Externally Bonded GFRP and NSM Steel Bars for Improved Strengthening of Rectangular Concrete Beams

Hayder Rasheed, Augustine Wuertz, Abdelbaset Traplsi, Hani Melhem, and Tarek Alkhrdaji

Synopsis: The technology of FRP strengthening has matured to a great extent. However, there is always room for performance improvements. In this study, external bonding of GFRP and near surface mounting of regular steel bars is combined to improve the behavior, delay the failure, and enhance the economy of the strengthening. E-Glass FRP is selected due to its inexpensive cost and non-conductive properties to shield the NSM steel bars from corrosion. On the other hand, the use of NSM bars gives redundancy against vandalism and environmental deterioration of the GFRP. An experimental program is conducted in which five rectangular cross-section beams are designed and built. The first beam is tested as a control beam failing at about 12 kips (53.4 kN). The second beam is strengthened using 5 layers of CFRP, which failed at 27.1 kips (120.5 kN). CFRP U-wraps were used to anchor this external reinforcement. The third beam is strengthened using two #5 steel NSM bars and 1 layer of GFRP, both extending to the support. GFRP U-wraps were applied to anchor this external reinforcement. This beam failed at 31.5 kips (140 kN). The fourth beam is strengthened with the same system used for the third beam. However, the NSM steel bars were cut short covering 26% of the shear-span only while the GFRP was extended to the support. This beam failed at 30.7 kips (136.5 kN) due to the lack of sufficient development of the NSM steel bars and the shear stress concentration at the steel bar cut off point. Nevertheless, the failure load developed was higher than that of 5 layers of CFRP used for beam 2. The fifth beam was strengthened exactly as the fourth beam, but once strengthened, was loaded five times to cracking load and then submerged in a highly concentrated saline solution for six months. The beam was then tested to failure with a failure load of 29.8 kips (132.6 kN), showing that the GFRP wrapping provided good corrosion resistance.

Keywords: concrete beams; externally bonded; fiber reinforced polymer (FRP); glass; near surface mounted (NSM); steel rebar; strengthening.

Hayder Rasheed is Associate Professor of Structural Engineering at Kansas State University. He is voting member of ACI Committees 440-Fiber Reinforced Polymer Reinforcement, 441-Columns and 335-Composite Hybrid Structures. His research interests are behavior of concrete structures reinforced and strengthened by FRP and stability of FRP composite structural components. He is a Fellow of ASCE.

Augustine Wuertz is a Graduate Research Assistant and Structural Engineering student in the Department of Civil Engineering at Kansas State University, performing research in the area of strengthening concrete beams with near surface mounted steel rebars and FRP wrapping.

Abdelbaset Traplsi is a Graduate Research Assistant and Structural Engineering student in the Department of Civil Engineering at Kansas State University, performing research in the area of strengthening concrete beams with near surface mounted steel rebars and FRP wrapping. He will be pursuing his Ph.D in Structural Engineering with research focused on FRP strengthened concrete.

Hani Melhem is Professor of Structural Engineering at Kansas State University. He received his Ph.D. from the University of Pittsburgh in 1989 with emphasis on fatigue of steel bridges and artificial intelligence applications. He served as chief co-editor of the ASCE Journal of Computing in Civil Engineering, and chair of the executive committee of the ASCE Technical Council on Computing and Information Technology (TCCIT). He is a registered professional engineering in Texas and is a Fellow of ASCE.

Tarek Alkhrdaji is Vice President-Engineering with Baltimore-based Structural Technologies. He has been involved in numerous projects involving structural repair and upgrade as well as full-scale in-situ load testing. He is an active member of ACI committees 562 (Repair Code) 440 (FRP Repair/Strengthening) and ACI 437 (Strength Evaluation), as well as a member of the International Concrete Repair Institute (ICRI) and the American Society of Civil Engineers (ASCE).

INTRODUCTION

The use of fiber reinforced polymer (FRP) technology for strengthening has become acknowledged by structural engineers and has reached a full acceptance. However, researchers are always looking for improvements in performance. The technique of externally bonded FRP in strengthening concrete members began in Switzerland when Meier used Carbon FRP with reinforced concrete beams [1]. Since then, a large volume of literature has been contributed in this area. It is important to note here that a large number of studies yielded premature flexural failures by debonding of the FRP sheets or concrete cover delamination. Usually, the debonding failure occurs within the span at one of the cracks whereas concrete cover delamination starts at the FRP end. One of the other known techniques of flexural strengthening is using near surface mounted (NSM) FRP bars. Basically, the NSM reinforcement technique consists of cutting grooves into the concrete cover and bonding the bars into the grooves by using appropriate adhesive material such as epoxy resin or cement mortar. The idea of NSM reinforcement started in Europe by using steel bars between 1940 and 1950 [2]. Corrosion of reinforcing steel is one of the big problems to the US economy, which is very costly (about $297 billion per year) [3]. There is a lack of adequate understanding of the long-term performance of epoxy bonded NSM steel bars. GFRP sheets, on the other hand, have some desirable properties such as lightweight, resistance to chlorides and chemical attack [3] as well as their nonconductive properties. In terms of conductivity, using GFRP sheets as externally bonded reinforcement has more advantages than using CFRP, which is known to be conductive. This study is intended to investigate the behavior of combining NSM steel reinforcement and GFRP wrapping as a flexural strengthening system along with GFRP U-Wraps as additional anchorage reinforcement. Another goal is to compare the results of using the combined technique with the state of the art technique of using 5 layers of externally bonded and continuously anchored CFRP reinforcement.

RESEARCH SIGNIFICANCE

The main intent of this study is to investigate the benefits of combining the techniques of using near surface mounted (NSM) steel bars with externally bonded GFRP and comparing the results to those of an identical beam strengthened with CFRP only. E-Glass FRP is selected due to its inexpensive cost and non-conductive properties to shield the NSM steel bars from corrosion in addition to providing some post-yielding strength. On the other hand, the use of NSM bars gives redundancy against vandalism and environmental deterioration of the GFRP as well as ductility. Another objective is to compare the effects of using full length NSM rebar to those of shorter NSM rebar, which could more easily be shielded with GFRP, and the way the behavior differs with the use of special GFRP anchorage at the NSM bars cut off points.

