OPTIMUM CFRP REPAIR CONFIGURATION FOR DAMAGED PRESTRESSED CONCRETE BRIDGE GIRDERS DUE TO VEHICLE COLLISION IMPACT

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ABSTRACT

This study presents a selected portion of a detailed experimental program performed to investigate the proper and most efficient configuration to apply Carbon Fiber Reinforcing Polymers (CFRP) to repair laterally damaged Prestressed Concrete (PSC) bridge girders. The flexural behavior of ten, 20-feet long, half-scaled AASHTO type II PSC girders is reported. The simulated vehicle collision impact damage was induced by saw cutting the girders' lower concrete corner and slicing through one of the pre-stressing strands. Then, to repair the damaged area, epoxy injections and other concrete repair materials and methods are used. Various configurations and multiple layers of CFRP (both longitudinal strips on girder soffit and U-wrapping) were applied to each girder to constitute the structural repair. The ten PSC girders were then tested in flexure until failure using a four point loading setup at the Florida Department of Transportation (FDOT) structures research lab. Measurements of the applied load, the deflection at five different locations, strains along the cross-section height at mid-span, and multiple strains longitudinally along the bottom soffit were recorded for each test performed. The Analysis of the results provided solid evidence for conclusions to the most efficient CFRP design schematic, the best configuration to avoid CFRP debonding, and other information that should be useful for properly repairing laterally damaged bridge girders.

Keywords: Assessment and Monitoring, FRP Reinforcement and Technologies, Research.

INTRODUCTION

Carbon fiber reinforcing polymers (CFRP) is becoming a more common option for repairing structural deficiencies and damaged structural members because of its appealing and unique properties. In the past, traditional repair techniques used have been expensive, labor intensive, and usually impede the ability for smooth traffic flow. CFRP has proven to be a more desirable solution providing an inexpensive and rapidly applicable repair method which maintains the original configuration and overhead clearance of the structure[1]. The emergence of CFRP applications as a preferred restoration solution has led to numerous studies documenting the behavior and design considerations for strengthening or retrofitting reinforced concrete (RC) members. Yet, in contrast to the abundant information available on RC research, data on the behavior of prestressed concrete (PSC) beams strengthened with CFRP laminates is limited[2]. Furthermore, of the limited studies available concerning PSC girders strengthened with CFRP, few address PSC members with pre-existing damaged repaired with CFRP[3-5]. The two primary sources of damage experienced by PSC bridge girders are corrosion and vehicle impacts[3]. Additionally, the combination of these two effects has been demonstrated to be significantly critical[6].

The majority of all bridge impacts are attributed to overheight vehicles colliding with girders of an overpass bridge. These overheight collisions are quite frequent, making efficient and cost effective repair options a major concern for transportation departments all over the nation. On average, in the United States between twenty-five and thirty-five bridges are damaged by colliding overheight vehicles every year, in each state[7]. Most of which are impacted multiple times. For example, in NY State thirty-two bridges have been impacted a total of five-hundred-ninety-five times since the mid 1990's[8]. The damaged caused by overheight vehicle collisions can be far too catastrophic for superficial repairs, but for less severe impacts, classifications for degrees of damage and applicable repair methods are available in *Kasan, 2009*[9], which was updated from *NCHRP Project 12-21*[10-11]. These classifications include acceptable damage for the use of non-prestressed CFRP laminates for repair and restoration. In addition, several field studies have demonstrated that impacted PSC bridge girders can be repaired using FRP materials after large losses of concrete cross-section and the rupture of a small number of prestressing strands[12-15].

However, research conducted in a laboratory setting to describe the overall behavior of impact damaged PSC girders is sparse and the documents present mixed results. *Di Ludovico et al. 2005, Green et al. 2004*, and *Klaiber et al. 1999* all report issues with premature debonding failures due to either inadequate transverse CFRP anchors or development lengths[16]. The common debonding problems reported, which result in early failures, is one of the main limiting factors for non-prestressed CFRP laminate repair designs. However, the current American reference for designing externally bonded CFRP laminate repairs, the ACI 440.2R-08, addresses some debonding behaviors as "areas that still require research"[17]. It continues to state that "more accurate methods of predicting debonding are still needed"[17]. It was the lack of reported laboratory testing, the frequency of overheight vehicle collisions, and the limitations of available design considerations that invoked this study.

The intent behind the research project, of which this paper is a portion of, was to conduct an extensive experimental analysis investigating the feasibility, performance, and most efficient configuration for repairing laterally damaged PSC bridge girders using bonded non-prestressed fabric CFRP laminates. Though it has been demonstrated that prestressed and post-tensioned CFRP repairs utilize the carbon fiber material more efficiently, the difficulties of implementation are more significant than the CFRP material savings[3]. The non-prestressed repair technique can be used for flexural strengthening[9-24] and shear strengthening[17&20-21]; though the performances are limited by the ability of the product to transfer stresses into the concrete substrate through that bond. Due to this limiting factor and the need to mitigate early debonding failures, specific points of investigation include: the effect of the U-wrappings on the strain developed in the longitudinal soffit laminates, the optimum configuration of the U-wrappings to mitigate debonding strains, the most beneficial level of strengthening (number of CFRP layers), and any design criteria needed for complete repair calculations. The information collected from previous research combined with the results from testing provided enough evidence to make beneficial conclusions regarding debonding predictions, the most efficient CFRP configuration to mitigate debonding, and the optimum number of layers for the level of capacity increased desired.

BACKGROUND

This paper presents the behavior and analysis of ten half-scaled AASHTO type II PSC girders with imposed simulated lateral damage and CFRP repair applications. These ten test beams represent only a portion of the entire research program. Prior to the testing of the included PSC specimens, a total of thirty-four RC beams with simulate lateral damage were also tested with various CFRP configurations and levels of strengthening. Similarly, following the reported ten PSC test girders, three more of identical sizes are being tested under fatigue loading to evaluate residual strengths and longevity. Both the thirty-four RC test samples and the thirteen half-scaled PSC serve as preliminary investigations for the ultimate testing of eight full-scale AASHTO type II girders; inclusively constituting an extensive investigation.

The RC samples previously tested were 8.0ft (2.44m) long with cross-section dimension of 5.5in by 10.0in (14cm by 25.4cm) and were reinforced using either #3 or #4 (grade 60) steel rebar. The simulated damage was imposed by cutting and bending one of the three longitudinal reinforcements at mid-span prior to casting; representing an impacted beam with a perfect concrete repair. They were then wrapped using various CFRP configurations with multiple levels of strengthening and tested until failure in flexure. A number of gages were instrumented for testing to measure the applied load, the corresponding deflections at multiple locations, the strains developed along the height of the beam, and the strains developed along the span of the beams' extreme bottom fiber. A detailed report of the experiment and basic results is documented in ElSafty and Graeff, 2011[22]. The RC testing and the resulting analysis was used as a stepping stone in order to understand the details of the problem and it has led to the following assumptions and considerations implemented when designing the repair application reported in this paper.

