FATIGUE TESTING OF HALF-SCALED AND FULL-SCALED AASHTO TYPE II BRIDGE GIRDERS LATERALLY DAMAGED AND REPAIRED USING CFRP LAMINATES

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ABSTRACT

It has become widely accepted to use carbon fiber reinforced polymers (CFRP) materials for repairing damaged prestressed concrete bridge girders due to accidental impacts by over-height vehicles. However, important aspects, such as fatigue in CFRP repaired prestressed girders, are yet to be adequately evaluated. Furthermore, investigations into fatigue behaviors under overloading conditions for CFRP repaired girders are even more limited. This paper describes the fatigue behavior of three full scale and three half-scale AASHTO type II bridge girders laterally damaged to simulate an over-height vehicle collision and repaired using CFRP applications. The load range selected for full scale girders was according to AASHTO LRFD guidelines. However, the load range for the half scales girders was used to replicate an overloading condition. Most of the literature reports virtually no degradation for CFRP repaired girders under normal fatigue conditions; yet, many concrete bridges need increased capacities. Another set of 10 half-scaled AASHTO type II girders were also tested under static flexure loading. The experimental testing addresses comparisons of the spacing between anchorage U-wrappings, levels of CFRP strengthening, and their behavior under fatigue overloading. The reported results can be used to properly evaluate the performance of CFRP repairs for laterally damaged bridge girders.

Keywords: CFRP, Repair, Fatigue, AASHTO, Collision, Damaged

INTRODUCTION

The Bridge maintenance agencies are routinely facing the problem of replacing or repairing prestressed concrete (PSC) bridge girders that have been damaged by over-height vehicles' lateral collision. Repairing the damaged girder using innovative CFRP techniques is becoming a more practical solution than replacement, when repair is feasible. Limited research has been performed on the assessment of impact damage and subsequent repair methods to PSC bridge girders. It was reported that although several impact-damaged PSC girders have been repaired in the field, a limited number of laboratory studies have been conducted (Anthony Miller et. al. 2006)[1]. In addition, the repair of impact damaged PSC girders using CFRP laminates has not been explored by many previous researchers.

Splicing the steel prestressing strands as a repair method was found to perform poorly during fatigue, and in many cases is unable to restore the ultimate strength of the girder[2,3]. There was a dire need to find a more innovative technique that could not only restore the ultimate strength capacity of the damaged girder, but withstand the repetitive service loadings of bridge girders. The use of fiber-reinforced polymer (FRP) is becoming a promising repair or strengthening alternative to traditional techniques (external post-tensioning and externally bonded steel plates) of both reinforced concrete (RC) and PSC structures[1]. The traditional repair techniques used have been expensive, labor intensive, and usually impede the ability for smooth traffic flow. The use of FRP can be very effective at increasing the structural capacity of PC members[4,5]. CFRP has also 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[6].

Data on the behavior of prestressed concrete (PSC) beams strengthened with CFRP laminates is limited[7]. Furthermore, few researches address PSC members with pre-existing damaged repaired with CFRP[8-10]. The two primary sources of damage experienced by PSC bridge girders are corrosion and vehicle impacts[8]. Additionally, the combination of these two effects has been demonstrated to be significantly critical[11].

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[12]. 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[13]. 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*[14], which was updated from *NCHRP Project 12-21*[15-16]. 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[17-20]. 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* reported issues with premature debonding failures due to either inadequate transverse CFRP anchors or development lengths[21].

The intent behind the research project was to conduct an extensive experimental analysis investigating the feasibility, performance, and most efficient configuration for repairing laterally damaged PSC bridge girders under fatigue and static loading using bonded non-prestressed fabric CFRP laminates.