DESIGN OF BEAMS

The design of the beams was performed based on ACI 318-11 [4] with strain compatibility and force equilibrium. The geometry was chosen so that the beam would fail in flexure. The external flexural reinforcement was designed based on ACI 440.2R-08 [5] using the same principles. Additional shear reinforcement was unnecessary. However, U-wrap anchorage external stirrups were used in order to prevent delamination of the concrete cover or debonding of the GFRP sheet. The U-wrap stirrups were designed based on the shear friction model of ACI 318-11 adapted by Rasheed et al. [6] to provide continuous anchorage to the GFRP and NSM bars. The tension force in the stirrups is determined by clamping a horizontal crack through shear friction. The horizontal shear per unit length of the plated shear span can be found from maximum tensile force divided by the shear span. The area of anchorage reinforcement needed can then be found by equating the tension in the U-wraps per unit length to the area of these stirrups multiplied by their allowable stress. Accordingly and based on the properties of the GFRP sheets, the spacing can then be calculated. The size of the GFRP stirrups was determined to be 8.5 inches (215.9 mm) wide and spaced every 1 foot (304.8 mm) on center applied as a single layer. An additional double anchorage of 20.5 inches (520.7 mm) wide U-wrap is applied, 10.25 inches (260.4 mm) on both sides of the short NSM bars cut off points in beams R4 and R5. Below are the calculations of the U-wrap anchorage. At the ultimate analytical flexural load of 30.8 kips (137 kN), the maximum tension force in the NSM bars and GFRP is:

$$T_{NSM} = 43.4\ k\ (193.1\ kN) \tag{1}$$

$$T_{GFRP} = 25.5\ k\ (113.4\ kN) \tag{2}$$

$$T_{total} = 68.9\ k\ (306.5\ kN) \tag{3}$$

$$V_{hu} = \frac{68.9\ k}{5.5\ ft} = 12.53\frac{k}{ft}\ \ (182.9\frac{kN}{m}) \tag{4}$$

$$T_{sf} = \frac{V_{hu}}{\mu} = \frac{12.53\ k/ft}{1.4} = 8.95\frac{k}{ft}\ (130.7\frac{kN}{m}) \tag{5}$$

$$T_{sf} = \emptyset\ A_{vf}E_f\varepsilon_{fe} \tag{6}$$

$$8.95\frac{k}{ft} = 0.75\ A_{vf} * 3790\ ksi * 0.00375 \tag{7}$$

$$A_{vf} = 0.84\frac{in^2}{ft}\ (1778\frac{mm^2}{m}) \tag{8}$$

$$A_{vf}S = 2nt_f\ w_f \tag{9}$$

$$0.84\frac{in^2}{ft} * 1ft = 2 * 1 * 0.05in\ w_f \tag{10}$$

$$w_f = 8.4\ in \approx 8.5\ in\ \ for\ every\ 1\ ft\ on\ center\ (216\ mm\ per\ 305\ mm\ on\ center) \tag{11}$$

SPECIMEN GEOMETRY AND REINFORCEMENT

Each rectangular beam had a width of 6 inches (153 mm) and a height of 12 inches (305 mm). The length of each beam was 16 ft (4.88 m) with a clear span of 15.5 ft (4.72 m). This geometry was chosen to ensure a flexural failure and to enable a direct comparison with an earlier experimental setup [7]. The primary tension reinforcement was composed of two steel rebars with diameter of 0.625 inches (#5) (15.875 mm). The longitudinal compression reinforcement consisted of two steel rebars with diameter of 0.375 inches (#3) (9.52 mm). Steel rebar stirrups, with diameter of 0.375 inches (#3) (9.52 mm), were used for shear reinforcement and were spaced every 5 inches (127 mm) on center. Concrete cover was 1 inch (25.4 mm) on all sides. Figure 1 below shows the cross section dimensions and the internal steel reinforcement.

SPECIMEN CONSTRUCTION

Steel cages were built by wiring together the tension and compression reinforcement and the stirrups. Strain gages were placed on the main tension reinforcement before casting the beams in order to determine the steel strain response during testing. Strain gages were also used on the top surface of the concrete and on the bottom of the beam attached to the GFRP sheet. Forms were constructed, the steel cages were placed in them, which can be seen in Figure 2 below, and then the beams were cast using ready mixed concrete and standard construction practices. Two square wooden rods were screwed to the bottom of the forms to create 1 in. by 1 in. (25.4 mm x 25.4 mm) square grooves, in which the NSM bars would be placed later. Typically, grooves would have to be cut from the existing beams being strengthened using a diamond bit saw or grinder. After 28 days, concrete cylinders were tested yielding a compressive strength of approximately 8000 psi (55.2 MPa).

