth of 18 inches and a web width of 10 inches, and the flanges of the beams were 4 inches thick with widths of 20 inches. The rebar for each beam consisted of three No. 6 bars for tension reinforcement and four No. 3 bars for compression reinforcement. Stirrups were placed throughout the beam to ensure the beam failed in flexure and not shear. The minimum spacing required between the stirrups was 6.5 inches for a No. 3 stirrup. For the final design of shear stirrups, No. 3 bars were spaced 6 inches on center. Figure 3.2 shows a cross section of the final beam design, and Figure 3.3 shows a side view of the beam with stirrups and support locations.

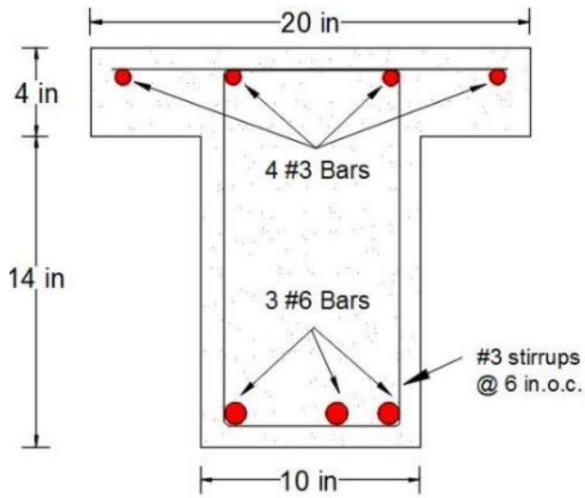

Figure 3.2: Cross Section of Final Beam Design

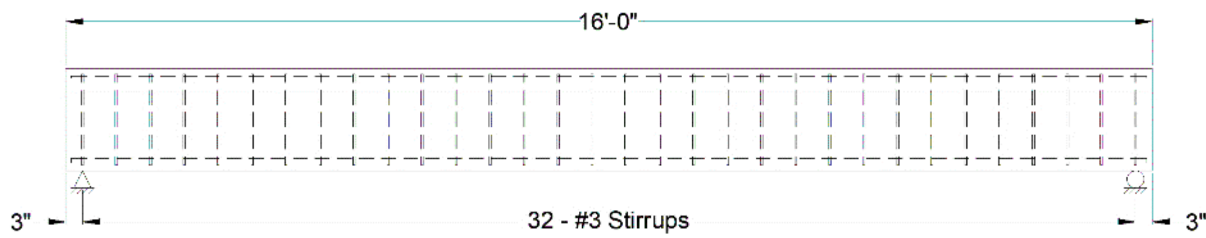

Figure 3.3: Shear Reinforcement and Support Locations for Final Beam Design

3.3 Strengthening Design for T-Beams

3.3.1 CFRP Sheets

Unidirectional high modulus CFRP sheets (V-Wrap C200HM) were used to strengthen the designed T-beams. The final strengthening design included one layer of CFRP with a width of 10 inches along the bottom face of every beam in this study. The CFRP sheet was 15 ft long, and each end of the sheet was placed 3 inches from the support location. Figure 3.4 shows Beam 2, which was the strengthened beam without an anchorage system.

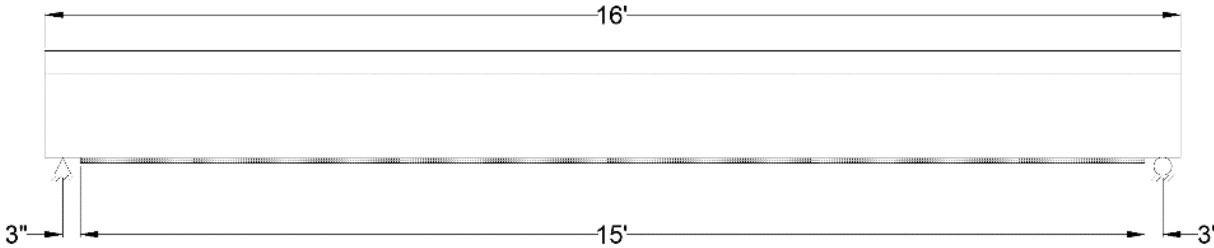

Figure 3.4: Profile of Strengthened Beam with CFRP Sheet Only

3.3.2 Anchorage with CFRP Splay Anchors

CFRP splay anchors for these beams had diameters of 0.625 inches. The process used to obtain a preliminary design for the number of anchors needed per shear span, as outlined by Zaki (2018), included calculating the maximum tension force in the externally bonded FRP sheet and then determining the maximum shear capacity of each anchor. The number of anchors was then determined by dividing the total tension force in the FRP by the shear capacity of each anchor. The following equations were used to complete this design:

$$\boldsymbol{V_{anchor} = \frac{T_{max}}{\#\ of\ anchors\ per\ shear\ span}} \qquad \textbf{Equation 3.1}$$

$$\boldsymbol{\gamma_{anchor} = (3.5°) \times (\frac{\pi}{180})} \qquad \textbf{Equation 3.2}$$

$$\boldsymbol{\tau_{anchor} = G_{12} \times \gamma_{anchor}} \qquad \textbf{Equation 3.3}$$

$$\boldsymbol{V_{anchor} = \tau_{anchor} \times A_{anchor}} \qquad \textbf{Equation 3.4}$$

In addition to the preliminary number of anchors required, the length of each anchor was calculated. Each anchor had an embedment depth of 4 inches, and the splay length was two-thirds of the length of the space between anchors. Figure 3.5 shows the preliminary design calculations.

Number of Anchors Required Calculation		
Ef =	14240	ksi
tf =	0.04	in
efu =	0.0127	
bf =	10	in
Tmax = Ef*tf*bf*efu =	72.3392	kips
γ anchor = (3.5 degrees)*(PI/180) =	0.061087	radians
G12 =	700	kips/in^2
τ anchor = G12* γ anchor =	42.76057	kips/in^2
ϕ =	0.85	
A anchor = (PI()/4)*((5/8)^2) =	0.306796	in^2
V anchor = ϕ*τ anchor*A anchor	11.15096	kips
Minimum Anchors per Shear Span	6.487262	anchors
Use 6 anchors per shear span		
Spacing from Support to Midspan =	93	in
Spacing from Support to First Anchor =	5	in
Spacing from Midspan to Anchor =	18	in
Spacing Between Anchors =	14	in
Length of Each Anchor =	13.38	in
Use anchors that are 14.00" long		

Figure 3.5: CFRP Splay Anchor Design

After performing initial design calculations, six anchors were determined to be needed per shear span to achieve CFRP rupture failure mode. Accordingly, the CFRP sheet for Beam 3 was secured with six anchors per shear span. The goal of the study was to adjust the number of anchors per shear span to determine the accuracy of the proposed design model using less anchors per shear span for the other beams. Therefore, using the same anchor length, Beam 4 had four anchors per shear span, Beam 5 had five anchors per shear span, and Beam 6 had five anchors per shear span but shorter anchor lengths (10 inches each). Figure 3.6 and Figure 3.7 show the anchor layouts for Beam 5 and Beam 6.

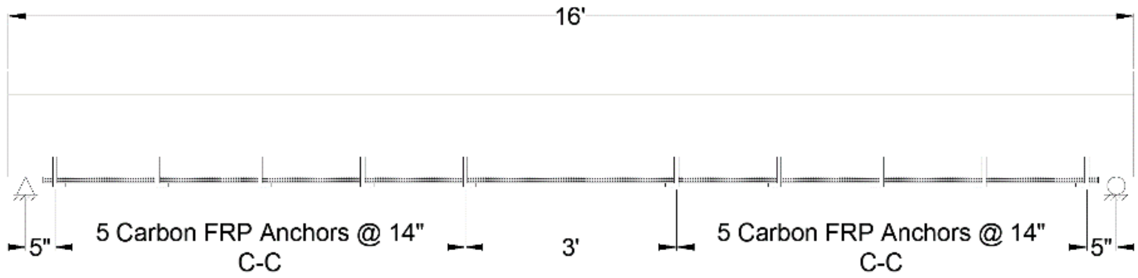

Figure 3.6: Profile of Strengthened Beam with CFRP Sheet and CFRP Splay Anchors

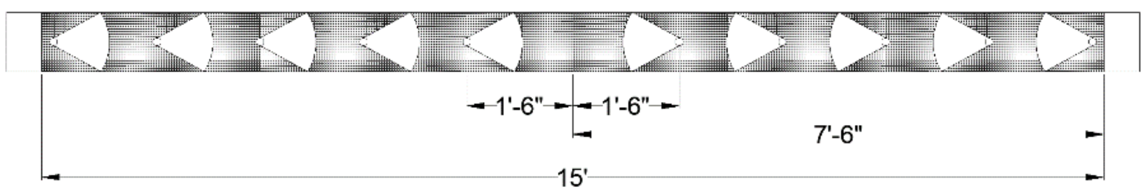

Figure 3.7: Bottom of Beam with CFRP Sheet and Splay Anchors

3.4 Formwork and Steel Caging

The formwork used to cast the beams was constructed from plywood sheets and lumber planks measuring 2 inches by 6 inches. The plywood used for the formwork was 0.75 inches thick and measured 4 ft wide and 8 ft long. The formwork was constructed in two sections that were each 8 ft long and then attached to create the 16-ft length needed for the beams. The formwork sections were constructed in the civil engineering woodshop at Kansas State University and then transported to the Civil Infrastructure Systems Laboratory (CISL) to be assembled into the larger sections. Because each set of formwork consisted of two beams, three total sets of formwork were constructed to cast six total beams. In the endcap of each set of formwork, a hole was predrilled at the centroid of the beam section so a piece of rebar could be held in place during casting. This rebar was used to flip the beams after casting and curing. Figure 3.8 and Figure 3.9 show the assembled formwork at CISL.


Figure 3.8: Two Sets of Assembled Formwork


Figure 3.9: Full-Length Formwork

The rebars used for the longitudinal steel were originally 20 ft long and then cut down using a steel cut-off wheel. The stirrups used for the caging were pre-bent and attached to the longitudinal steel using rebar ties. In addition, transverse bars were placed at each stirrup to tie in the compression bars outside the stirrups in the flange. These bars were also cut down using a steel cut-off wheel and tied to the stirrups and longitudinal bars using rebar ties. Figure 3.10 shows a section of finished rebar caging, and Figure 3.11 shows the finished rebar cages. Two strain gauges were attached at the midspan of each rebar cage, with one on each outer tension reinforcement bar. In order to protect the strain gauges during casting, the strain gauge was taped over and the wires were run out of the beam along the tension reinforcement and then up a stirrup, with various points taped to the rebar. One-inch steel chairs were used to raise the cage off the bottom of the formwork to create the desired clear cover. After the cages were placed in the formwork, a No. 6 bar was placed through a predrilled hole in the end caps of the formwork so the beams could be flipped after casting. Figure 3.12 shows the rebar caging in the finished formwork before casting. Rebar hooks were cut, bent, and placed in the rebar caging to make the beams easier to move after curing. Each rebar hook consisted of two No. 3 bars tied together. Four total hooks were used for each beam, with two hooks located approximately 10 inches from the midspan of the beam and two hooks located at the end of the rebar caging.