EXPERIMENTAL STUDY

The experimental testing presented in this paper investigated the behavior and analysis of three full scale and three half scale AASHTO type II PSC girders with imposed simulated lateral damage and CFRP repair applications under fatigue loading. In addition ten half-scaled AASHTO type II PSC girders were also investigated under static loading. Following the testing of the ten ½ scale PSC girders in flexure under static loading, three identical girders were tested under fatigue loading to evaluate residual strengths and longevity. That served as a preliminary investigation for the testing of eight full-scale AASHTO type II girders; five of which under static loading and three under fatigue loading. This paper only reports the fatigue test data of the three full scale and three ½ scale girders.

The half-scale and full scale AASHTO type II PSC girders had an imposed simulated damage and applied CFRP laminates. The repaired girders varied in both CFRP configurations and levels of strengthening. Two of the ten beams represented the control samples, damaged and undamaged, receiving no CFRP. The full scale PSC girders were tested under fatigue loading for 2 million cycles of 2 Hz. Then, they were tested in flexure until failure under a four point loading arrangement. The half scale girders were also intended to be tested under fatigue for 2 million cycles but using a higher fatigue load range. Yet the ½ scale girders failed prematurely at less than 1 million cycles. 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. 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[13-28]. 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 x 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 x 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 prestressing steel used in the 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

DESIGN OF FULL SCALE AND HALF SCALE GIRDERS

For the full scale AASHTO Type II girders, the overall length was 40 ft and the deck was 8 inches. For the ½ scale prestressed girders, the PSC girders tested were twenty feet long and had cross-sectional dimensions representing a half-scale model of an AASHTO type II girder, as shown Figure 1. 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 the girders had a compressive strength of approx. 10,000 psi (68.9 MPa) on the days of testing. 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. 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.