SURFACE PREPARATION AND APPLICATION OF GFRP SHEETS

First, the wooden grooves were hand chiseled out of the beams and then the grooves, soffit, and sides of the beams were sand blasted to remove any additional wood, dirt, or undesirable material, which can be seen in Figure 3 below. This was done to ensure proper bonding between GFRP and concrete. Large “bug holes” or deformations in the beams were filled in with epoxy, to create an even surface to apply the GFRP sheets [8]. Second, the NSM steel rebars were cut to size since one beam had NSM rebars running the entire length of the beam and two had the NSM rebars spanning only a distance of 7 feet (2.13 m), centered on the beam. These were adhered into the grooves by applying epoxy mixture in the grooves until they were half-way full. Then the rebars were pressed into the epoxy mixture and the grooves were filled all the way. The surface was then scraped until it had a smooth finish which can be seen in Figure 4 below. The epoxy resin used was a special mixture of epoxy and silica fume as prescribed by the supplier (Structural Technologies). Then the edges of the beam were rounded off to approximately a 0.5 inch (12.7 mm) radius using a grinding wheel and diamond bit. This was done to ensure proper bonding of the GFRP and to avoid damaging the fibers across the corner edges. Any other deformations were also ground down using the grinding wheel [5]. Then the beams were moved inside to avoid weather conditions that might be detrimental to the process of applying the GFRP sheets. Finally, applying the GFRP sheets was done in a wet lay-up process which took several steps. The sheets were cut to 15 ft. long by 16 in. wide (4.57 m x 40.64 cm) continuous pieces. Also the U-wraps were cut to 22 inches long by 8.5 inches wide (558.8 mm x 215.9 mm). Then, a layer of resin was applied to the beam surface with a paint roller to ensure the FRP sheet would bond to the concrete surface. Next, the GFRP sheet was pressed in the resin using wooden and plastic rollers. This was important since it forced any air out that may be trapped under the sheet. Then another layer of resin was applied to fully saturate the GFRP fibers. This process was repeated for the U-wraps immediately after the flexural sheet was applied. The U-wraps were 8.5 inches wide (215.9 mm) placed transversely every 1 foot (304.8 mm) on center, Figure 5. In the case of the beam with the 7 foot (2.13 m) long NSM rebars, two layers of U-wraps, that were 22 inches long by 20.5 inches wide (558.8 mm x 520.7 mm), were used only at the cut off points of the NSM rebars, since this is a critical point for the stress transfer between the NSM steel and the GFRP sheet, Figure 6. In the case of the beam with CFRP only, the U-wraps were only 5.5 inches (139.7 mm) wide and spaced every 1 foot (304.8 mm) on center, Figure 7. Table 1 below shows the strengthening details of each of the beams. Figure 8 shows the installed FRP sheet and the application of the U-wraps. Afterwards, the beam was allowed to sit for 24 hours to ensure the resin was properly cured [9].

MECHANICAL PROPERTIES OF STRENGTHENING MATERIALS

The steel had a modulus of 29000 ksi (200 GPa) with an average tensile yield strength of approximately 70 ksi (483 MPa). The epoxy, resin, and GFRP materials were all supplied by VSL Industries (Baltimore, MD). The epoxy/resin was the V-Wrap 700 Adhesive Resin, which has a tensile modulus of 500 ksi (3.45 GPa) and tensile strength of 9 ksi (62 MPa). The GFRP sheets had a modulus of 3790 ksi (26.1 GPa) and a tensile strength of 83.4 ksi (575 MPa) based on a thickness of 0.05 inches (1.27 mm) according to the manufacturer. Table 2 shows the mechanical properties of GFRP as well as the CFRP used for comparison as tested by the authors. The CFRP had an average modulus and strength of approximately 8700 ksi (60 GPa) and 110 ksi (758.4 MPa) respectively, based on coupon testing according to ASTM D3039. These values correspond with an ultimate strain of 0.013 [7].

TEST SETUP AND PROCEDURES

For the flexural tests, each beam was simply supported at a distance of 3 in. (7.62 cm) from the ends, giving a test clear span of 15.5 ft (4.72 m). A hydraulic actuator was used to load the beams at mid span along with a 4 ft. (1.22 m) wide spreader beam to apply the load to the beams at two points symmetrically. This was done to ensure a constant moment region of the beam. Figure 9 below shows the setup of the beams. Two 13 in. (330.2 mm), linear variable displacement transducers (LVDTs) were used at mid span to measure the deflection while loading. The actual deflection was taken as the average of these two readings. Strain gages were placed on the top of the beam on the concrete, the tension rebar, and on the FRP at the bottom of the beam, all at mid span. The strain gauges and LVDTs were wired to a data acquisition system. Each beam was statically loaded to failure. The beams were loaded at a rate of 1000 lbs/min (4.45 kN/min) and the load, deflection, and strain readings were taken approximately every 1.5 seconds or every 25 lbs (111 N). After each test, the data was then used for analysis and comparisons.

ANALYSIS OF RESULTS

The analysis program used to design and analyze the specimens was a Microsoft Excel based program developed by Calvin Reed, a former graduate student at Kansas State University. This program gives the user the option of selecting the cross-section type, either rectangular or T-shaped, and then the appropriate dimensions are entered. The program then allows the user to select a loading type from uniform loading, three-point bending, or four-point bending. Material properties are then input such as concrete compressive strength and steel yielding strength. Different types of reinforcement can be entered such as mild steel, prestressed steel, glass bars, and/or FRP sheets. The user can also enter the properties, size, and location of each of these different types of reinforcement.

The program model predicts the flexural response of the beam by using strain compatibility and incremental deformation techniques based on the specimen geometry and material properties. The program also allows the user to access the code to adjust the model as needed. The program uses an iterative process to determine the moment curvature relationship. An incremental analysis is performed by dividing the specimen into a large number of segments. A moment is calculated for each segment and then the curvature is determined using the moment curvature relationship. The deflection is then calculated using the numerical integration of curvature by the moment area method. The results from this program are used to analyze the beams tested under this program as well as develop the theoretical load-deflection curves which are compared to the experimental results in the following section.