Figure 3.10: Section of Finished Rebar Caging

Figure 3.11: Finished Rebar Caging for All Beams

Figure 3.12: Rebar Caging in Formwork before Casting

3.5 Casting of Beams

The beams were cast using 4,500 psi ready-mix concrete provided by Midwest Concrete Materials, a local provider. Based on the amount of concrete needed to cast all six beams, the concrete was delivered in one batch to guarantee consistent mechanical properties of the mix. No additives were applied to the mix, but the mix had a slump of 4 inches. Several graduate students helped cast the beams by vibrating the concrete, directing the truck, and screeding the top of the beams. In addition to the beams, 12 cylinders were poured for the ready-mix batch. The beams were covered with tarps since the temperature at the time of casting was mild. The outside of all the formwork was removed one week after casting. Figure 3.13 through Figure 3.15 are photographs of the casting process. Figure 3.16 shows the beams once the outside of the formwork was removed.

Figure 3.13: Casting of Beams

Figure 3.14: Placing and Vibrating Concrete in the Formwork

Figure 3.15: Screeding the Tops of the Beams

Figure 3.16: Finished Beams with Outside of Formwork Removed

3.6 Surface Preparation

Prior to the installation of any FRP, the surfaces of each beam required surface preparation. However, before any surface preparation occurred, the beams were moved inside the CISL and flipped using the previously inserted rebar, as shown in Figure 3.17. A masonry grinding wheel was used to slightly roughen the bottom surface of the beam web of each strengthened beam to expose small air pockets and aggregate in the concrete so the epoxy could fill the gaps and grip the surface (Figure 3.18). In addition to surface grinding, holes measuring 4 inches deep and 0.75 inches in diameter were drilled into the bottom of the beam at each anchor, as shown in Figure 3.19. Figure 3.20 shows a beam with paper towels temporarily inserted into the holes to avoid epoxy fillings while the sheet is initially laid down, and Figure 3.21 shows the anchor holes in the beam.

Figure 3.17: Flipping Beam for Surface Preparation

Figure 3.18: Preparing Beam Surfaces with Masonry Wheel

Figure 3.19: Drilled Holes in the Beam to Install CFRP Splay Anchors

Figure 3.20: Filling Drilled Holes with Paper Towels Prior to Installing CFRP Splay Anchors

Figure 3.21: Four Anchor Holes Drilled for Beam 4

3.7 FRP Installation

After the surface preparation was complete, the FRP was installed onto the strengthened beams. FRP installation was performed inside the CISL so that moisture or temperature did not affect the resin and FRP. Compared to Beam 2, the installation process for Beams 3, 4, 5, and 6 was slightly more involved due to the installation of fiber anchors. The two processes are outlined in this section.

The first step in the installation process for Beam 2 was to mix the resin according to manufacturer specifications. Two batches of resin were mixed; the first batch was the regular resin, and the second batch was a thickened resin consisting of silica fume added to the regular resin according to manufacturer specifications. In the second step of installation, a layer of regular resin was applied to the surface of each beam, and a layer of the putty or thickened resin was placed on the beam in the third step. In the fourth step of installation, the CFRP sheet was saturated separately on top of plastic sheets, and then the saturated CFRP was rolled onto a PVC pipe and spread on the beam soffit. Finally, a ribbed roller was used on the CFRP sheet to remove all air pockets.

Following placement of the CFRP, the next step for Beam 3 through Beam 6 was to install the CFRP splay anchors. The first step for installation was to remove the paper towels used to prevent the holes from filling with epoxy. The second step was to fill approximately half of each anchor hole with the thickened resin, and then the end of each anchor was inserted into the holes using a metal rod to ensure each anchor reached the bottom of the hole. Finally, the ends of the splay anchors outside each beam were splayed along the bottom of the beam towards the centerline and rolled with resin to saturate each end. Once the beams were strengthened, they were left to cure for approximately seven days to ensure the resin had adequate time to cure before testing. Figure 3.22 through Figure 3.26 illustrate the steps of the FRP installation process used for Beam 2. Figure 3.27 through Figure 3.32 show the FRP installation process for installing anchors in Beam 3 through Beam 6.

Figure 3.22: Mixing Regular Resin

Figure 3.23: Applying Primer Resin and Putty to Beam Surface

Figure 3.24: Saturating the CFRP Sheet

Figure 3.25: Rolling Saturated FRP Sheet on the Beam

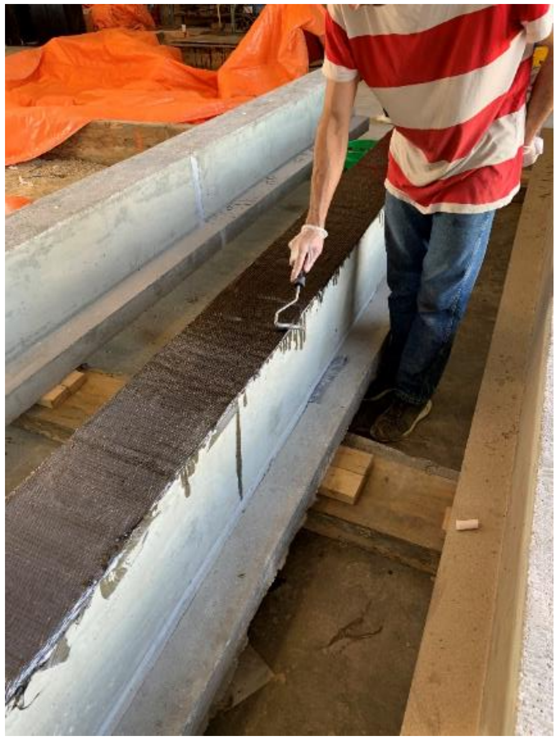

Figure 3.26: Forcing Out Air Pockets Using a Ribbed Roller

Figure 3.27: Inserting Thickened Resin into Anchor Holes

Figure 3.28: CFRP Splay Anchors Installed in Anchor Holes

Figure 3.29: Splaying CFRP Anchors with Resin

Figure 3.30: Finishing the Anchor Splay for Beam 4

Figure 3.31: Splayed Short Anchors of Beam 6

Figure 3.32: Long and Short CFRP Anchors of Beams 5 and 6

Chapter 4: Material Properties

4.1 Testing of Concrete Cylinders

For casting the beams, a ready-mix concrete was used with a requested nominal compressive strength of 4,500 psi. The concrete was delivered in one batch to make pouring and working with the concrete more consistent. On May 1, 2019, 12 cylinders measuring 4 inches by 8 inches were cast to test the experimental compressive strength of the concrete. All of the cylinders were cured in a moisture room at Kansas State University for 28 days, and all the cylinders were tested exactly 28 days from the date of the pour. The average compressive strength of the cylinders was 5,407.42 psi, with a standard deviation of 224.61 psi. Table 4.1 shows the results of the cylinder tests. F`igure 4.1 through Figure 4.3 show the concrete cylinder testing.

Table 4.1: Results from Concrete Cylinder Testing

No.	Weight (lb)	Density (lb/ft^3)	Max Load (lb)	Max Stress (psi)
1	8.19	140.7757303	70,987	5,649
2	8.19	140.7757303	69,973	5,568
3	8.19	140.7757303	67,803	5,396
4	8.22	141.2913923	69,174	5,505
5	8.23	141.4632796	70,299	5,594
6	8.17	140.4319556	66,127	5,262
7	8.22	141.2913923	65,971	5,235
8	8.18	140.6038429	71,074	5,656
9	8.21	141.1195049	68,850	5,479
10	8.15	140.0881809	66,190	5,267
11	8.18	140.6038429	61,204	4,870
12	8.17	140.4319556	67,964	5,408
	Ave=	**140.8043782**	**Ave=**	**5,407.416667**
			St Dev=	**224.6089362**

Figure 4.1: Concrete Cylinders Prior to Testing

Figure 4.2: Concrete Cylinder in Testing Apparatus

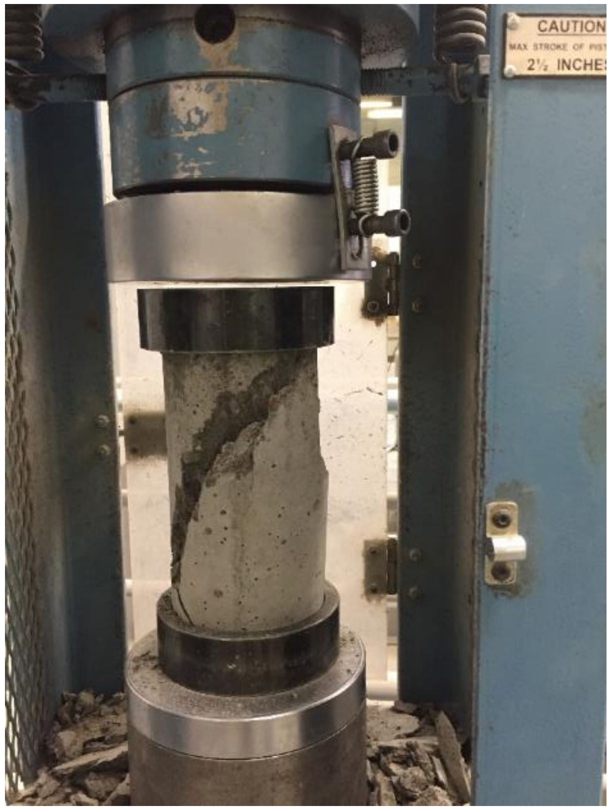

Figure 4.3: Concrete Cylinder Failure

4.2 Testing of Steel Rebar

As mentioned, the steel rebar used in the beam construction consisted of No. 6 bars for the tension reinforcement and No. 3 bars for the compression reinforcement and stirrups. The rebar had manufacturer-given properties of 60 ksi for minimum yield strength and 29,000 ksi for the modulus. Samples of No. 6 bars were tested at a new research laboratory at Kansas State University and samples of No. 3 bars were tested at the KDOT Materials and Research Center to verify average yield strength and ultimate strength of the rebar. Three samples of each bar size were tested to obtain average yield strength for each size. The average experimental yield strength of the No. 6 bars was 82 ksi, and the average experimental ultimate strength of the No. 6 bars was 100.88 ksi. The average experimental secant steel modulus of the No. 6 bars was 25,145 ksi, while the average experimental yield strength of the No. 3 bars was 66.90 ksi. The average experimental ultimate strength of the No. 3 bars was 101.63 ksi, and the average experimental secant steel modulus of the No. 3 bars was 29,261 ksi. Figure 4.4 illustrates the No. 6 bar testing at Kansas

State University, Figure 4.5 shows the rebar testing machine at the KDOT Materials and Research Center, and Figure 4.6 shows a graph of the average stress-strain curve of the No. 6 bars based on two-strain gauge results.

Figure 4.4: Testing No. 6 Steel Rebar at Kansas State University

Figure 4.5: Apparatus for Steel Rebar Testing

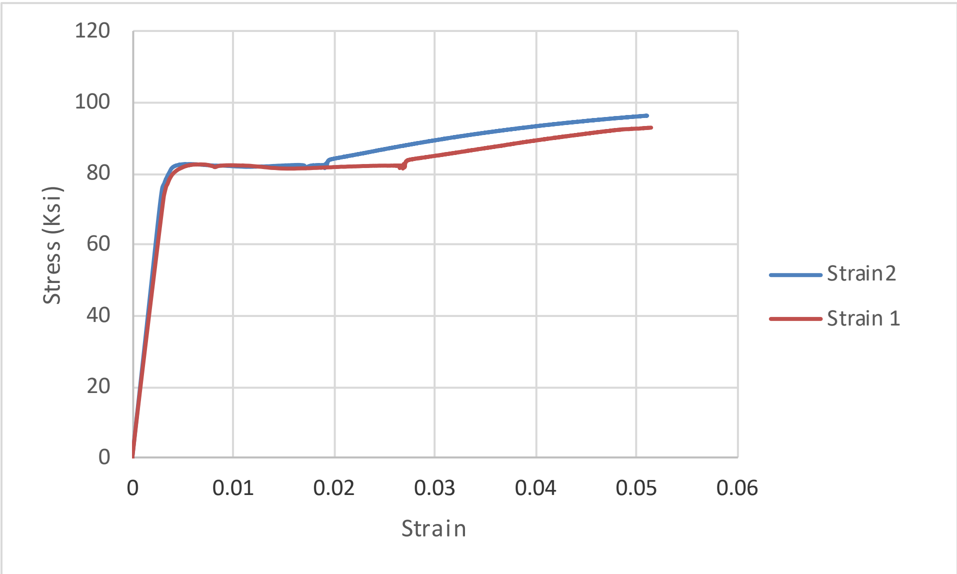

Figure 4.6: Average Stress-Strain Curve of No. 6 Steel Rebar

4.3 FRP Properties

This study used a relatively new type of CFRP developed by Structural Technologies: V-Wrap C200HM, a high modulus CFRP that was used as the externally bonded flexural strengthening material for the beams. Figure 4.7 shows the manufacturer-given properties for the CFRP. The CFRP fiber anchors used in this study, which were also manufactured by Structural Technologies, were made of high modulus carbon fibers with diameters of 0.625 inches. Figure 4.8 shows the manufacturer-given properties for the carbon fiber used for the anchors.