1. The ACI 440.2R-08 document provides adequate to conservative capacity estimations for repair designs, provided that transverse U-wrappings are used appropriately to mitigate early debonding failures.
2. The longitudinal CFRP reinforcement should extend as far as possible within the span and should terminate no closer than specified in the ACI 440.2R-08 for development length requirements.
3. If CFRP shear enhancements are not needed, the configuration of transverse U-wraps with spacings between them has shown to provide the same flexural benefits when compared to a fully wrapped beam.
4. Evenly spaced transverse U-wrappings provide the most efficient configuration for CFRP flexural enhancement repairs to mitigate debonding.
5. Without consideration for shear enhancements, the optimum spacing for transverse anchoring is theorized to be between a distance of $^{2}/_{3}d$ and $2d$, where d is the height of the AASHTO beam (or ½ to 1½ the height of the entire composite cross-section).
6. When repairing laterally damaged girders having a loss of steel reinforcements it is necessary to cover the damaged section with transverse strips to reduce the crack propagation in the critical region which initiates early debonding.

EXPERIMENTAL STUDY

The experimental testing presented in this paper included a total of ten half-scale AASHTO type II PSC girders having imposed simulated damage and applied CFRP laminates. The repaired girders varied in both CFRP configurations and levels of strengthening; which were decided upon based on the preliminary testing of the RC beams. Two of the ten beams represented the control samples, damaged and undamaged, receiving no CFRP. All ten of the PSC girders were tested in flexure until failure under a four point loading arrangement. Load measurements, deflection measurements, and strain measurements were recorded for all girders during their testing. Similarly, the modes of failure and observed behaviors were also documented during testing, all of which are discussed with the results and analysis.

TEST SPECIMENS

MATERIALS

The CFRP product decided upon for the research was a unidirectional carbon fiber fabric. It was used in conjunction with the saturant provided, which is an epoxy designed by the manufacturer specifically for the CFRP product. This product was also used for the repair applications for the preliminary testing of the RC beams mentioned earlier. A unidirectional fiber was desired for the research because of its affordability and efficiency. The specific unidirectional fiber product chosen was selected based on the properties and outcomes reported in previous research documents [9-23] and the local availability of products. All of the design values provided for the reinforcement properties of the materials used in the test specimens are listed in Tables 1 and 2.

Table 1. Properties of CFRP materials utilized in repair methods

CFRP Material Properties	Tensile Strength	Tensile Modulus	Ultimate Elongation	Density	Weight per Sq yd.	Nominal Thickness
Typical Dry Fiber Properties	550 ksi 3.79 GPa	33.4×10^6 psi 230 GPa	1.70%	0.063 lbs/in^3 1.74 g/cm^3	19oz. 644 g/m^2	N/A
*Composite Gross Laminate Properties	121 ksi 834 MPa	11.9×10^6 psi 82 GPa	0.85%	N/A	N/A	0.04 in. 1.0 mm
*Gross laminate design properties based on ACI 440 suggested guidelines will vary slightly						

Table 2. Properties of steel reinforcements used to design test specimens

Steel reinforcements	Dia.	Bar Area	grade	Young's Modulus	Weight	Yield Strength	Ultimate Strength
PS strand	0.4375 in 11.1 mm	0.115 in^2 96.9 mm^2	270	27.5x10^6 psi	0.367 lbs/ft	243,000 psi 1676 MPa	270,000 psi 1862 MPa
#3 bars	0.375 in 9.53 mm	0.11 in^2 71.3 mm^2	60	29x10^6 psi	0.376 lbs/ft	60,000 psi 345 N/mm^2	90,000 psi 621 N/mm^2
#4 bars	0.5 in 12.7 mm	0.2 in^2 126 mm^2	60	29x10^6 psi	0.683 lbs/ft	60,000 psi 345 N/mm^2	90,000 psi 621 N/mm^2

GIRDER DESIGN

The PSC girders tested were twenty feet long and had cross-sectional dimensions representing a half-scale model of an AASHTO type II girder. An additional decking four inches thick was also cast on top of the girders to simulate a bridge deck composite with the PSC girder. The concrete used for manufacturing the girders ended up having a compressive strength of approx. 10,000 psi (68.9 MPa) on the days of testing, though it was specified to be designed at 6,500 psi (44.82 MPa). A total of five low-relaxation grade 270 seven-wire prestressing strands were used to reinforce each girder. In addition, three non-prestressed rebar were provided in the girder flanges and two rebar in the deck topping. Half of the steel stirrups, provided for shear, extended vertically from the girder to the decking while the other half remained entirely in the girder. They were spaced every six inches alternating between the two height sizes, providing nearly the maximum amount of shear reinforcement for the cross-section. The girders were designed to be heavily reinforced in shear in order to avoid any premature failures which could jeopardize the test results and the investigations into the debonding issues. Fig. 1 presents a diagram of the cross-section and the reinforcements.