RESULTS

The control beam R1 failed at an ultimate load of 12.2 kips (54.3 kN) with a failure mode of concrete crushing after yielding of tension steel. This is a desirable failure mode and is recommended by ACI 318 when designing concrete beams. Figure 10 below shows a comparison of the load vs. deflection for the theoretical and experimental results. The beam reinforced with the full length NSM steel rebars (Beam R2) failed at an ultimate load of 31.6 kips (140.5 kN) with a failure mode of concrete crushing inside the constant moment region of the beam after yielding of tensile steel. This is a desirable failure mode since the beam gives warning sign as to when failure is about to occur. This failure mode occurred due to the fact that the beam reached its ultimate compressive strength in its compression zone. No debonding of the GFRP sheet or U-wraps was observed, only post-failure rupture of the sheet fibers at the critical section. Figure 11 below shows the failure of the beam with the full length NSM rebars. Figure 12 shows a comparison of the load vs. deflection for the theoretical and experimental results. The beam R3 strengthened with 5 layers of CFRP along its soffit and anchored with transverse CFRP U-wraps failed at an ultimate load of 27.1 kips (120.5 kN) with a failure mode of concrete crushing followed by rupture of the FRP fibers at the ultimate load. Figure 13 below shows the failed beam with the crushed concrete and the rupture of the CFRP flexural reinforcement. Figure 14 shows a comparison of the load vs. deflection for the theoretical and experimental results. The beam R4 strengthened with the 7 ft (2.13 m) long NSM steel rebars and full external GFRP failed at an ultimate load of 30.7 kips (136.5 kN) with a failure mode of concrete crushing preceded by the GFRP rupture of the double U-wrap. The GFRP failed at the location of the end of the NSM steel rebar, which shows that there was stress concentrations at that location, which is outside of the constant moment region, shearing apart the U-wrap, rupturing the flexural GFRP and causing evident concrete crushing directly afterwards. As can be seen in Figure 15 below, the double layered U-wrap sheared off in the middle, which ultimately led to the failure of the beam. Figure 16 shows a comparison of the load vs. deflection for the theoretical and experimental results.

A summary of the experimental results from each beam can be seen in Table 3 below. As seen in the table, the technique of combining NSM rebars and externally bonded GFRP resulted in an increase in the flexural capacity by 2.5 times that of the control beam. This has out-performed that of using 5 layers of externally bonded CFRP only with continuous anchorage along the beam. Figure 17 shows the experimental comparison between the load-deflection responses of each beam. This shows that not only do the beams that use the combined externally bonded FRP and NSM steel rebars have a higher flexural capacity but they also have a more ductile failure. Also, it would be prudent to compare the results of having the combined strengthening technique of externally bonded FRP and NSM rebar to strengthening a beam with either externally bonded FRP only or NSM rebar only, which could give a relationship on how much strength is derived from each technique. Figure 18 gives this comparison using the theoretical data for the strengthening technique of NSM rebar only and externally bonded GFRP sheet only and the experimental data for Specimens R1 and R2.

BEHAVIOR OF FRP STRENGTHENED BEAMS

The behavior of FRP strengthened concrete beams varied with the various strengthening techniques examined: The first strengthened beam (R2) had two full-length NSM steel reinforcement bars, a GFRP longitudinal sheet wrapped around the tension zone and GFRP U-wraps evenly distributed along the beam for continuous anchorage. All the FRP remained intact up to flexural failure by yielding of the NSM steel bars followed concrete crushing at the critical section, Figure 11. It is evident in that Figure that the GFRP locally ruptured after the primary failure mode described above. The second strengthened beam (R3) had five layers of CFRP covering the entire soffit of the beam as well as CFRP U-wraps distributed along the beam for continuous anchorage. The anchorage was successful in preventing any premature debonding failure of the CFRP, which ruptured locally after the main flexural failure mode of concrete crushing of the compression zone at mid span. The third strengthened beam (R4) had two short NSM bars (7-ft long) (2.13 m) centered at the beam mid span, the same GFRP longitudinal sheet wrapped around the tension zone and extended to the supports and GFRP U-wraps evenly distributed along the beam for continuous anchorage in addition to double layer wide U-wraps at the NSM bars cut off points for added anchorage. Failure took place at one of the NSM bars cut off points outside the constant moment region by delamination of the GFRP and the NSM bar ends accompanied by concrete crushing of the compression zone. The double U-wrap is believed to have delayed the premature delamination of the short NSM bars.

CONCLUSIONS

The following conclusions may be drawn about the usage of NSM steel reinforcement combined with the technology of externally bonded GFRP sheets from this study:

- The combination of NSM steel reinforcement with externally bonded GFRP sheets resulted in an increase of at least 2.5 times the capacity of the control beam.
- The development length of the 7-ft (2.13 m) NSM steel strengthened beam, even though insufficient caused sufficient performance due to the use of double end anchorage.
- The use of 7-ft (2.13 m) NSM steel rebars is very comparable to the use of the full length NSM steel rebars.
- The NSM steel rebar helps create a more ductile failure mode compared to using only CFRP while maintaining similar strength.
- The use of the NSM rebar with one layer of GFRP may be less costly than using several layers of CFRP with respect to the materials cost.
- The externally bonded GFRP sheet provided good corrosion resistance from the saline solution for the NSM rebar.
- As a result, the use of NSM rebar with GFRP proved to be more effective than using 5 layers of CFRP.

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TABLE 1—Specimen Strengthening Details and Parameters

Specimen	Externally Bonded FRP Type	NSM rebar length (ft.)	U-wrap width (in.)	Double layer of U-wrap at end of NSM rebar? (YES/NO)
(R1)	None	None	N/A	N/A
(R2)	GFRP	16 (4.877 m)	8.5 (215.9 mm)	NO
(R3)	CFRP	None	5.5 (139.7 mm)	N/A
(R4)	GFRP	7 (2.13 m)	8.5 (215.9 mm)	YES