Strengthening Solutions

V-Wrap™ C200HM

High Modulus Carbon Fiber Fabric

struc'tur'al TECHNOLOGIES

structuraltechnologies.com

+1-410-859-6539

Typical Data for V-Wrap C200HM		
Storage Conditions:	Store dry at 40°F – 90°F (4°C – 32°C)	
Color:	Black	
Primary Fiber Direction:	0° (unidirectional)	
Weight:	17.7 oz/yd^2 (600 g/m^2)	
Shelf life:	10 years	
Fiber Properties (Dry)		
Tensile Strength:	790,000 psi (5,440 MPa)	
Tensile Modulus:	42 x 10^6 psi (289,550 MPa)	
Elongation:	1.9 %	
Cured Laminate Properties	**Average Values**	**Design Values***
Tensile Strength:	180,000 psi (1,241 MPa)	155,000 psi (1,068 MPa)
Modulus of Elasticity:	14.24 x 10^6 psi (98,181 MPa)	14.0 x 10^6 psi (96,527 MPa)
Elongation at Break:	1.27%	1.1%
Thickness:	0.04 in. (1.02 mm)	0.04 in. (1.02 mm)
Strength per Unit Width:	7,200 lbs/in. (1.26 kN/mm)	6,200 lbs/in. (1.09 kN/mm)

*Design properties are based on ACI 440.2R using average minus three standard deviations.

Figure 4.7: CFRP Manufacturer Properties

Strengthening Solutions

V-Wrap HM Carbon Fiber Anchors

structural TECHNOLOGIES
structuraltechnologies.com
+1-410-859-6539

Typical Data for V-Wrap HM Carbon Fiber Anchors

Storage Conditions:	Store dry at 40°F – 90°F (4°C to 32°C)
Color:	Black
Shelf life:	10 years

Fiber Properties (Dry)

Tensile Strength:	790,000 psi (5,440 MPa)
Tensile Modulus:	42 x 10^6 psi (289,550 MPa)
Elongation:	1.9%

Cured Laminate Properties	**Design Value**
Tensile Strength:	165,000 psi (1,138 MPa)
Modulus of Elasticity:	15.0 x 10^6 psi (103,420 MPa)
Elongation at Break:	1.1%

Anchor Sizes Anchors are available in diameters ranging from 0.375" to 1.5" (9 mm to 37 mm) in 1/8" increments.

Figure 4.8: High Modulus Carbon Fiber Anchor Manufacturer Properties

Chapter 5: Experimental Setup and Testing

5.1 Experimental Setup

Flexural tests were performed at Kansas State University in the structural engineering testing laboratory. The beams were loaded in four-point bending using a 4-ft spreader beam and a 150-kip capacity hydraulic actuator. The actuator was run by a servo-hydraulic system produced by MTS. The system included an accurate data acquisition program and required MTS certification to operate.

For the applied loading, the beams were simply-supported using plates and rollers at each support location, with one support allowing movement in the direction of the beam span and the other support allowing only rotation. The supports were each placed 3 inches from the ends of the beam on center, resulting in a clear span for each beam equal to 15 ft, 6 inches. Figure 5.1 shows a schematic of the experimental setup.

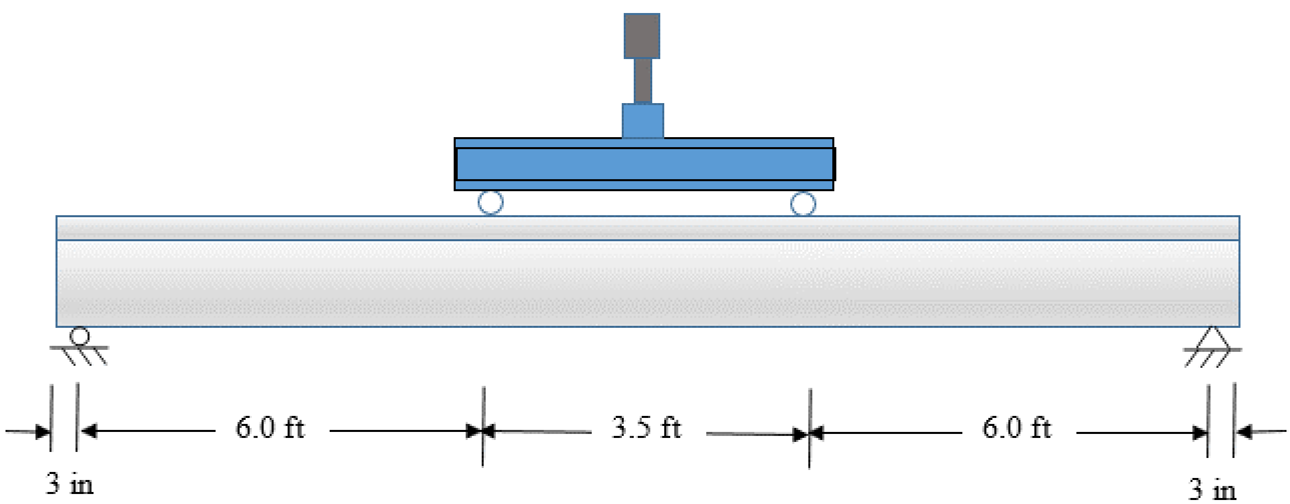

Figure 5.1: Experimental Test Setup

Data that was collected for each beam was the applied load, the deflection at midspan, the strain in the concrete at the top of the beam on either side of the flange, and the strain in the tension steel. In addition, the strain in the FRP at midspan was recorded for the five strengthened beams. Two linear variable differential transducer (LVDT) sensors were used to determine the deflection at the midspan of the beam. Concrete strains were determined using two 120 ohm (Ω) strain gauges, and steel strains were measured using two 120 Ω strain gauges installed on the tension rebar at the midspan of the two outside tension reinforcement bars. FRP strains were measured using 350 Ω strain gauges.

Data from the instrumentation was collected using a data acquisition system called Series 7000, developed by Vishay. The data were recorded every 1.5 seconds during the test, and the beams were loaded using displacement control at a rate of 0.1 inch per minute. When the test was complete, all data points for the load, displacement, and strains were imported into a Microsoft Excel file for analysis. Figure 5.2 and Figure 5.3 show how the concrete and FRP strain gauges were installed on the beams.

Figure 5.2: Concrete Strain Gauge at the Top of the Flange at Midspan

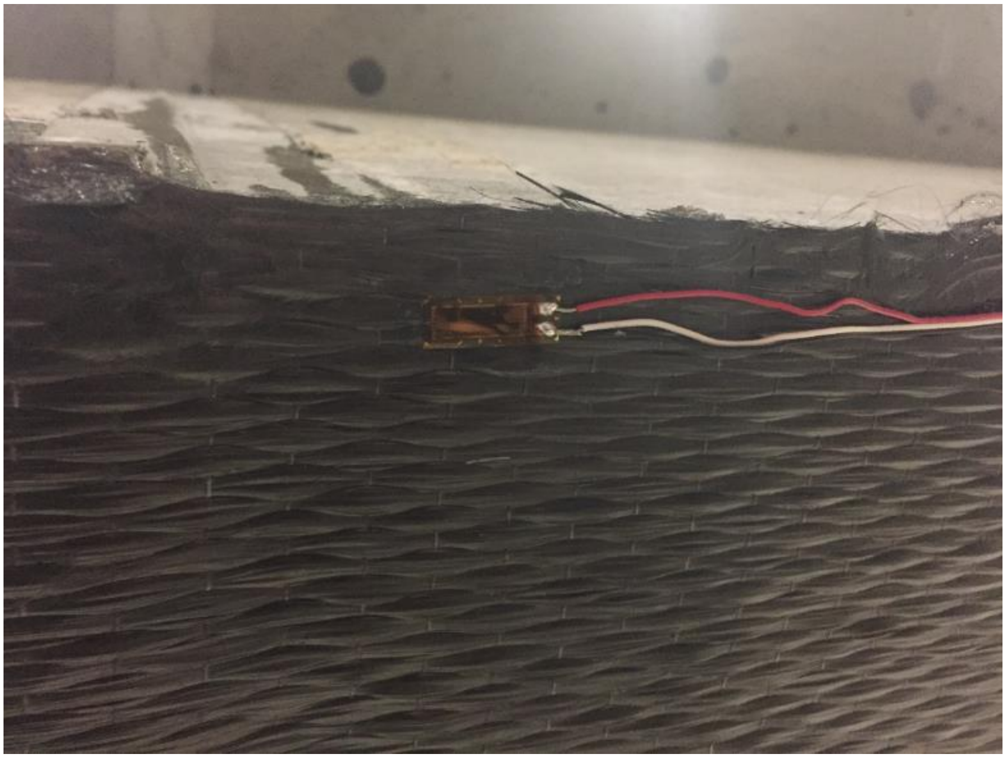

Figure 5.3: FRP Strain Gauge at the Bottom of the Web at Midspan

5.2 Test Results

5.2.1 Control Beam 1

The first beam tested for this experiment was the control beam, Beam 1. Experimental test results showed that the maximum load was 53.73 kips at a deflection of 3.54 inches at the midspan, the maximum concrete compressive strain was 0.0018, and the maximum recorded steel strain was 0.034. Figure 5.4 shows the control beam setup before the test, and Figure 5.5 and Figure 5.6 show the control beam after testing. Figure 5.7 graphs the load versus deflection of the beam, while Figure 5.8 and Figure 5.9 graph the load versus concrete strain and load versus steel strain, respectively.