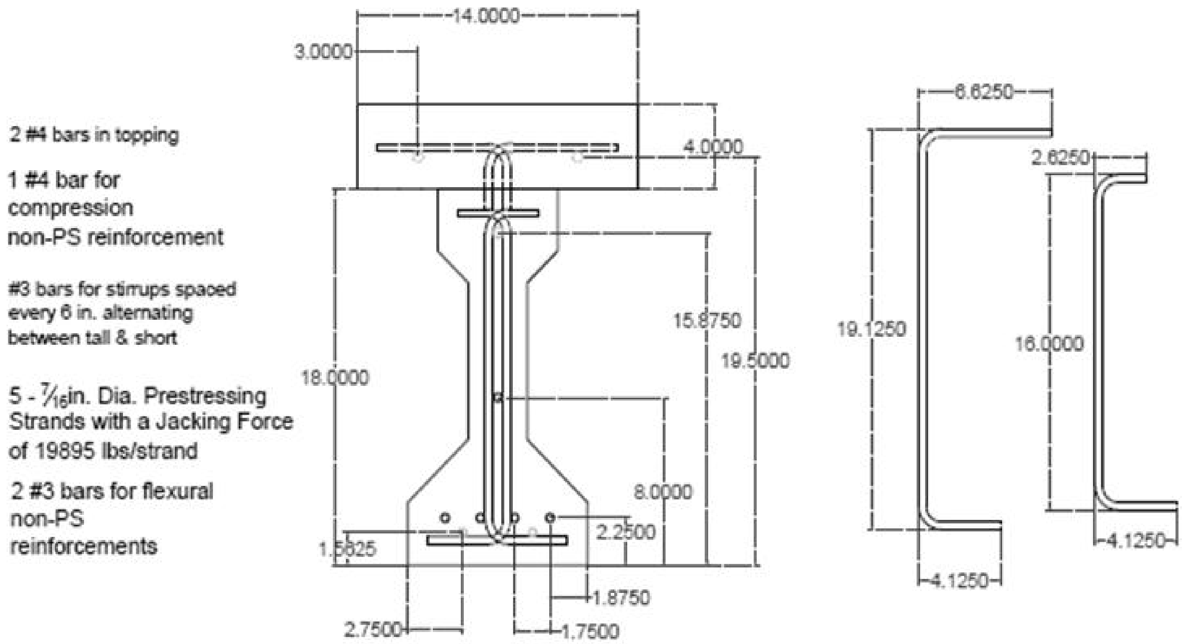

Fig. 1: PSC test girder cross-section and reinforcements

The lateral damage simulation was achieved by saw cutting through the concrete at the bottom flange of each girder and slicing through one of the prestressing strands. A schematic of this procedure and a picture of the resulting cut are shown in Fig 2. To repair the cut, the opening left from the saw was first roughened up using chisel tools to help improve the bonding area. The surface of the concrete exposed by the cut was then thoroughly cleaned with a water jet and pressurized air. The cleaned opening was filled with a high strength cementitous repair mortar and a high pressure epoxy injection procedure was performed after the mortar set. The procedure resulted in a near perfect repaired concrete cross-section.

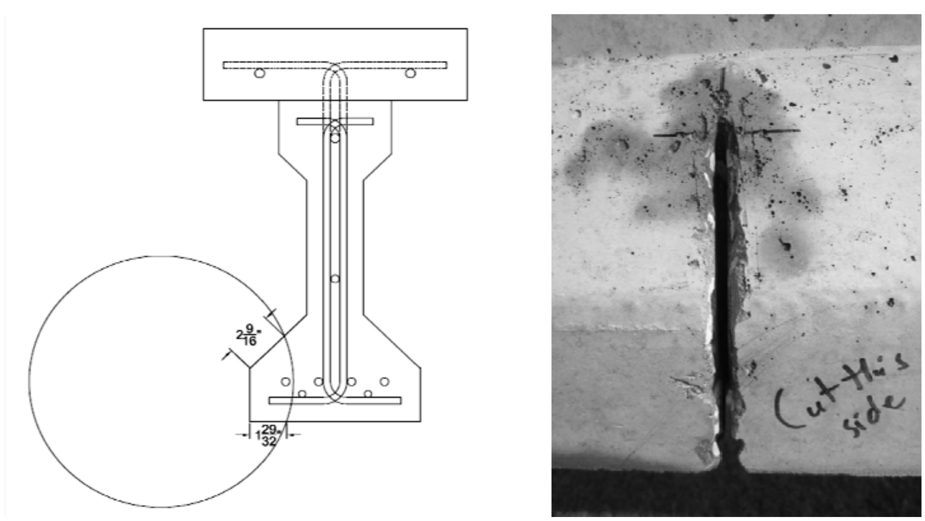

Fig. 2: (left) Diagram of saw cutting used to simulate damage in the girders; (right) Photo showing resulting cut in actual girder sample

CFRP CONFIGURATIONS

Multiple CFRP configurations and strengthening levels were used to repair the ten girders. The longitudinal strips were all eight inches wide and started at seventeen feet long, reducing six inches per each additional layer applied to each beam. The transverse U-wrappings were twelve inches wide and extended to the top of the web of the each girder. Fig. 3 & 4 show the CFRP configurations for the half-scaled AASHTO type II girders tested.

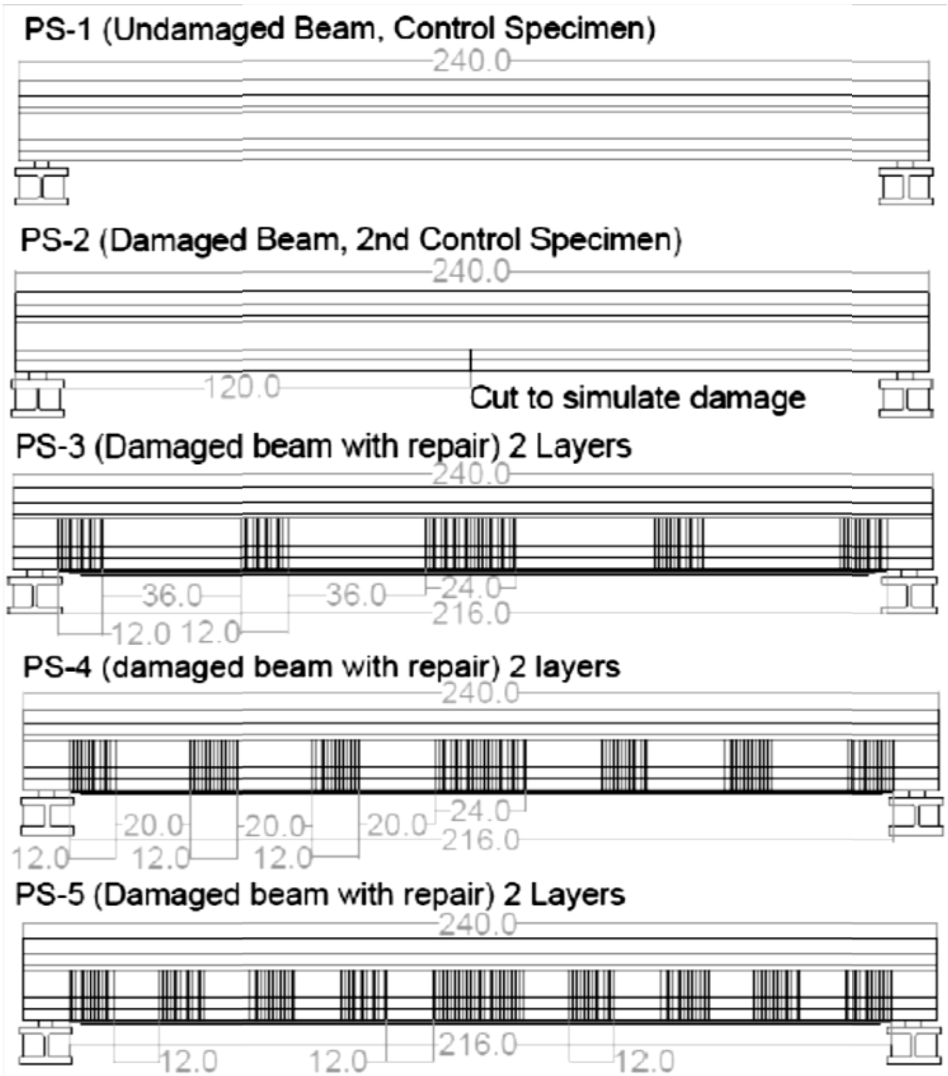

Fig. 3: CFRP configurations used for tested girders PS-1 to PS-5

In Fig. 3, the first girder (PS-1) is a control girder that represents an undamaged and unrepaired specimen. Similarly, the second girder (PS-2) is a damaged specimen which has received no CFRP repair (only concrete repair) representing the lower bound of the tested samples. The remaining girders had both simulated impact damage imposed on them and 2 layers of CFRP at various spacing to constitute the repair. The spacing between U-wrappings was set at a distance of twelve inches, twenty inches, or thirty-six inches.