TABLE 2—Mechanical Properties of GFRP and CFRP Coupons

Specimen	Ultimate Strength (ksi)	Modulus (ksi)	Ultimate Strain (με)
GFRP-1	32.7 (225.5 MPa)	2173 (15.0 GPa)	15065
GFRP-2	32.7 (225.5 MPa)	1971 (13.6 GPa)	16598
GFRP-3	31.9 (220 MPa)	1886 (13.0 GPa)	16936
GFRP-4	44.5 (306.8 MPa)	2026 (13.97 GPa)	21954
GFRP-5	38.5 (265.4 MPa)	2265 (15.6 GPa)	16987
GFRP-6	50.2 (346.1 MPa)	2688 (18.5 GPa)	18675
Average GFRP	38.4 (264.8 MPa)	2168 (14.95 GPa)	17702
Specimen	Ultimate Strength (ksi)	Modulus (ksi)	Ultimate Strain (με)
CFRP-1	110.3 (760.5 MPa)	9099 (62.7 GPa)	12514
CFRP-2	107.7 (742.6 MPa)	7628 (52.6 GPa)	14123
CFRP-3	111.6 (769.5 MPa)	8631 (59.5 GPa)	12931
CFRP-4	113.3 (781.2 MPa)	9484 (65.4 GPa)	11947
CFRP-5	113.9 (785.3 MPa)	8764 (60.4 GPa)	12993
Average CFRP	111.4 (768.1 MPa)	8721 (60.1 GPa)	12902

Table 3—Summary of Experimental Results

Specimen	Ultimate Load (kips)	Deflection (in.)	Load Increase (%)	Failure Mode
Control (R1)	12.2 (54.3 kN)	4.62 (11.7 cm)	N/A	Concrete crushing after yielding of Steel
Full Length NSM rebars (R2)	31.6 (140.6 kN)	3.81 (9.7 cm)	259.0	Concrete Crushing after yielding of Steel
CFRP only (R3)	27.1 (120.5 kN)	2.77 (7.04 cm)	222.1	Concrete Crushing followed by FRP Rupture
7 ft. NSM rebars (R4)	30.7 (136.6 kN)	3.38 (8.6 cm)	251.6	Concrete crushing after FRP U-wrap Rupture

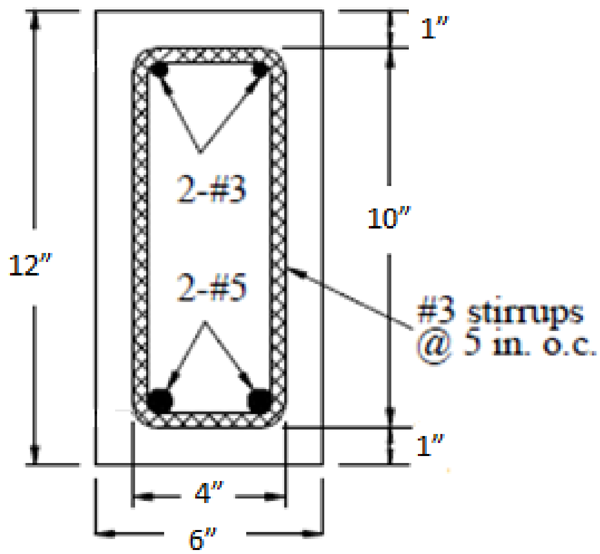

Figure 1—Cross Section of Rectangular Beam with Internal Steel Reinforcement [7]

Figure 2—Caging inside of Formwork before Casting

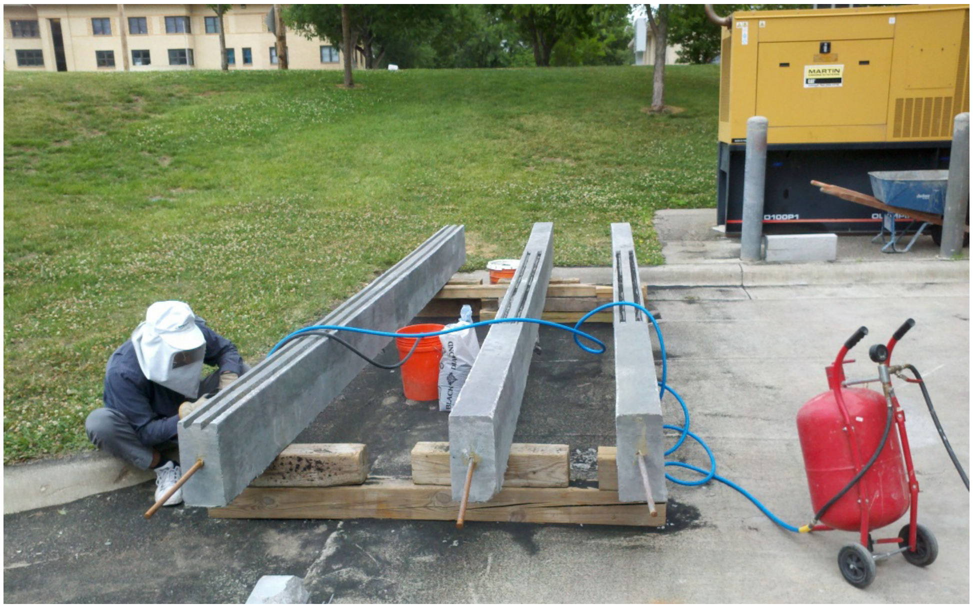

Figure 3—Sandblasting Beams after Wooden Grooves have been removed

Figure 4—Applying Epoxy to NSM Grooves after steel rebars are placed

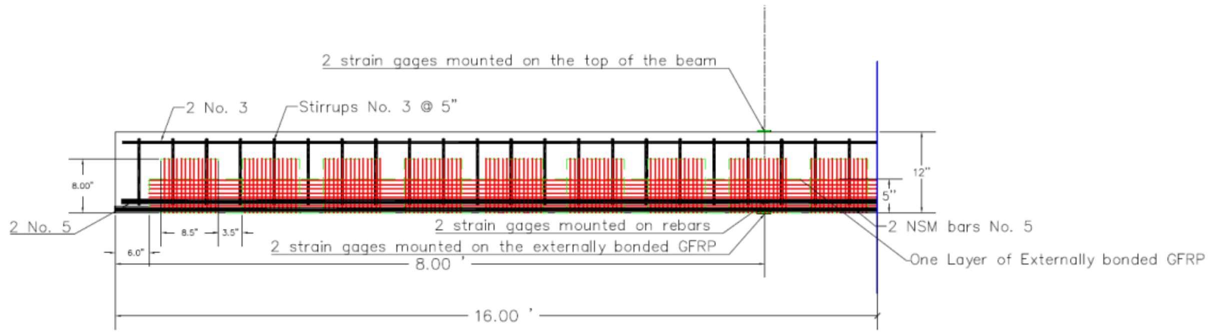

Figure 5—Profile of Beam R2 showing the flexural and anchorage GFRP and Full Length NSM Steel Bars