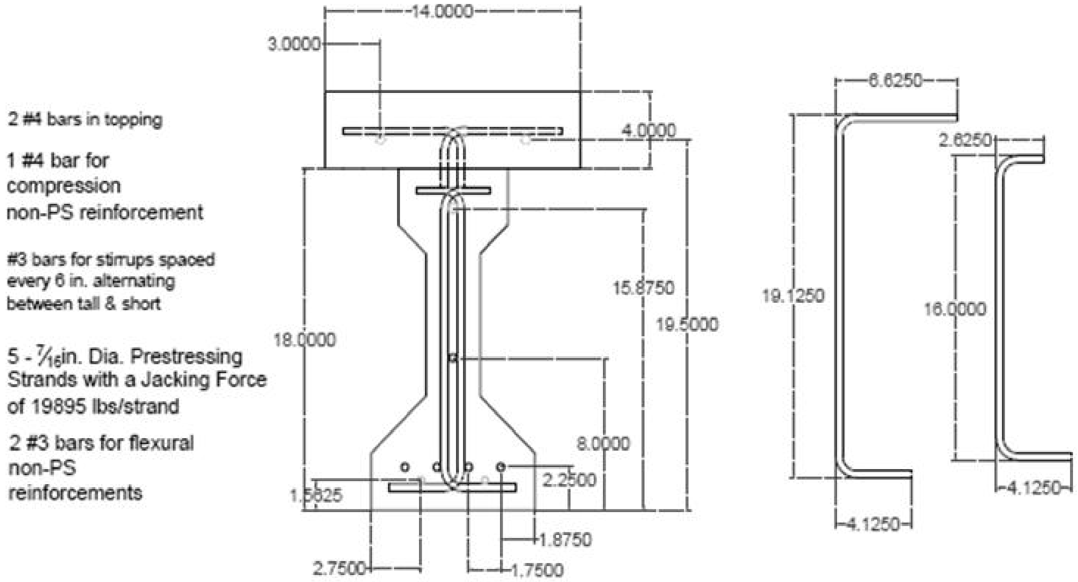

Fig. 1: Half Scale PSC test girder cross-section and reinforcements

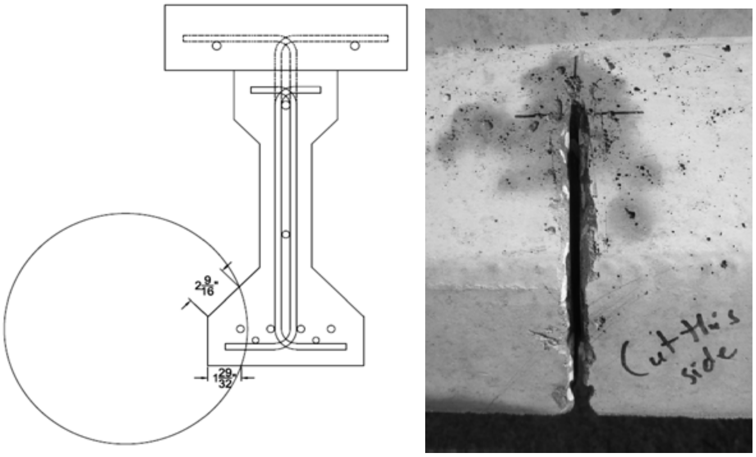

Fig. 2: Saw cutting to simulate damage

CFRP CONFIGURATIONS

Multiple CFRP configurations and strengthening levels were used to repair the full scale and ½ scale AASHTO Type II girders. For the ½ scale 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. Figures 5 and 6 show the CFRP configurations for the fatigue specimens; half-scale and full scale AASHTO type II girders; respectively.

The first girder (PSC-7) is a control girder that represents an undamaged and unrepaired specimen. Similarly, the second girder (PSC-8) 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.

The first three girders presented (PSC-12 through PSC-14) 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 (PSC-15 & PSC-16) 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 PSC-16 were overlapped by inch, whereas those applied to PSC-15 were not overlapped. This was intended to investigate a simple question of continuity in the direction opposite to that of the fibers.

The three best performing repairs from the initial ten half-scaled girders that were chosen for fatigue testing were both the 2 layer and 3 layer repairs with 20 inches spacing and the 2 layer with 36 inches spacing. These configurations were recreated exactly, maintaining the 8in. wide longitudinal laminates which started at 17ft. while reduced 6in. per each additional layer applied and the 12 inches wide transverse U-wrappings which extended to the top of the web of each girder. Figure 3 shows the CFRP configurations for ½ scale girders.

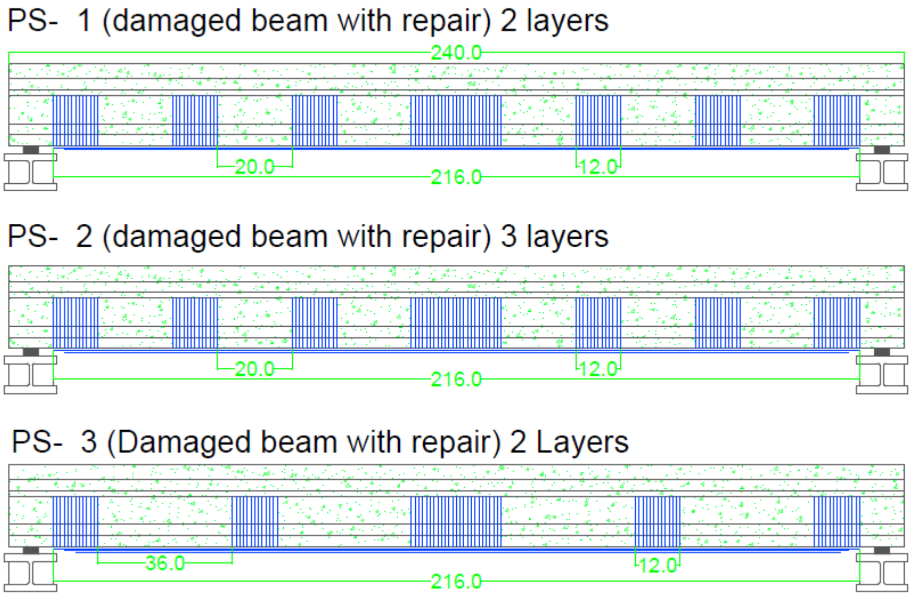

Fig. 3: CFRP repair configuration layout for half-scaled PSC girders tested in fatigue

The full scale AASHTO Type II girders are shown in the Figure 4.