Similarly, Fig. 4 displays the CFRP configurations for the remaining girders tested. The first three girders presented (PS-6 through PS-8) are damaged and repaired with 3 layers of CFRP at the girder soffit and U-wrappings at the same spacings of twelve inches, twenty inches, or thirty-six inches. The final two beams (PS-9 & PS-10) are fully wrapped girders (U-wrappings cover entire beam) using 2 layers of CFRP for the repairs (soffit and U-wrapping). However, the U-wrappings applied to PS-10 were overlapped by inch, whereas those applied to PS-9 were not. This was intended to investigate a simple question of continuity in the direction opposite to that of the fibers.

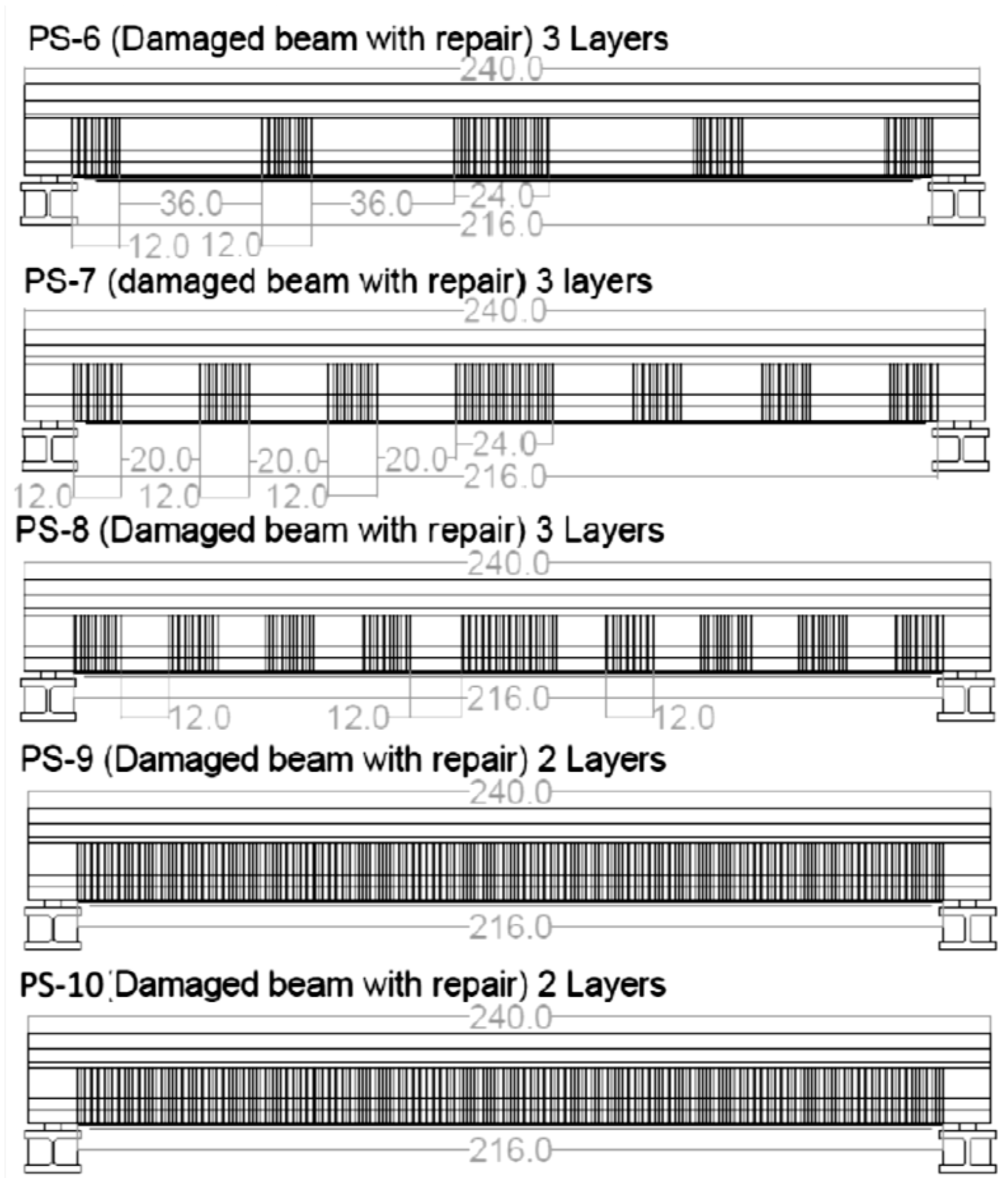

Fig. 4: CFRP configurations used for tested girders PS-6 to PS-10

TEST SETUP & INSTRUMENTATION

The girders were tested under four point loading using an 800 kip load actuator at the FDOT structures research lab. The 20-ft long PSC girders spanned nineteen feet between the centerlines of the bearing pads which rested on stationary supports. The girder loading was applied using a steel spreader beam resting on another set of two pads with a center to center distance of fifty inches. Fig. 5 shows one of the tested girders (PS-7) just prior to loading.

Fig. 5: Girder test setup diagram and photo of specimen during testing

Along with the structural arrangement, measurements were recorded through the set-up of many gage devices. Other than load measurements recorded by the actuator, the girders were also instrumented with six LVDT (linear variable differential transformer) deflection gages and up to twelve strain gages (30 mm long- 120 ohm). Two LVDT deflection gages were positioned at center span on each side of the girder, two LVDTs were placed at girder top surface above the support areas, and the remaining two LVDTs were placed at quarter points of the girder span. On each girder, four of the strain gages were placed along the height of the cross-section at mid-span and the remaining strain gages were distributed along the flexural tension side at various locations depending on the CFRP configuration. The general placements of all measurement devices mentioned are also shown in Fig. 6.