Figure 5.4: Beam 1 Setup before Testing

Figure 5.5: Beam 1 Crushing Failure at Midspan after Testing

Figure 5.6: Beam 1 after Testing

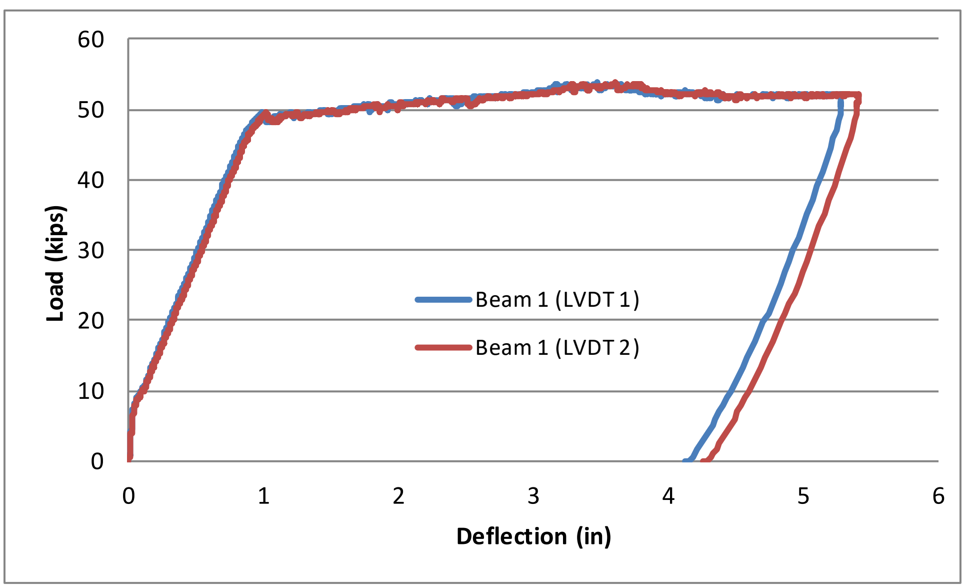

Figure 5.7: Beam 1 Load versus Deflection

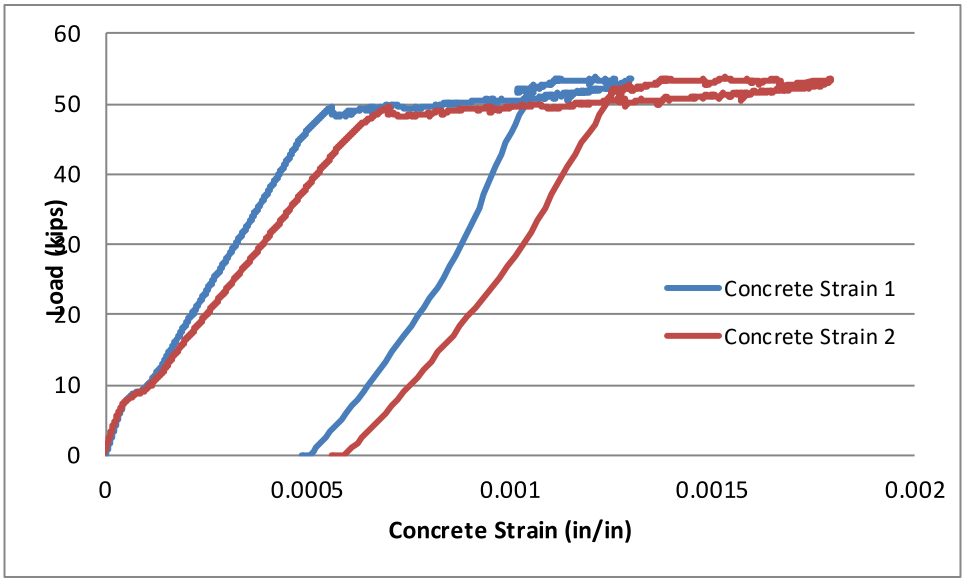

Figure 5.8: Beam 1 Load versus Concrete Top Strain

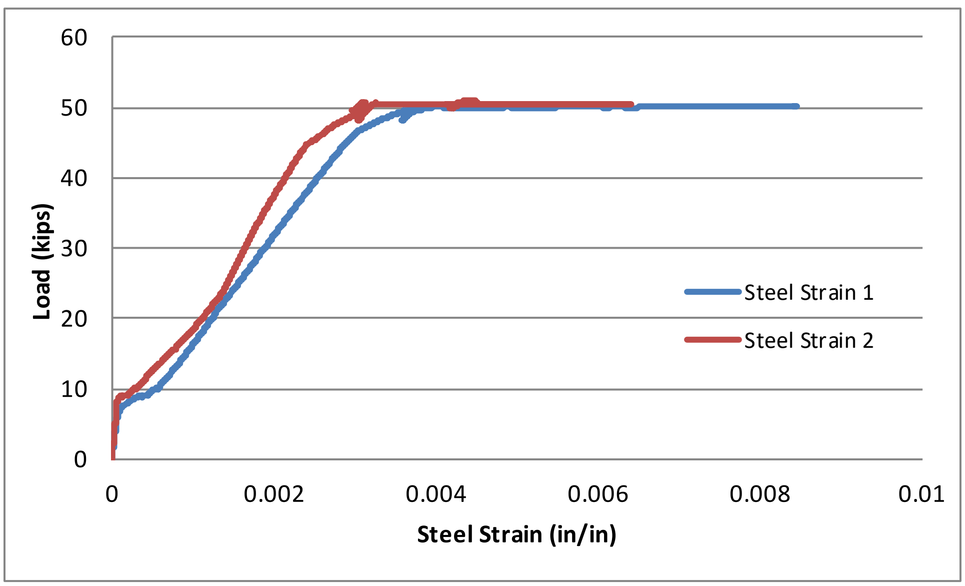

Figure 5.9: Beam 1 Load versus Steel Bar Strain

5.2.2 Beam 2 (with CFRP Flexural Reinforcement)

The second beam tested, Beam 2, was strengthened with one layer of CFRP on the bottom of the web. This beam reached a failure load due to FRP debonding mode, and the CFRP sheet at failure completely debonded from half the beam. Experimental test results showed that the ultimate load at failure was 67.91 kips at a deflection of 1.68 inches at the midspan. The maximum strain in the concrete was 0.00101, and the maximum strain in the steel was 0.00342 when the strain gauge failed. For the FRP strain gauges at the midspan of the beam, one strain gauge reached a maximum strain of 0.009045; the second strain gauge followed a similar trend and had similar values until failure was reached and the strain reached 0.00719 at debonding.

Figure 5.10 shows Beam 2 before the test, and Figure 5.11 and Figure 5.12 show the beam after testing. Figure 5.13 graphs the load versus deflection of the beam, while Figure 5.14 and Figure 5.15 graph the load versus concrete strain and load versus steel strain, respectively. Figure 5.16 shows the load versus FRP strain at the midspan of the beam.