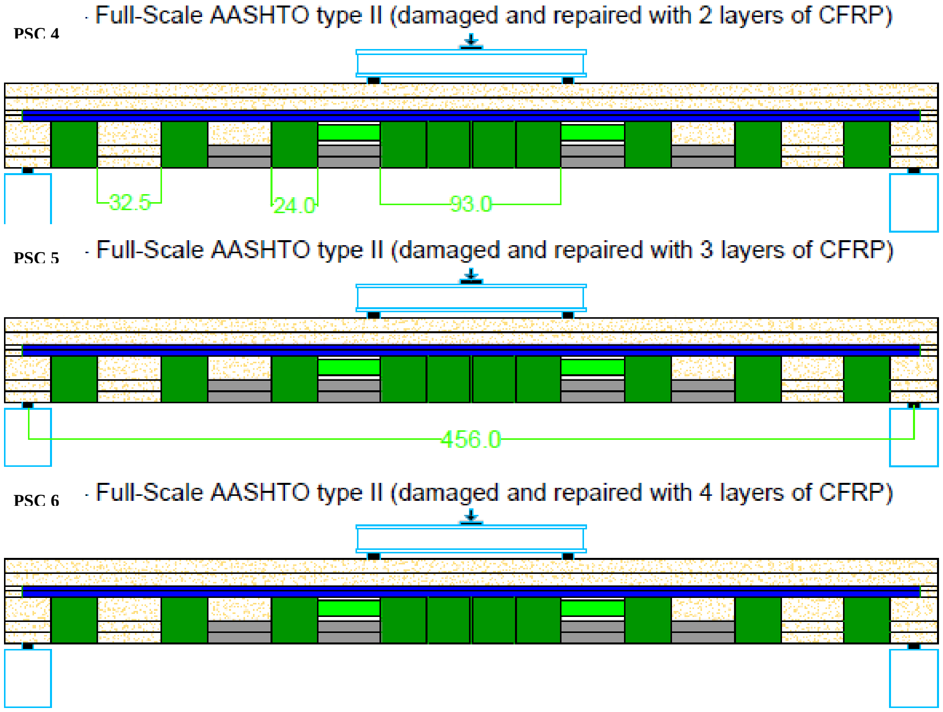

Fig. 4: CFRP repair configuration for Full-scaled PSC girders tested in fatigue

TEST SETUP & INSTRUMENTATION

The ½ scale PSC girders were tested in flexure 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 just prior to loading. However, the full-scale prestressed test specimens were 40ft. long AASHTO type II girders.

Fig. 5: Half Scale girder test setup during testing

Measurements were recorded through the set-up of many gage devices. Load and deflection measurements were recorded by the actuator. Also, the girders were 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 Figures 6 and 7 for ½ scale and full scale girders.