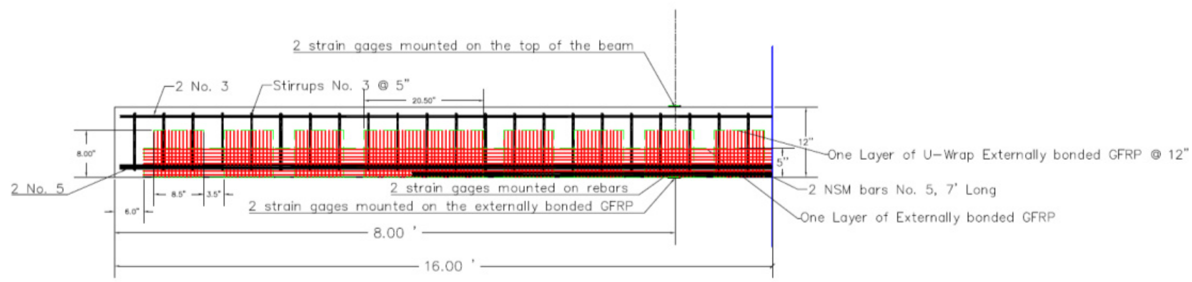

Figure 6—Profile of Beam R4 showing the flexural and anchorage GFRP and Short NSM Steel Bars