Figure 5.10: Beam 2 Setup before Testing

Figure 5.11: Beam 2 after Failure with Debonded FRP Sheet

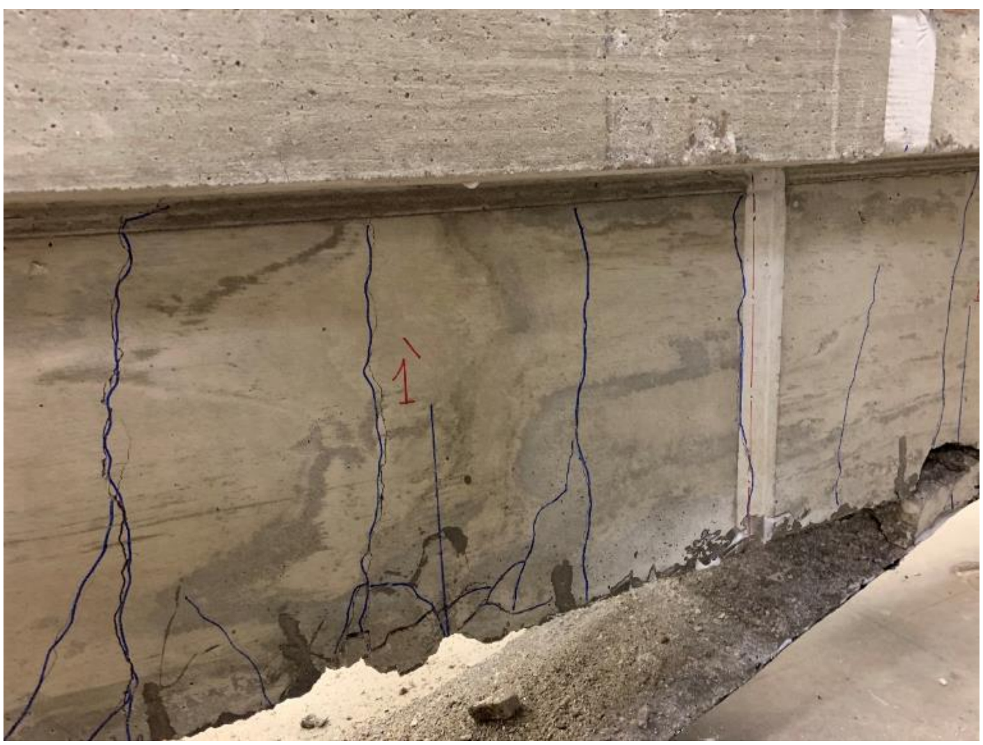

Figure 5.12: Beam 2 FRP Debonding at Failure

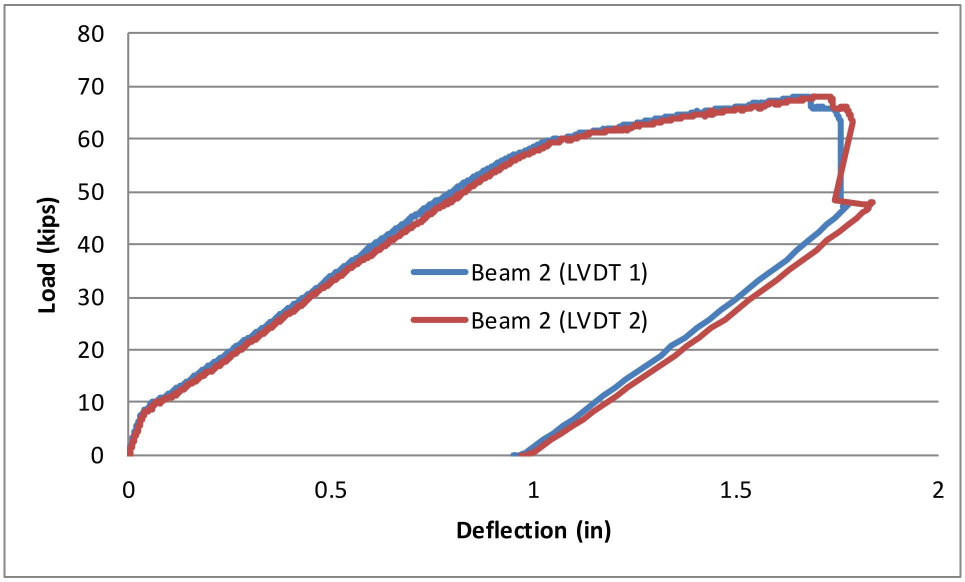

Figure 5.13: Beam 2 Load versus Deflection

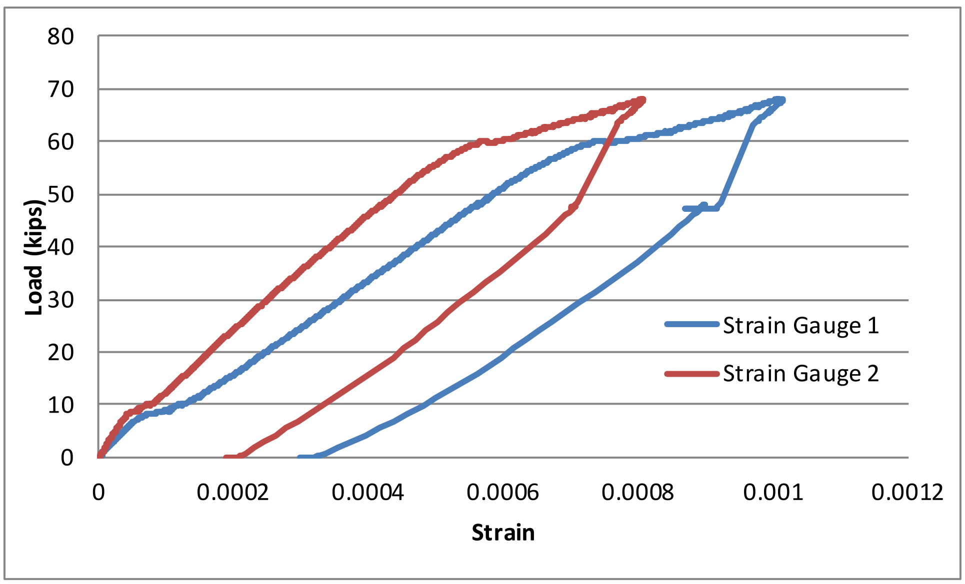

Figure 5.14: Beam 2 Load versus Concrete Top Strain

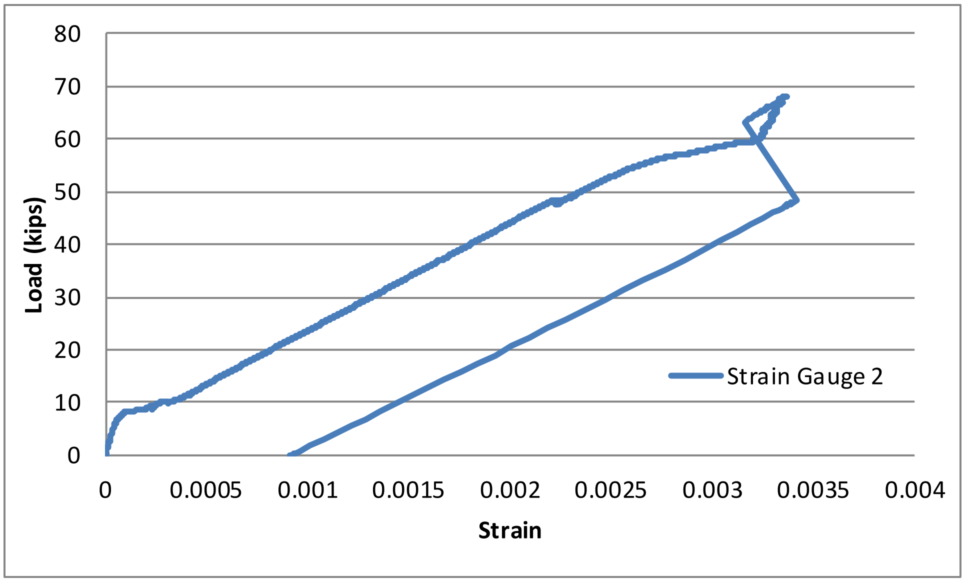

Figure 5.15: Beam 2 Load versus Steel Bar Strain

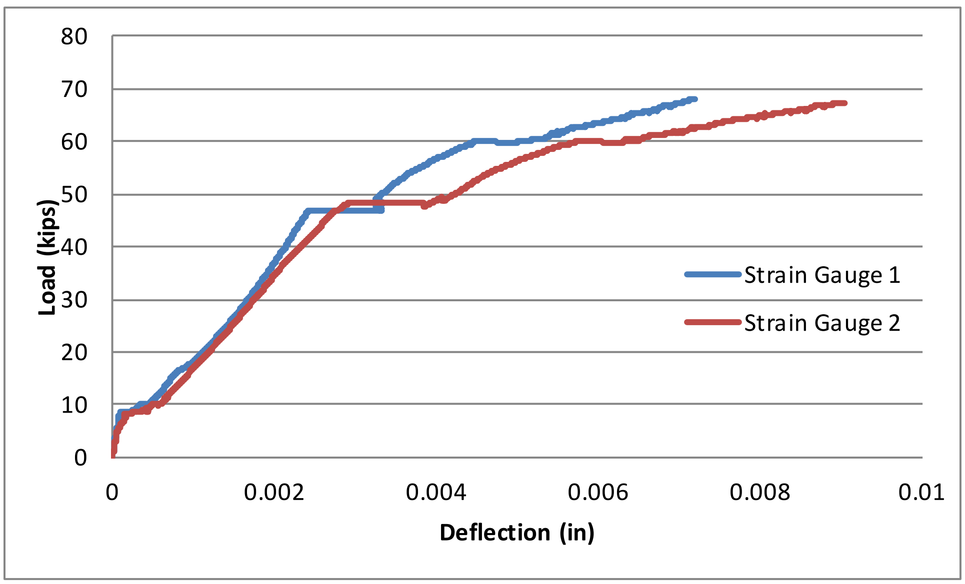

Figure 5.16: Beam 2 Load versus FRP Strain at the Beam Midspan

5.2.3 Beam 3 (with CFRP Flexural Reinforcement and Six CFRP Splay Anchors per Shear Span)

Beam 3 was strengthened with one layer of CFRP on the bottom of the web and anchored with six CFRP splay anchors per shear span. FRP rupture caused the beam to reach a failure load of 79.66 kips. Deflection corresponding to the ultimate load was 2.652 inches at the midspan, while the maximum strain in the concrete was 0.001204 and the maximum strain in the steel was 0.016933. One FRP strain gauge at the midspan of the beam reached a maximum strain of 0.01355, and the second strain gauge reached a maximum strain of 0.005044. The rupture strain of the CFRP was 0.0127.

Figure 5.17 shows Beam 3 before the test, and Figure 5.18 and Figure 5.19 show the beam after testing. Figure 5.20 graphs the load versus deflection of the beam, while Figure 5.21 and Figure 5.22 graph the load versus concrete strain and load versus steel strain, respectively. Figure 5.23 shows the load versus FRP strain for the two strain gauges.