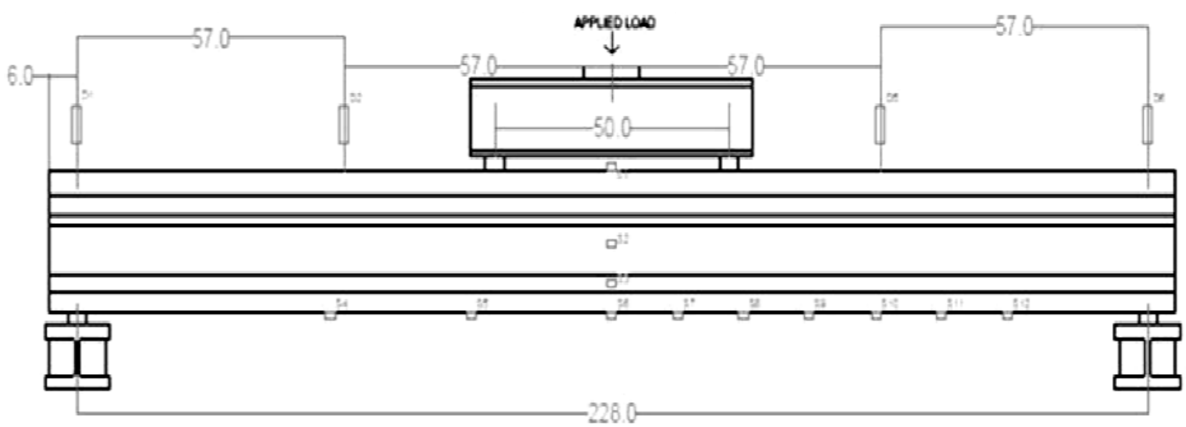

Fig. 6: Schematic of loading set-up and measurement device layouts

TESTING RESULTS & ANALYSIS

LOAD & DEFLECTION

The maximum loads reached, the corresponding deflections, and the increased capacity results from testing are listed in table 3. It is shown that a comparison between the failure load of control girder PS-2 (un-strengthened with CFRP) and repaired girders with 2 layers of CFRP shows that CFRP repair enhanced the flexural capacity by a range of 27.53% to 45.66% compared to control girder with one less strand. Also, for repaired girders with 3 layers of CFRP, increases in the flexural capacity were reported to range from 60.24% to 68.74% compared to control girder PS-2. An increase in the failure load of 24.85% was observed for the fully CFRP wrapped repaired girder compared to the un-strengthened control beam PS-2.

Table 3: Flexure test results for PSC girders

Girder designation	Max Load (kips)	Corresponding deflection (in.)	% increase compared to damaged girder PS-2	% increase compared to un-damaged girder PS-1
PS-1	75.87	6.94	22.60*	N/A
PS-2	61.88	5.38	0.00	-18.44**
PS-3	90.14	2.44	45.66	18.81
PS-4	84.75	2.14	36.94	11.70
PS-5	78.92	1.61	27.53	4.02
PS-6	100.91	2.39	63.07	33.01
PS-7	104.42	2.74	68.74	37.63
PS-8	99.16	2.29	60.24	30.70
PS-9	77.26	1.58	24.85	1.83
PS-10	87.68	2.14	41.69	15.57

* Increase of flexural capacity of PS-1 compared to that of PS-2

** Loss of flexural capacity of PS-1 due to strand cutting; a percentage of its original capacity

The load-deflection behaviors for each girder tested are presented in various comparisons in Fig. 7 through Fig. 12.