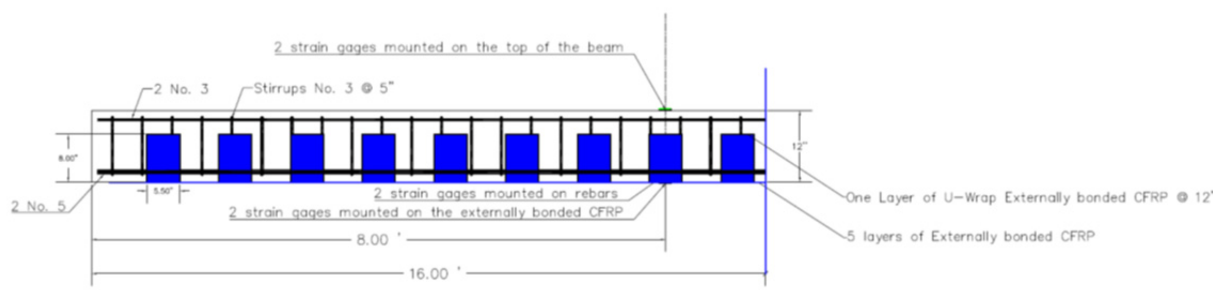

Figure 7—Profile of Beam R3 showing the flexural and anchorage CFRP Sheets

Figure 8—Applying U-Wraps to Beam R4

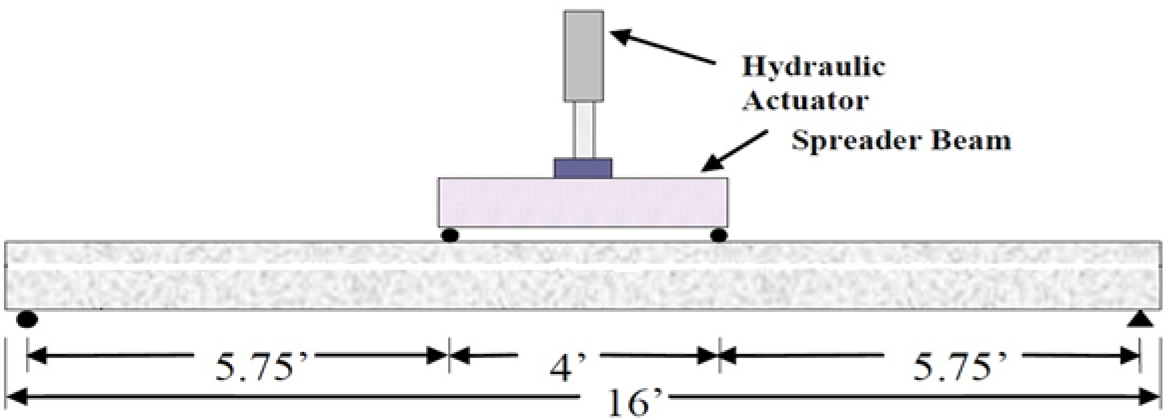

Figure 9—Testing Setup of Beams

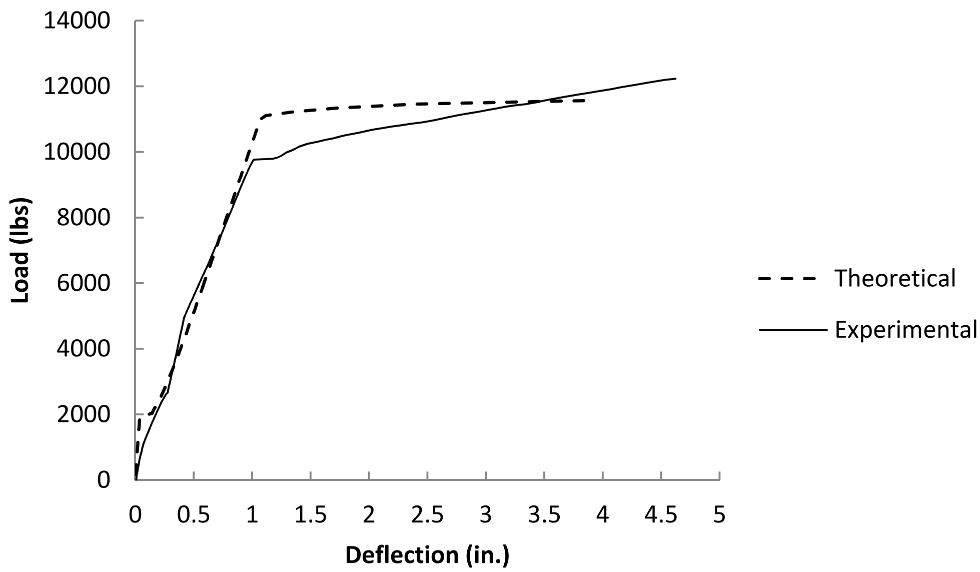

Figure 10—Load vs. Deflection Response for the Control Beam R1

Figure 11—Concrete Crushing of the Beam R2 with the Full Length NSM rebars

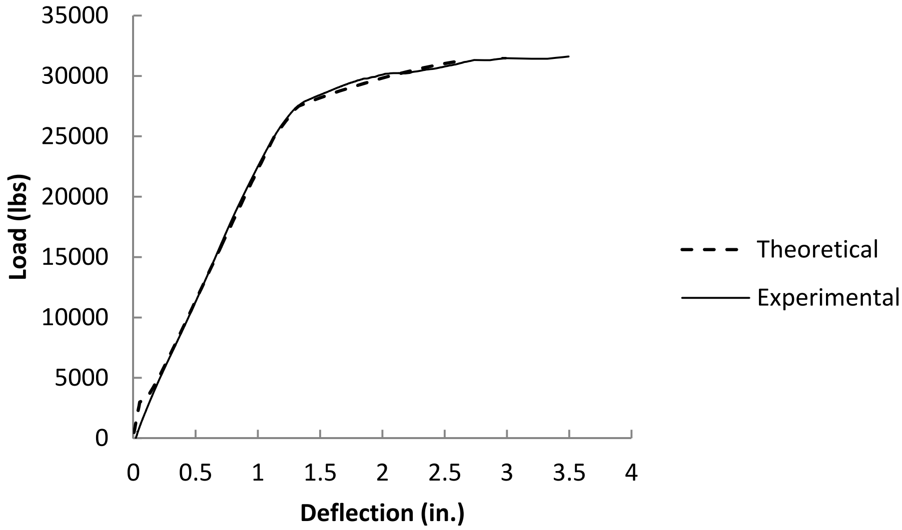

Figure 12—Load vs. Deflection Response for the Beam R2 with the Full Length NSM rebars

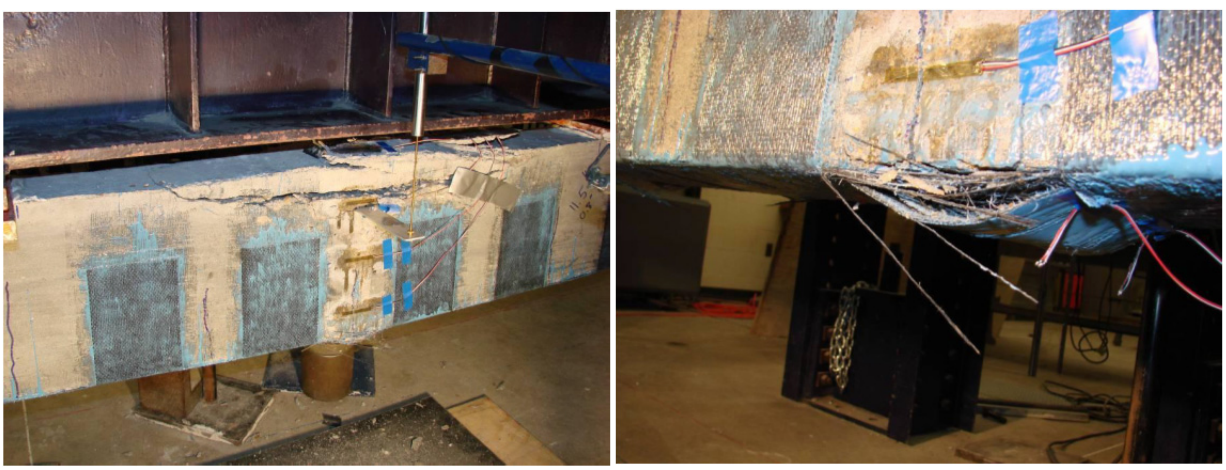

Figure 13—Concrete Crushing followed by CFRP Rupture in Beam R3

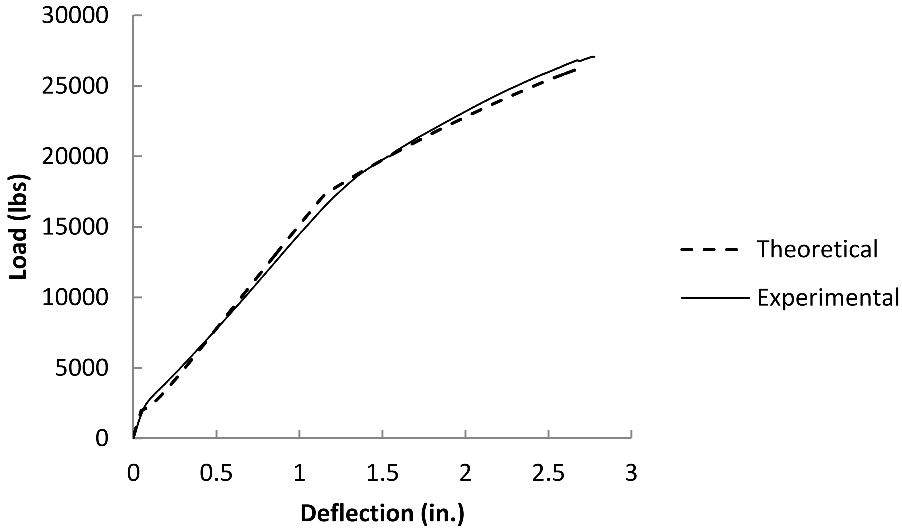

Figure 14—Load vs. Deflection Response for the Beam R3 with CFRP only

Figure 15—FRP Rupture of Shear Reinforcement and Concrete Crushing of the Beam R4 with 7-ft NSM **rebars**