Figure 5.17: Beam 3 Setup before Testing

Figure 5.18: Beam 3 after Testing

Figure 5.19: Beam 3 FRP Rupture at Midspan

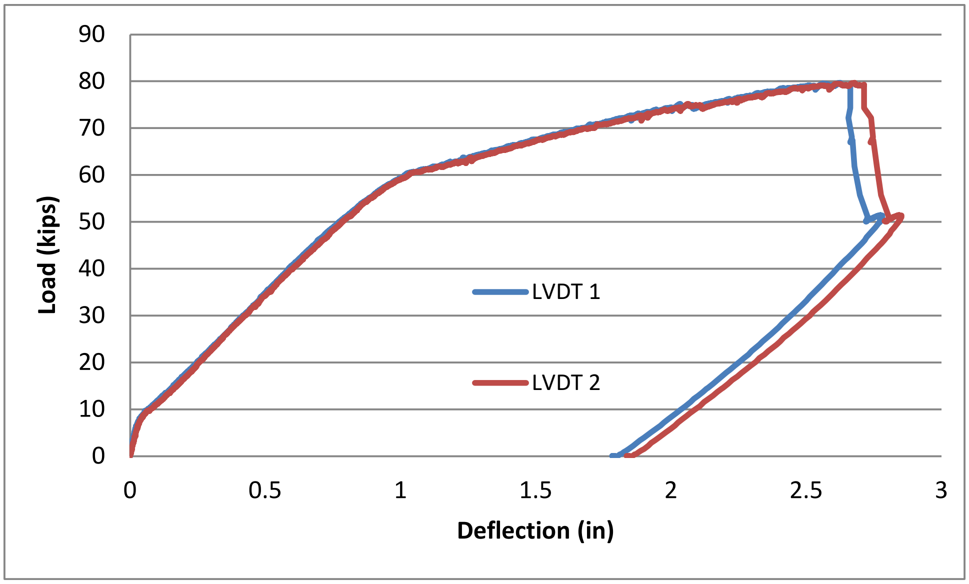

Figure 5.20: Beam 3 Load versus Deflection

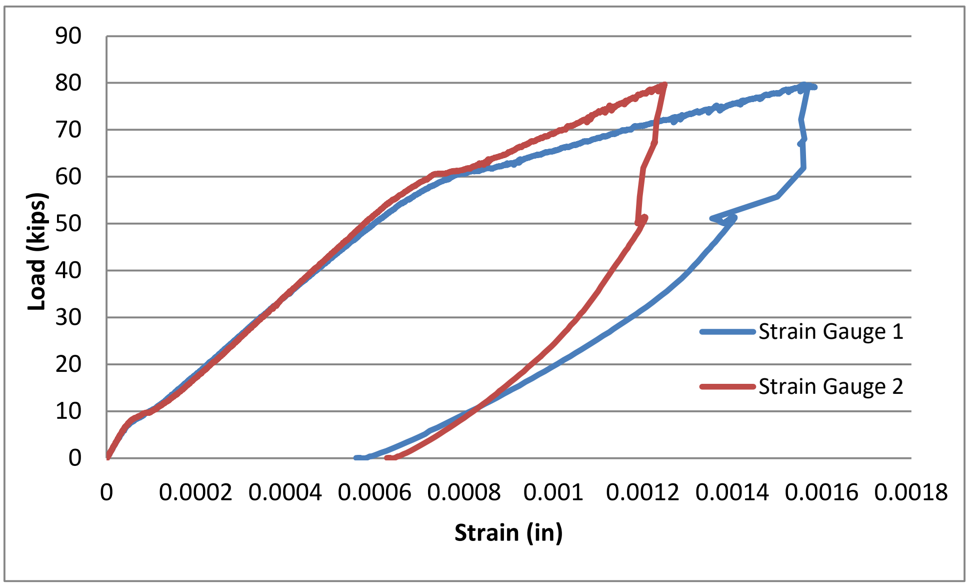

Figure 5.21: Beam 3 Load versus Concrete Top Strain

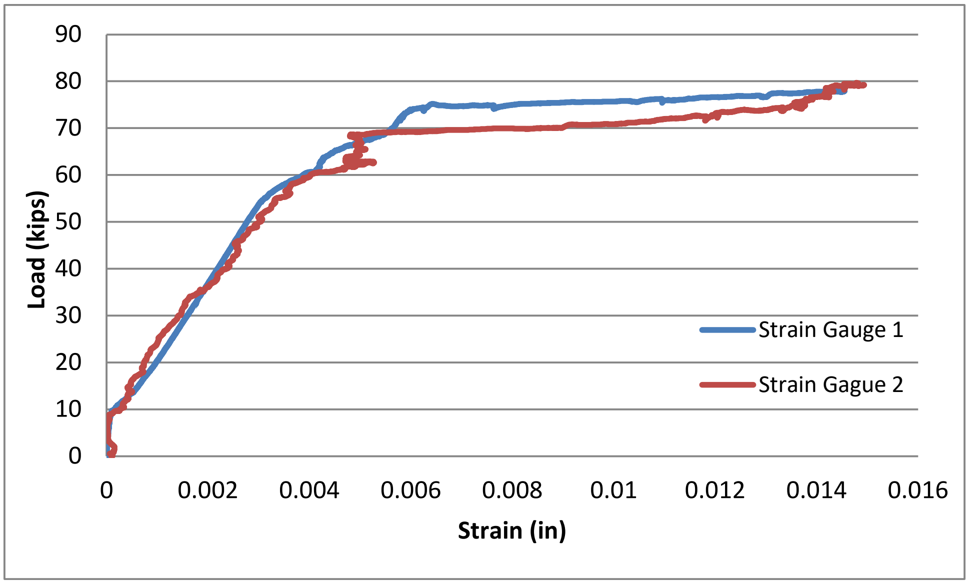

Figure 5.22: Beam 3 Load versus Steel Bar Strain

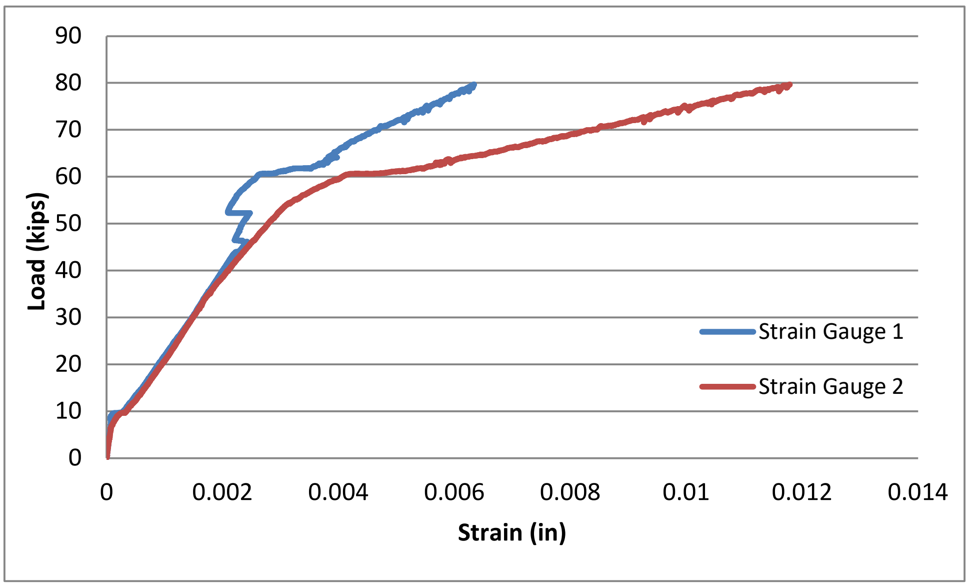

Figure 5.23: Beam 3 Load versus FRP Strain at Midspan

5.2.4 Beam 4 (with CFRP Flexural Reinforcement and Four Splay Anchors per Shear Span)

Beam 4 was strengthened with one layer of CFRP on the bottom of the web and anchored with four anchors per shear span at spacings of 18.75 inches. This beam reached a failure load of 74.60 kips due to FRP debonding of the CFRP sheet while the anchors were in place. The deflection corresponding to the ultimate load was 2.535 inches at the midspan. The maximum strain in the concrete was 0.001623, and the maximum strain in the steel was 0.01284. One FRP strain gauge at the midspan reached a maximum strain of 0.00879, while the second strain gauge reached a maximum strain of 0.00851.

Figure 5.24 shows Beam 4 before the test, and Figure 5.25 and Figure 5.26 show the beam after testing. Figure 5.27 graphs the load versus deflection of the beam, while Figure 5.28 and Figure 5.29 graph the load versus concrete strain and load versus steel strain, respectively. Figure 5.30 shows the load versus FRP strain at the midspan of the beam.

Figure 5.24: Beam 4 Setup before Testing

Figure 5.25: Beam 4 after Testing

Figure 5.26: Beam 4 Full Debonding Held by Anchors

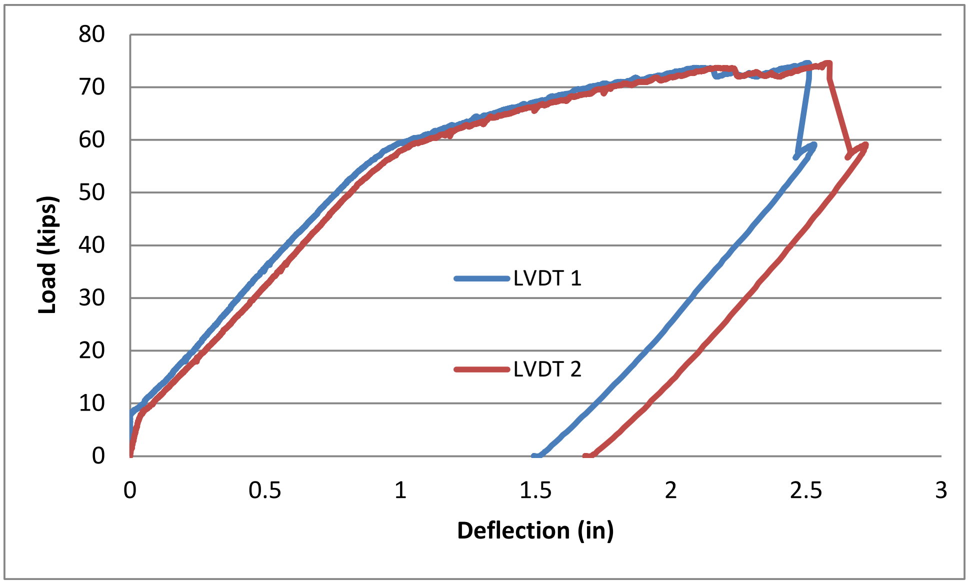

Figure 5.27: Beam 4 Load versus Deflection

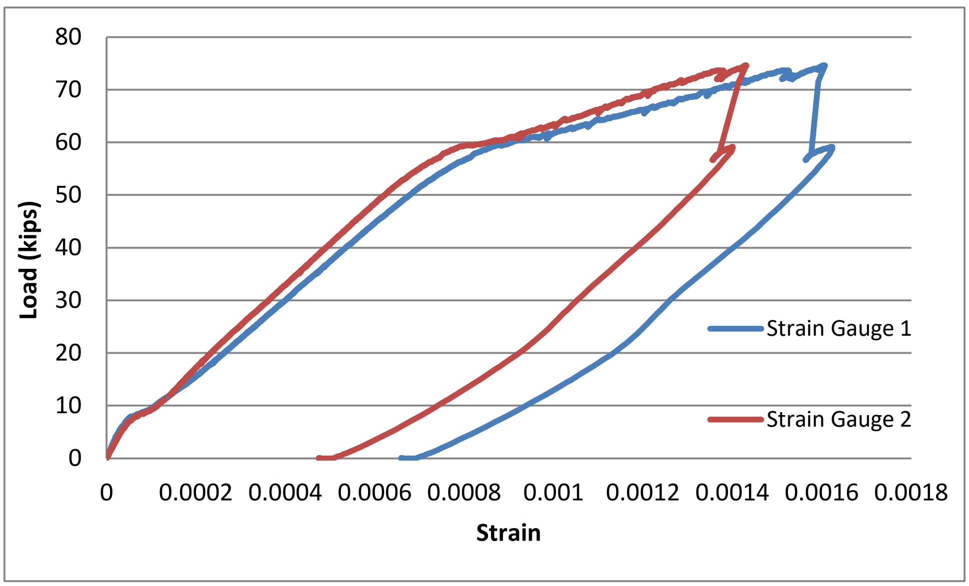

Figure 5.28: Beam 4 Load versus Concrete Top Strain

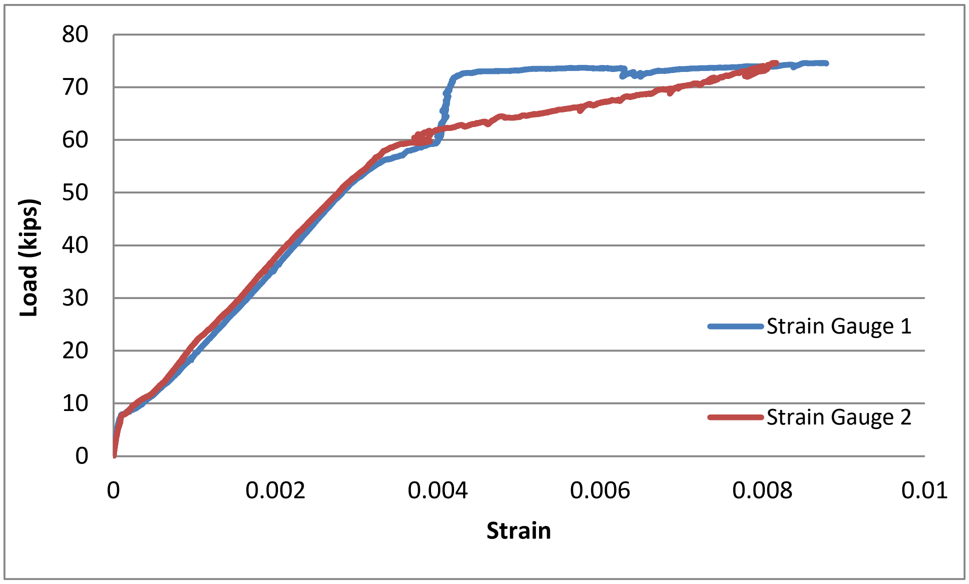

Figure 5.29: Beam 4 Load versus Steel Bar Strain

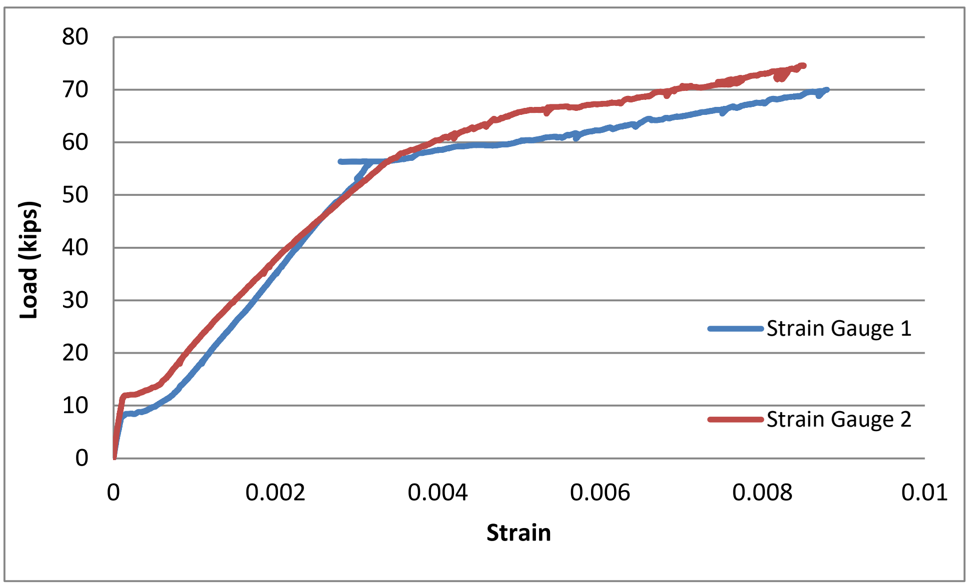

Figure 5.30: Beam 4 Load versus FRP Strain at Midspan

5.2.5 Beam 5 (with CFRP Flexural Reinforcement and Five Anchors per Shear Span)

Beam 5 was strengthened with one layer of CFRP on the bottom of the web and anchored with five splay anchors per shear span, identical in dimensions and properties to Beam 3 and Beam 4. This beam reached a failure load of 76.72 kips due to full FRP debonding of the sheet while the anchors remained in place. The deflection corresponding to the ultimate load was 2.624 inches at the midspan. The maximum strain in the concrete was 0.00208, and the maximum strain in the steel at failure load was 0.01772. One FRP strain gauge at the midspan reached a maximum strain of 0.01068, while the second gauge reached a maximum strain of 0.008309.

Figure 5.31 shows Beam 5 before the test, and Figure 5.32 and Figure 5.33 show the beam after testing. Figure 5.34 graphs the load versus deflection of the beam, while Figure 5.35 and Figure 5.36 graph the load versus concrete strain and load versus steel strain, respectively. Figure 5.37 shows the load versus FRP strain at midspan.