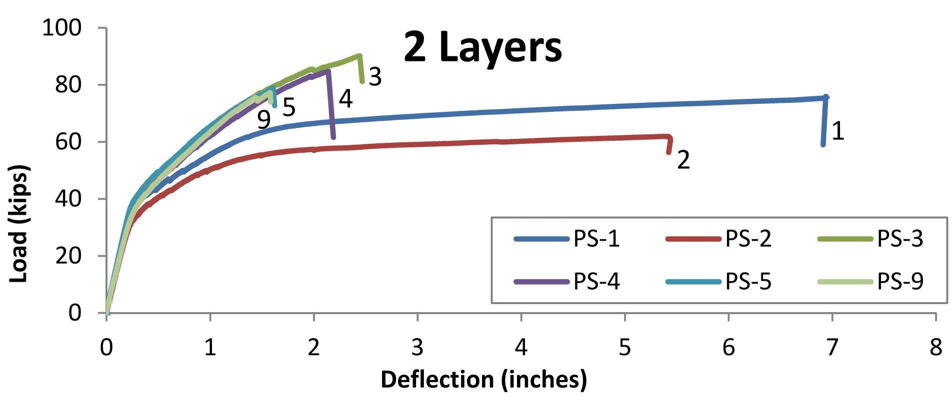

Fig. 7: Load vs. deflection for controls and girders with 2 layers of CFRP

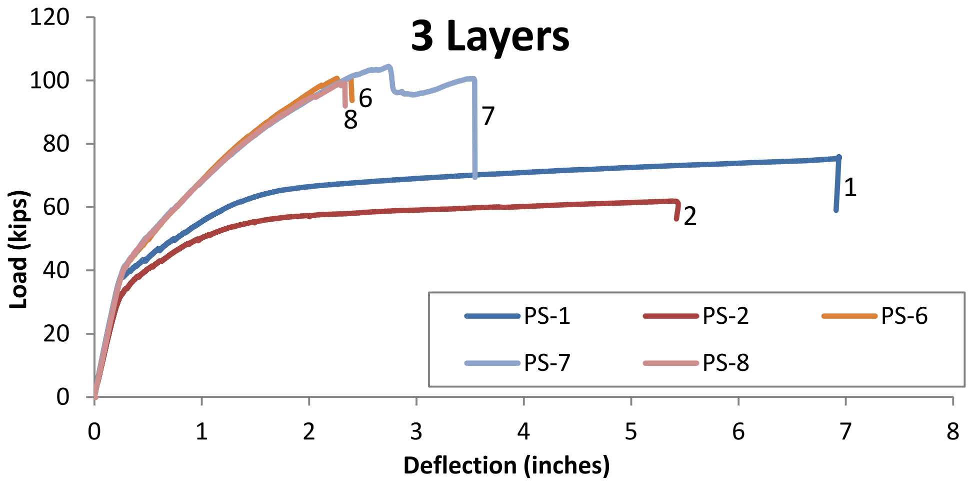

Fig. 8: Load vs. deflection for controls and girders with 3 layers of CFRP

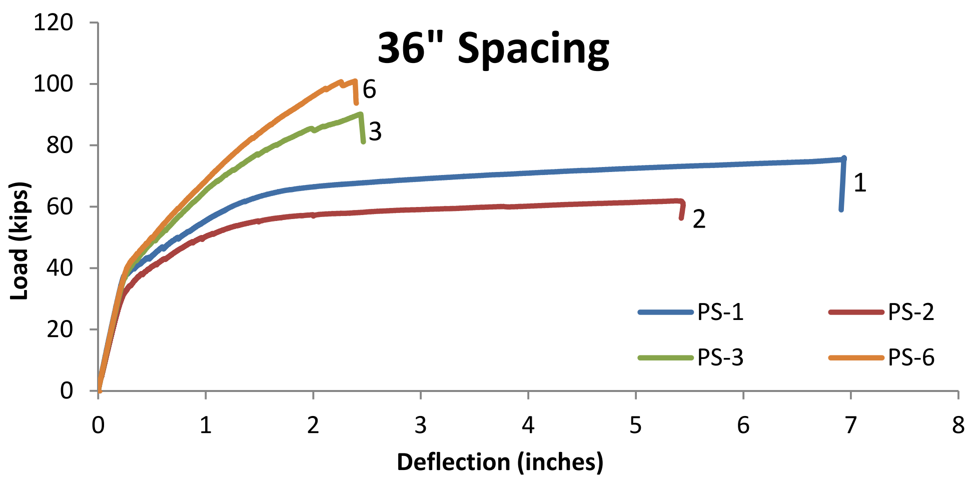

Fig. 9: Load vs. deflection for controls and 36" spacing configurations

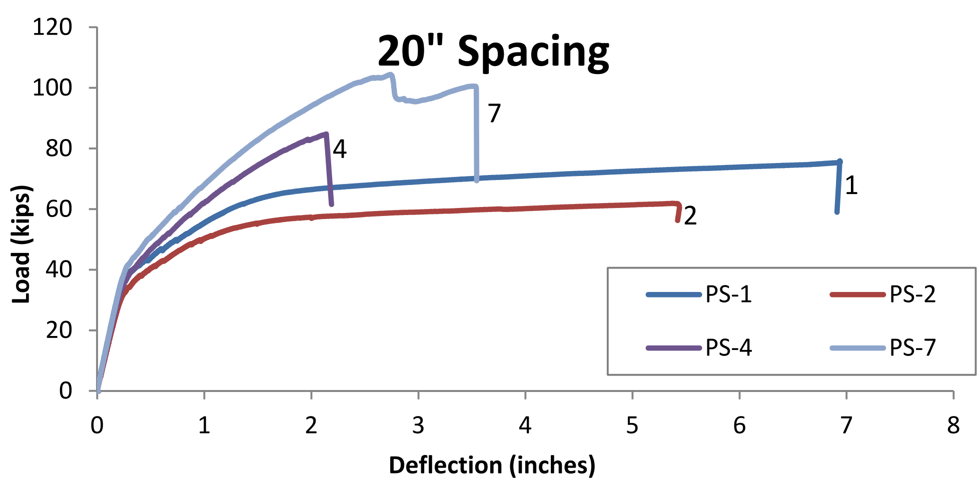

Fig. 10: Load vs. deflection for controls and 20" spacing configurations

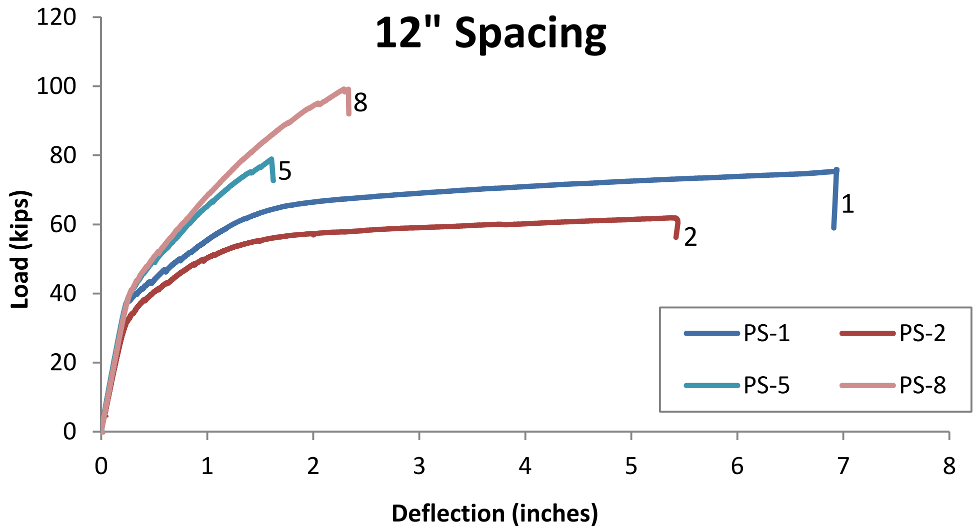

Fig. 11: Load vs. deflection for controls and 12” spacing configurations

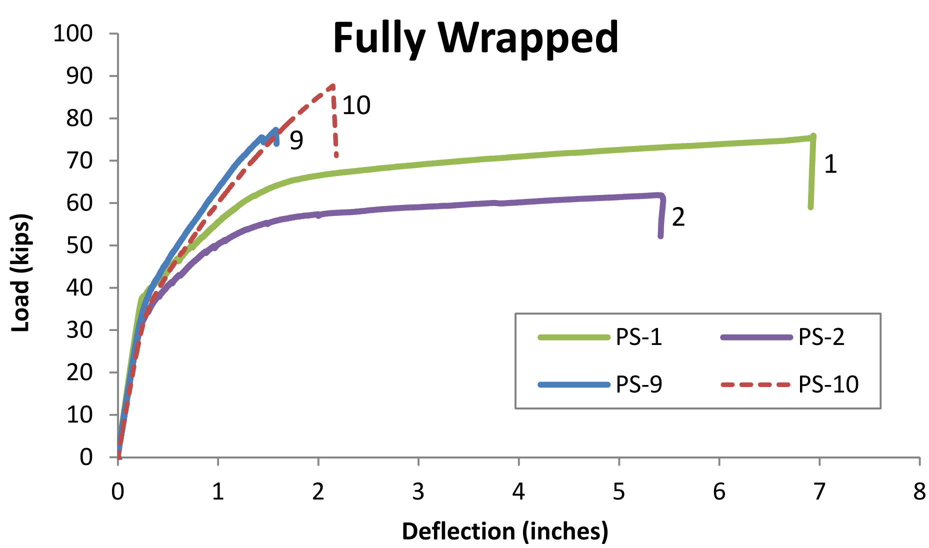

Fig. 12: Load vs. deflection for controls and fully wrapped configurations

As seen from the results, the damage and cutting of one of the prestressing strands (Girder PS-2) resulted in 18.44% loss in flexural capacity compared to the undamaged control girder (PS-1). The CFRP repair of a damaged girder, as shown in girders PS3 to PS9, restored the damaged girder’s capacity and exceeded the capacity of the undamaged control girder PS-1 by up to 37.63%. The results also show that U-shaped wrapping of CFRP laminates (Girders PS-3 to PS-8) enhanced the flexural capacity even if the U-wrapping was not continuously covering the entire girder side (not fully wrapped). By comparing the two fully wrapped beams, it is understood that overlapping transverse U-wrappings is needed to develop proper continuity; even in a direction perpendicular to the direction of the fibers.