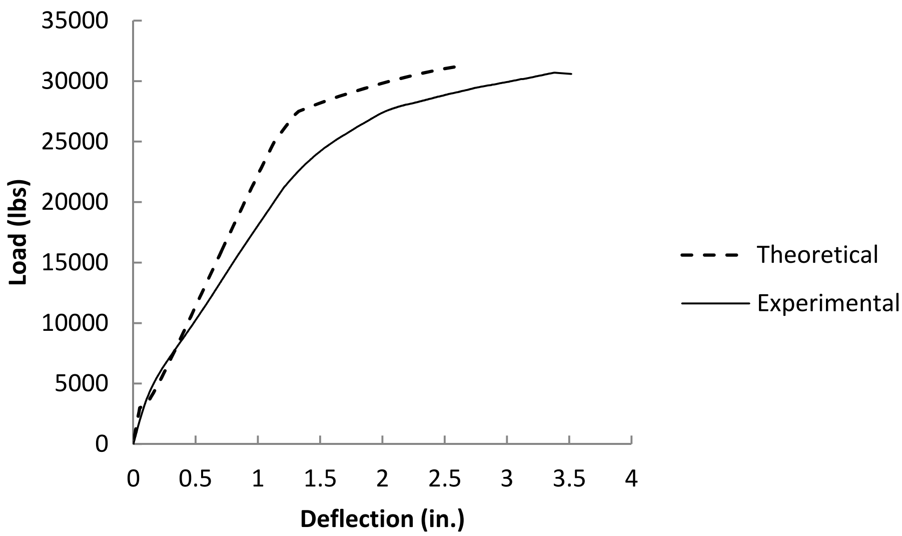

Figure 16—Load vs. Deflection for the Beam R4 with the 7-ft long NSM rebars

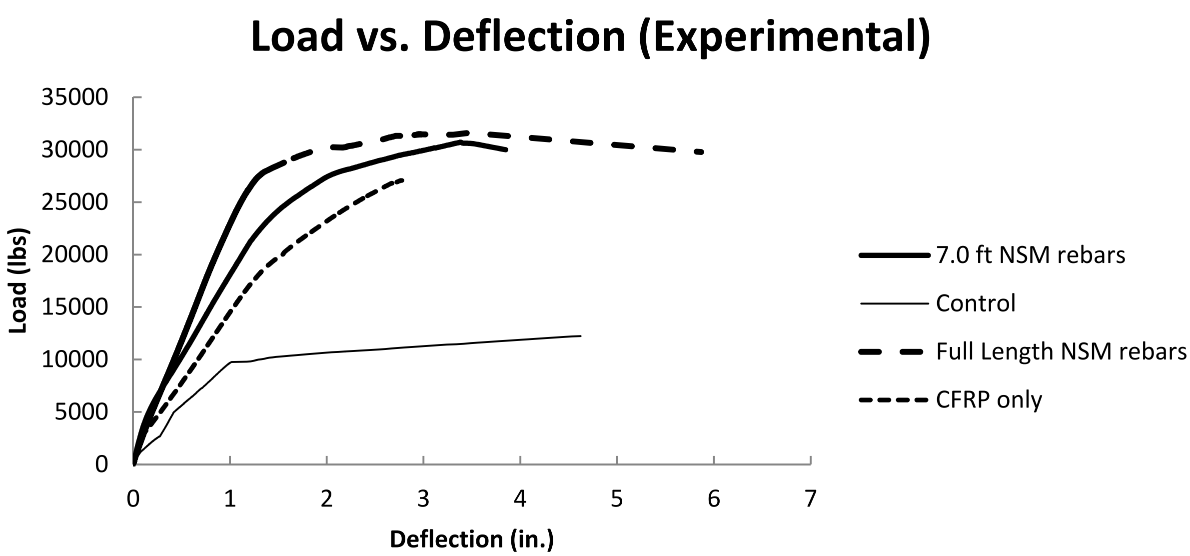

Figure 17—Load vs. Deflection Comparison for Experimental Results

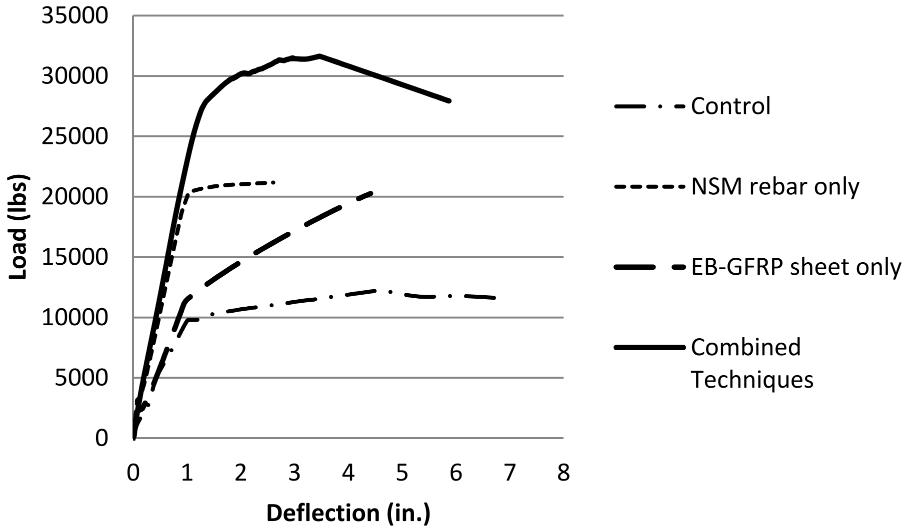

Figure 18—Load vs. Deflection Comparison for the NSM or EB vs. Combined Technique