Figure 5.31: Beam 5 Setup before Testing

Figure 5.32: Beam 5 after Testing

Figure 5.33: Beam 5 Full Debonding of CFRP Sheet Held by Anchors

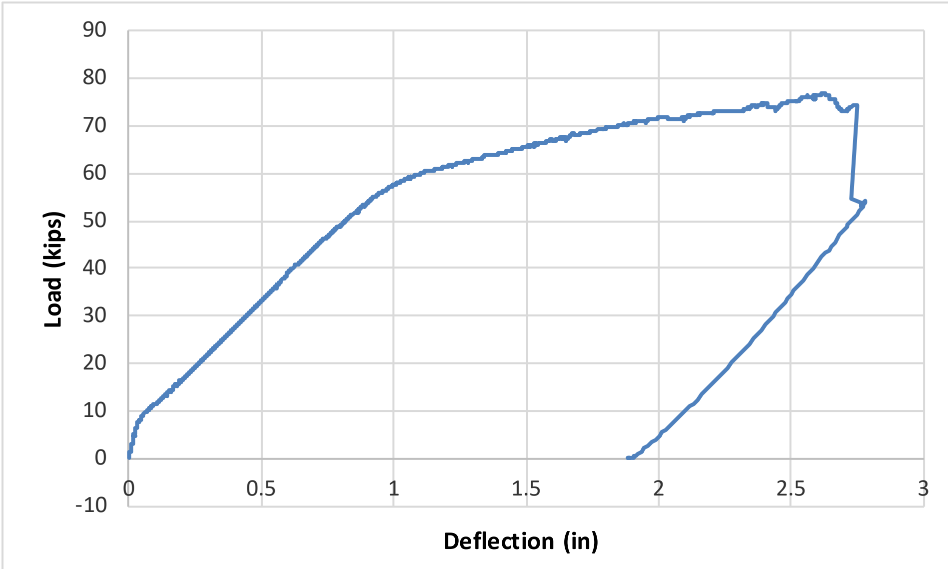

Figure 5.34: Beam 5 Load versus Deflection

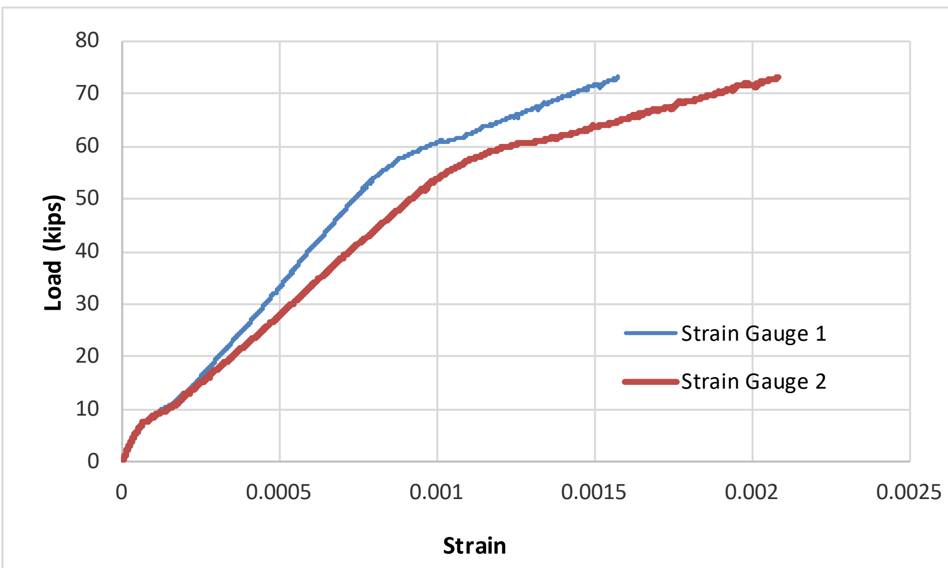

Figure 5.35: Beam 5 Load versus Concrete Top Strain

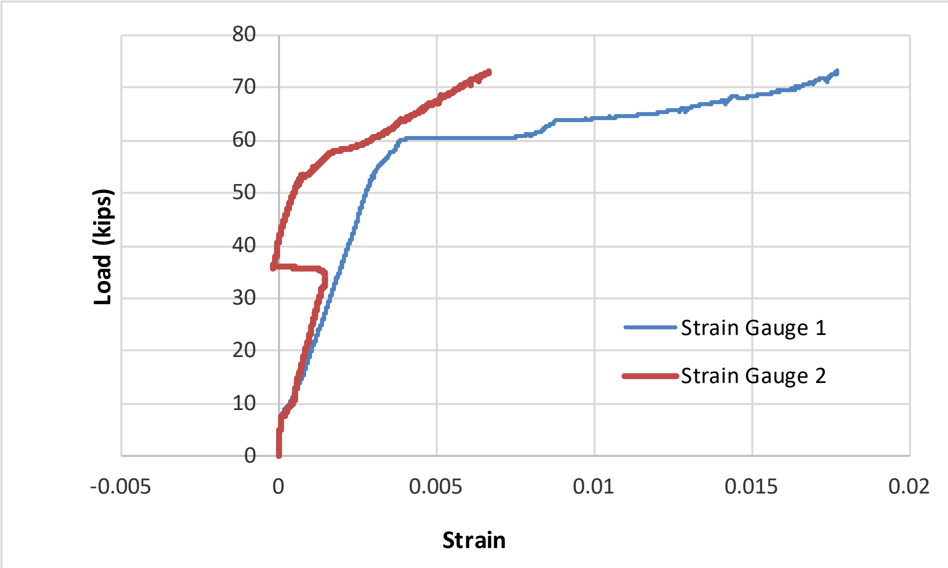

Figure 5.36: Beam 5 Load versus Steel Bar Strain

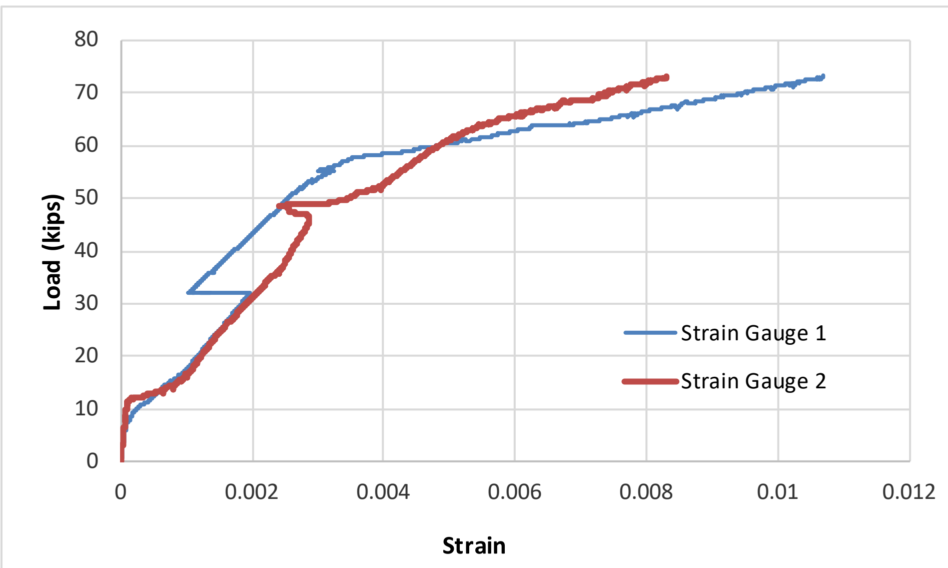

Figure 5.37: Beam 5 Load versus FRP Strain at Midspan

5.2.6 Beam 6 (with CFRP Flexural Reinforcement and Five Short Anchors per Shear Span)

The final beam, Beam 6, was strengthened with one layer of CFRP on the bottom of the web and anchored with five anchors per shear span but with a shorter splay length or area. This beam reached a failure load of 73.47 kips due to a support bearing failure since the main rebar unintentionally did not extend all the way to the end of the beam. The deflection corresponding to the ultimate load was 2.167 inches at the midspan. The maximum strain in the concrete was 0.001364, and the maximum strain in the steel was 0.01555. One FRP strain gauge at the midspan reached a maximum strain of 0.08878, while the second gauge reached a maximum strain of 0.006748.

Figure 5.38 shows Beam 6 before the test, and Figure 5.39 and Figure 5.40 show the beam after testing. Figure 5.41 graphs the load versus deflection of the beam, while Figure 5.42 and Figure 5.43 graph the load versus concrete strain and load versus steel strain, respectively. Figure 5.44 shows the load versus FRP strain at midspan.