STRAIN CHARACTERISTICS

The strains measured at a load level of 20 and 70 kips are presented in Fig. 12 & 13. Half of the span lengths of the symmetrical girders were instrumented with a multitude of strain gages while the other half of the span length had one strain gage. Therefore, the profiles shown depict much more detailed behavior of the girders' right side of the center span.

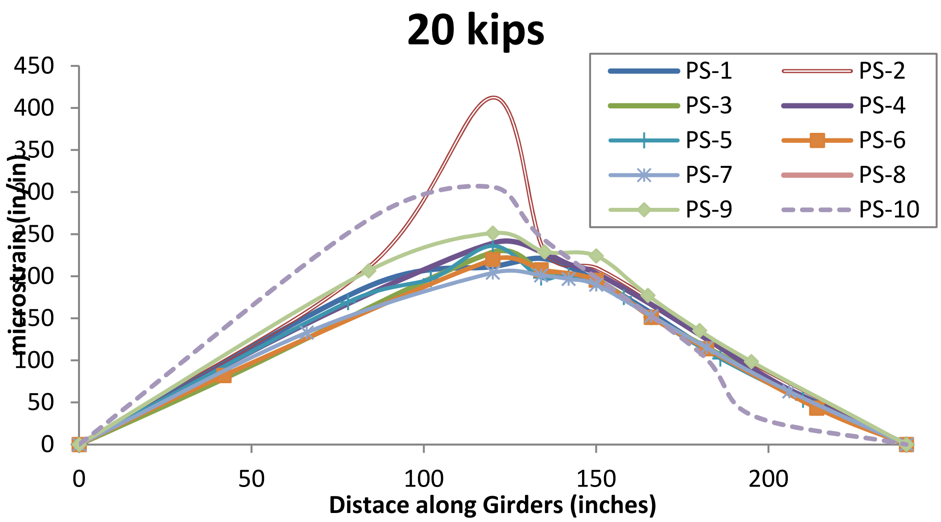

Fig. 12: Strain at extreme bottom fiber of girder soffit vs. length for all girders

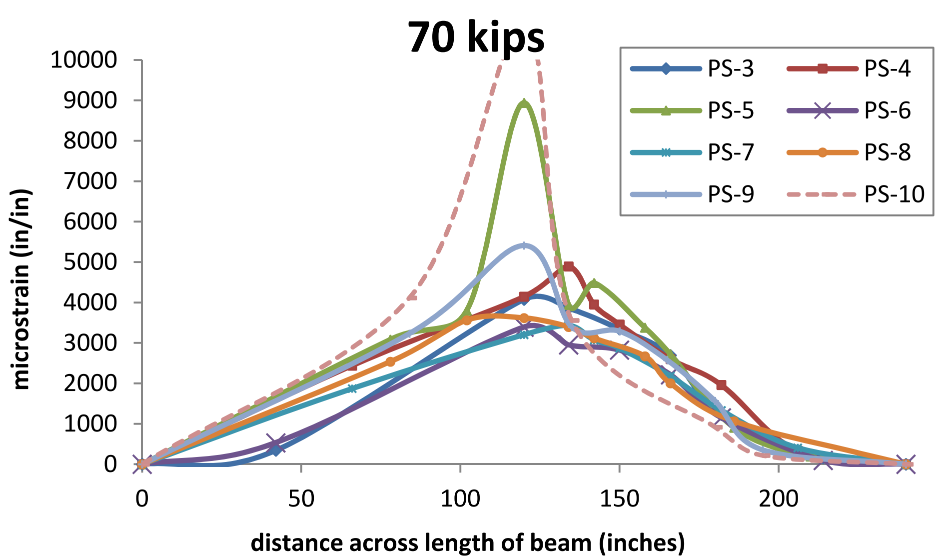

Fig. 13: Strain of CFRP at girder soffit vs. length for repaired girders

Though the comparisons are quite similar, one important thing to note is that when comparing the strain profiles with the configuration layouts it can be noticed that on the side of U-wrapping towards diminishing moment the strain in the longitudinal CFRP may spike briefly (more detailed measurements will be needed to verify).

FAILURE MODES

The modes of failure for the girders were also documented and reported. The control girders PS-1 and PS-2 experienced ductile flexural failure with excessive deflection and cracking. The repaired girders experienced CFRP rupture or localized debonding followed by rupture of CFRP; as shown in Fig. 11 & 12. Some repaired girders also experienced debonding of some of their U-wrappings, as shown in Fig. 11 (right).

Fig. 11: (left) Rupture of longitudinal CFRP; (right) Debonding of CFRP U-wrapping

Fig. 12: (left) Debonding of soffit CFRP; (right) Rupture of longitudinal & transverse CFRP

PREDICTION MODEL & ANALYSIS

The model used for design and capacity predictions was the ACI 440.2 R-08 document[17]. The following equations are major design equations from the design model and the resulting values for the designed repaired specimen in this research. This model identifies failure modes through the governing strain limitations due to either concrete crushing, FRP rupture, FRP debonding or prestressing steel rupture. The effective design strain for FRP rupture at a limit state controlled by concrete crushing can be calculated through Eq. 1.

$$\varepsilon_{fe} = \varepsilon_{cu}\left(\frac{d_f - c}{c}\right) - \varepsilon_{bi} \leq \varepsilon_{fd} \qquad \textbf{Eq. 1}$$

The values of 0.0173 in/in and 0.0136 in/in are calculated for the test specimen with two layers and three layers of longitudinal CFRP laminates respectfully. However, ε_{fd} is calculated as 0.0079 in/in for two layers and 0.0064 in/in for three layers thereby maintaining debonding as the limiting factor.

The ultimate limit state with rupture of the prestressing steel as the governing failure mode uses Eq. 2 & 3 for max strain calculations.

$$\varepsilon_{fe} = \left(\varepsilon_{pu} - \varepsilon_{pi}\right)\left(\frac{d_f - c}{d_p - c}\right) - \varepsilon_{bi} \leq \varepsilon_{fd} \qquad \textbf{Eq. 2}$$

$$\varepsilon_{pi} = \frac{P_e}{A_p E_p} + \frac{P_e}{A_c E_c}\left(1 + \frac{e^2}{r^2}\right) \qquad \textbf{Eq. 3}$$

A value of .0356 in/in is calculated here for both repair scenarios since the previous equations do not consider the changed design of the fibers. This value is still greater than that of $\varepsilon_{fd,}$ which will still control failure.