Figure 5.38: Beam 6 Setup before Testing

Figure 5.39: Beam 6 after Testing

Figure 5.40: Beam 6 Debonding Past Anchor 2 from Midspan at Failure

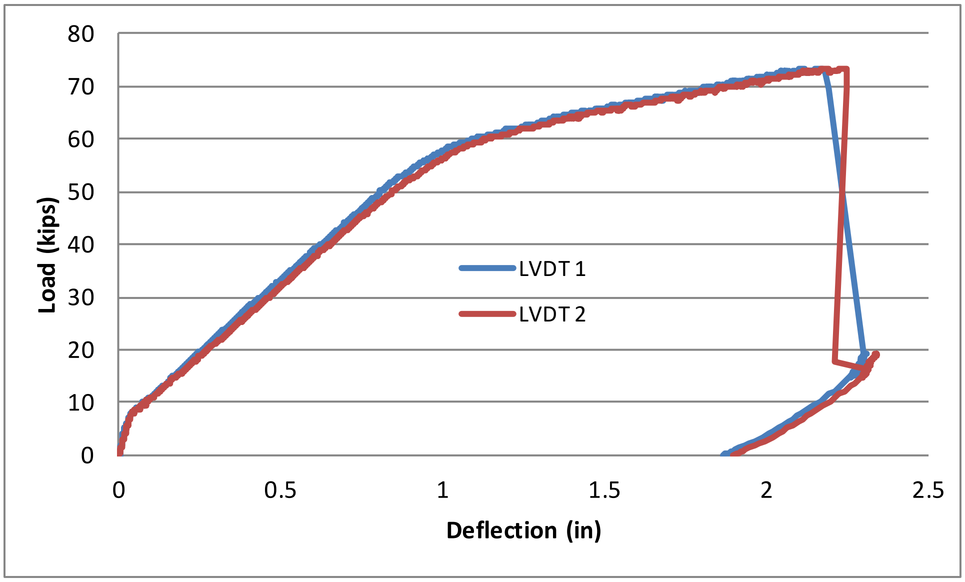

Figure 5.41: Beam 6 Load versus Deflection

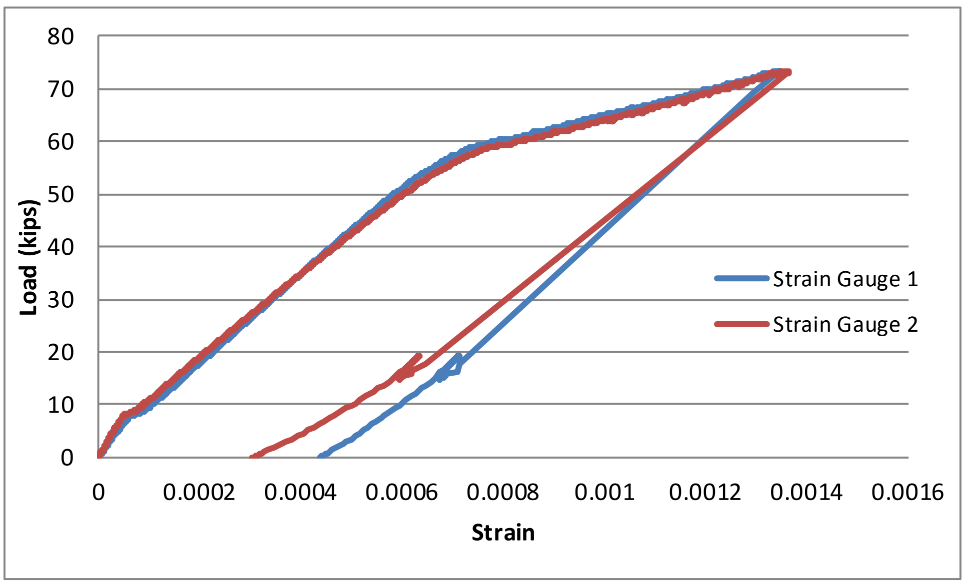

Figure 5.42: Beam 6 Load versus Concrete Top Strain

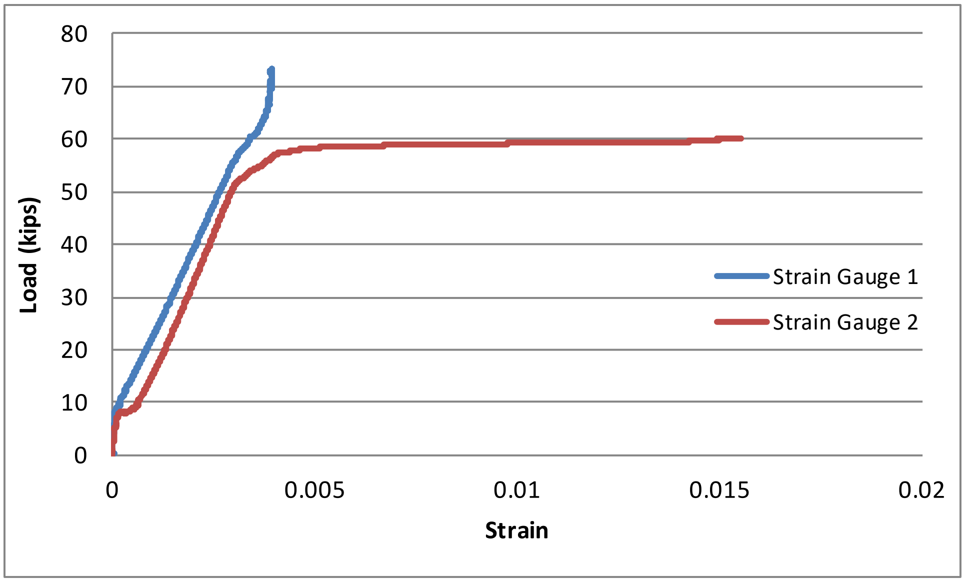

Figure 5.43: Beam 6 Load versus Steel Bar Strain

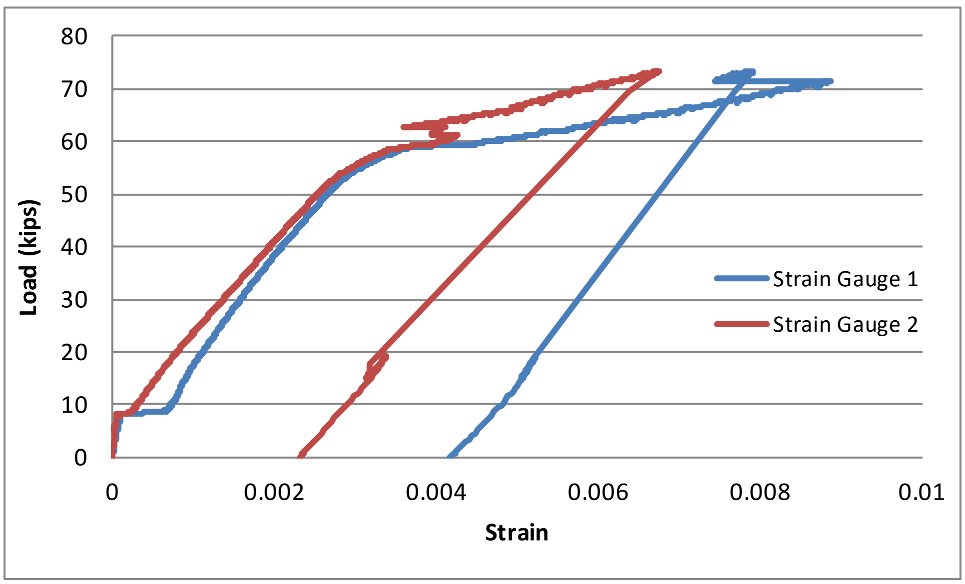

Figure 5.44: Beam 6 Load versus FRP Strain at Midspan

5.3 Comparison of Beam Results

Figure 5.45 compares the load and deflection behavior for the control beam, the unanchored strengthened beam, and the four beams anchored with 6, 4, and 5 long and short anchors.

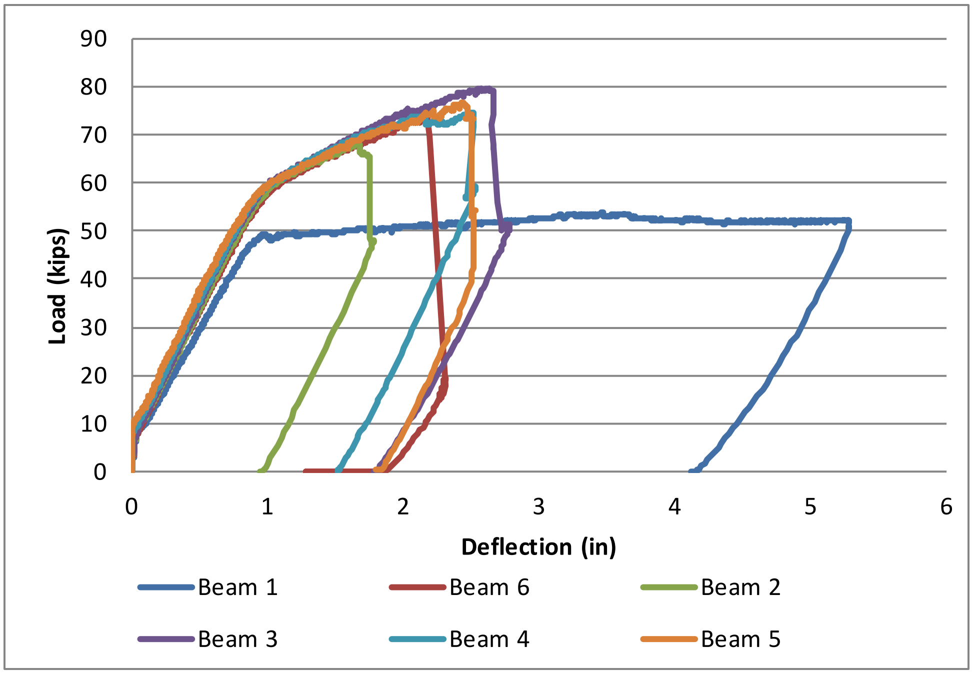

Figure 5.45: Comparison of Beam Load versus Deflection

Results shown in Table 5.1 reveal that failure loads were consistent with the number of anchors used. The theoretical model required the use of six anchors per shear span (Beam 3), resulting in CFRP rupture failure mode. However, when five anchors per shear span (Beam 5) were used, debonding failure mode occurred at a slightly lower load (see Table 5.1). On the other hand, the use of four anchors per shear span caused failure in debonding, which occurred at a slightly lower load than Beam 5 (see Table 5.1). Although Beam 6, which contained five short anchors per shear span, failed at an even lower load than Beam 4, failure by debonding almost occurred when a support bearing failure ensued because the main No. 6 rebars did not extend all the way to the beam end of that support (see Table 5.1).

Table 5.1: Properties, Ultimate Loads, and Failure Modes of Tested Beams

Beam*	CFRP Strengthening	CFRP Anchors	Anchor Spacing	Splay Length	Embedment Depth	Exp. Peak Load (k)	Failure Mode
Beam 1	None	None	None	None	None	53.73	Concrete Crushing
Beam 2	1 sheet-V-Wrap C200HM	None	None	None	None	67.91	CFRP Debonding
Beam 3**	1 sheet-V-Wrap C200HM	6 anchors per shear span	11.25 in.	10 in.	4 in.	79.66	CFRP Rupture
Beam 4	1 sheet-V-Wrap C200HM	4 anchors per shear span	18.75 in.	10 in.	4 in.	74.6	CFRP Debonding
Beam 5	1 sheet-V-Wrap C200HM	5 anchors per shear span	14 in.	10 in.	4 in.	76.72	CFRP Debonding
Beam 6***	1 sheet-V-Wrap C200HM	5 anchors per shear span	14 in.	6 in.	4 in.	73.47	Bearing Failure

*Each anchor had a diameter of 0.625 inches.
**The splay angles for Beams 3–5 was 60°.
***The splay angle for Beam 6 was 112°.

Table 5.2: Strain Efficiency Factors for Strengthened Beams

Beam #	ACI 440 Debonding Strain	Analysis Debonding Strain	Analysis Peak Load (k)	Exp. Peak Load (k)	Efficiency Factor (κ_ε)
Beam 2	0.0081	0.0088	67.99	67.91	1.09
Beam 3	0.0081	0.013	79.76	79.66	1.60
Beam 4	0.0081	0.0112	74.69	74.6	1.38
Beam 5	0.0081	0.012	76.88	76.72	1.48
Beam 6	0.0081	0.0108	73.66	73.47	1.33

Chapter 6: Summary, Conclusions, and Recommendations

6.1 Summary

This study used four-point bending to test six T-beams, one of which was a control beam. Five of the beams were strengthened with a CFRP sheet: one beam was unanchored; one beam was anchored with six CFRP splay anchors per shear span; one beam was anchored with four CFRP splay anchors per shear span; one beam was anchored with five CFRP splay anchors per shear span; and one beam was identical to Beam 5 except that it utilized shorter splay length. The control beam failed at a load of 53.73 kips and a deflection of 3.54 inches at the midspan at maximum load. Beam 2, which was strengthened with only one layer of CFRP, failed at a load of 67.91 kips and a deflection of 1.68 inches at the midspan at maximum load. Beam 3, which was strengthened with one layer of CFRP and anchored with six CFRP splay anchors per shear span, failed at a load of 79.66 kips and a deflection of 2.652 inches at the midspan at maximum load. Beam 4, which was strengthened with one layer of CFRP and anchored with four CFRP splay anchors per shear span, failed at a load of 74.60 kips with a deflection of 2.535 inches at the midspan at maximum load. Beam 5, which was strengthened with one layer of CFRP and anchored with five CFRP splay anchors per shear span, failed at a load of 76.72 kips and a deflection of 2.624 inches at the midspan at maximum load. Likewise, Beam 6 was strengthened with one layer of CFRP and anchored with five CFRP splay anchors per shear span, but the anchors had shorter splay lengths. Beam 6 failed at a load of 73.47 kips and a deflection of 2.167 inches at the midspan at maximum load.

6.2 Conclusions

Results from this study prompted several conclusions. The first conclusion was that all four anchored beams effectively increased beam strength beyond the strength of an unanchored CFRP sheet. The six CFRP splay anchors sufficiently anchored the sheet to attain full flexural capacity. The second conclusion was that beams anchored with less than six anchors failed by debonding at slightly lower loads than the CFRP rupture loads. The third conclusion asserted that anchor usage could be optimized to balance a larger ultimate load than that of unanchored FRP debonding and that there are practical benefits of decreased anchor usage. Finally, the design model developed from previous results of joist-scale T-beams was accurate to achieve rupture failure, and a results comparison indicated that a small number of uniformly distributed anchors efficiently increased the ultimate capacity of the tested beams but did not secure FRP rupture.

6.3 Recommendations for Future Work

Following the results of this study, additional testing should be conducted on anchors in one-way slabs to establish accuracy of the anchor design model for this type of structural element. In addition, experimental results and the anchor design model should be shared with ACI Committee 440 to promote inclusion in its ACI 440.2 guidelines.

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