For FRP rupture or debonding as the ultimate limit state of failure, which is the case we have, ε_{fe} is chosen to be that of ε_{fd} thereby resulting in values of 110.17 ksi for two layers of CFRP and 89.96 ksi for three layers for f_{fe} in Eq. 4.

$$f_{fe} = E_f \varepsilon_{fe} \qquad \textbf{Eq. 4}$$

At this point a neutral axis is assumed and calculations proceed. For both cases it is assumed to be 1.69 inches. Yet, after computing Eq. 5 – 10, a new value for "c" of 3.28 inches and 4.03 inches was determined for the two layered and three layered repairs.

$$\varepsilon_{ps} = \varepsilon_{pe} + \frac{P_e}{A_c E_c}\left(1 + \frac{e^2}{r^2}\right) + \varepsilon_{pnet} \leq 0.035 \qquad \textbf{Eq. 5}$$

$$\varepsilon_{pnet} = \left(\varepsilon_{fe} - \varepsilon_{bi}\right)\left(\frac{d_p - c}{d_f - c}\right) \qquad \textbf{Eq. 6}$$

$$f_{ps} = \begin{cases} 28{,}500\varepsilon_{ps} & for\ \varepsilon_{ps} \leq 0.0086 \\ 270 - \frac{0.04}{\varepsilon_{ps} - 0.007} & for\ \varepsilon_{ps} > 0.0086 \end{cases} \quad \text{in.-lb units} \qquad \textbf{Eq. 7}$$

$$c = \frac{A_p f_{ps} + A_f f_{fe}}{\alpha_1 f'_c \beta_1 b} \qquad \textbf{Eq. 8}$$

The final values calculated for Eq. 5-7 were 0.0128 in/in, 0.0066 in/in, and 263.07 ksi respectfully for the girders repaired with two layers of CFRP. Similarly, the values of 0.0116 in/in, 0.0053 in/in, and 261.21 ksi were received for Eq. 5-7 using the three layered repair configurations. These values were all then used to calculate the theoretical ultimate moment capacity of each set of repaired girders.

The ultimate moment capacity of the repaired beams is calculated with the use of Eq. 9, which again was taken from the ACI 440.2 R-08 document.

$$M_n = A_p f_{ps}\left(d_p - \frac{\beta_1 c}{2}\right) + \psi_f A_f f_{fe}\left(d_f - \frac{\beta_1 c}{2}\right) \qquad \textbf{Eq. 9}$$

The resulting theoretical ultimate moment capacities were 295.61 kip-ft for the girders having two layers of CFRP applied, and 317.3 kip-ft for the girders having three layers applied. This translates into a predicted debonding failure load of 79.7 kips for two layered designs and 85.6 kips for the three layered designs. However, the intermediate U-wrappings used in the repair configurations are not account for in the ACI design provisions and alter the outcomes. Predictions for controls are made based on simple analysis using the Whitney stress block approach.

Table 4. Tested Values, Predictions, and Comparisons

Girder designation	Tested Max Load (kips)	Predicted Max Load (kips)	% increase or decrease compared to prediction
PS-1	75.87	81.9	Decrease 7.3%
PS-2	61.88	66.5	Decrease 6.9%
PS-3	90.14	79.7	Increased 13%
PS-4	84.75	79.7	Increased 6.3%
PS-5	78.92	79.7	Decreased 0.9%
PS-6	100.91	85.6	Increased 17.8%
PS-7	104.42	85.6	Increased 21.9%
PS-8	99.16	85.6	Increased15.8%
PS-9	77.26	79.7	Decreased 3.1%
PS-10	87.68	79.7	Increased 10.0%

CONCLUSIONS

Of the ten PSC half-scale AASHTO type II girders, eight were repaired using CFRP repair applications and tested under static loading conditions until failure. After analyzing the results and behaviors of the specimens the following conclusions can be made:

1. The longitudinal CFRP strips applied to the girder soffit along with U-wrapping instead of full wrap proved to be an excellent repair alternative for damaged girders.
2. Different U-wrapping configurations with varied spacing have proven to significantly enhance the flexural capacity of damaged prestressed concrete girders and prevent premature debonding of longitudinal.
3. A comparison between the failure load of control girder (with cut strand and un-strengthened with CFRP) and repaired girders with 2 layers of CFRP shows that CFRP repair enhanced the flexural capacity by 27.53% to 45.66% compared to control girder (with cut strand and un-strengthened with CFRP).

4. For repaired girders with 3 layers of CFRP, increases in the flexural capacity were reported to range from 60.24% to 68.74% compared to control girder (with cut strand and un-strengthened with CFRP).
5. An increase in the failure load of 24.85% to 41.69% was observed for the fully CFRP wrapped repaired girders compared to the un-strengthened control girder.
6. The damage and cutting of one of the prestressing strands (Girder PS-2) resulted in 18.44% loss in flexural capacity compared to the undamaged control girder. The CFRP repair of the damaged girder restored its capacity and exceeded the capacity of the undamaged intact control girder with no cut strand by up to 37.63%.
7. Proper CFRP repair design in terms of the number of CFRP longitudinal layers and U-wrapping spacing could result in obtaining significant enhancement for the capacity and desired failure modes for the repaired girders.
8. Favorable failure modes of the repaired girders can be maintained using a CFRP repair configuration utilizing spacing between the U-wrappings to prevent undesirable modes of failure such as debonding of the longitudinal CFRP strips from the girder concrete soffit. If shear improvement is not needed for girders after enhancing their flexural capacity, a spacing of close to the depth of the composite girder can be applied for the U-wrap configuration design to constitute a safe CFRP repair.
9. Debonding of some U-wraps was experienced at high load levels after restoring the girder flexural capacity. Therefore, it is recommended that a secondary CFRP strip be applied in the longitudinal direction to anchor the top end of the U-wraps

ACKNOWLEDGEMENTS

The authors would like to thank the Florida Department of Transportation for their support and interest of this research. Also, appreciation is extended to the Gate Precast Concrete Company for their participation and donations in regards to this research. In particular, special thanks go out to Rodney Chamberlain, Sam Fallaha, David Wagner, and William Potter the engineers of the FDOT Structural research center in Tallahassee, Florida. We would also like to include a special thanks to the laboratory staff at the FDOT Structural Research center including Paul, Steve, Chris, Frank, Tony, Dave, and all of the OPS guys/gals.

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