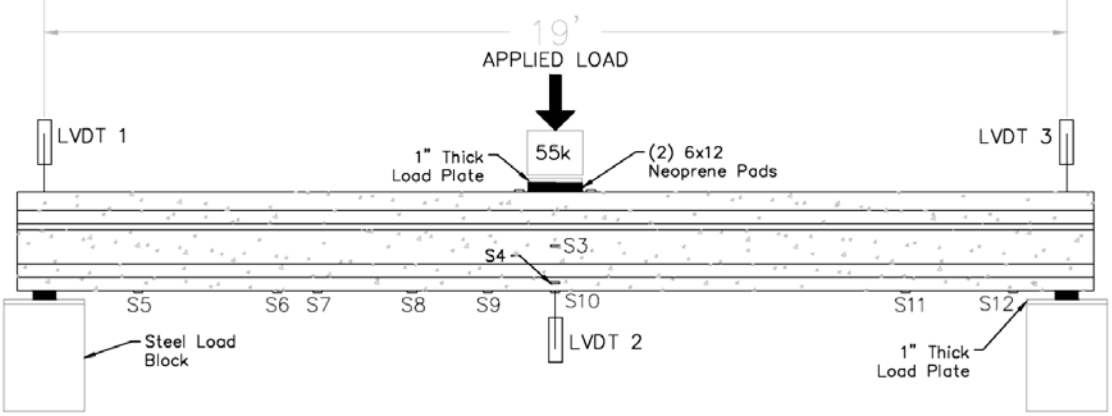

Fig. 6: Fatigue loading setup arrangement for half-scaled AASHTO PSC girders

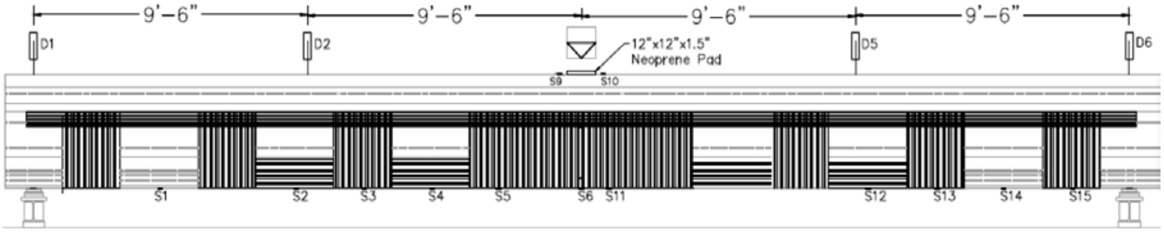

Fig. 7: Fatigue loading setup arrangement for full-scaled AASHTO PSC girders

TESTING RESULTS & ANALYSIS

LOAD & DEFLECTION

For the static testing of half scale beams, the maximum loads reached, the corresponding deflections, and the increased capacity results are listed in tables 3 and 4. It is shown that a comparison between the failure load of control girder PS-8 (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-8. 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-8.

Table 3: Flexure test results for PSC girders

Girder designation	Max Load (kips)	Corresponding deflection (in.)	% increase compared to damaged girder PSC-8	% increase compared to un-damaged girder PSC-7
PSC-7	75.87	6.94	22.60*	N/A
PSC-8	61.88	5.38	0.00	-18.44**
PSC-9	90.14	2.44	45.66	18.81
PSC-10	84.75	2.14	36.94	11.70
PSC-11	78.92	1.61	27.53	4.02
PSC-12	100.91	2.39	63.07	33.01
PSC-13	104.42	2.74	68.74	37.63
PSC-14	99.16	2.29	60.24	30.70
PSC-15	77.26	1.58	24.85	1.83
PSC-16	87.68	2.14	41.69	15.57

* Increase of flexural capacity of PSC-7 compared to that of PSC-8

** Loss of flexural capacity of PSC-7 due to strand cutting; a percentage of its original capacity

Table 4. Tested Values, Predictions, and Comparisons

Girder designation	Tested Max Load (kips)	Predicted Max Load (kips)	% increase or decrease compared to prediction
PSC-7	75.87	81.9	Decrease 7.3%
PSC-8	61.88	66.5	Decrease 6.9%
PSC-9	90.14	79.7	Increased 13%
PSC-10	84.75	79.7	Increased 6.3%
PSC-11	78.92	79.7	Decreased 0.9%
PSC-12	100.91	85.6	Increased 17.8%
PSC-13	104.42	85.6	Increased 21.9%
PSC-14	99.16	85.6	Increased15.8%
PSC-15	77.26	79.7	Decreased 3.1%
PSC-16	87.68	79.7	Increased 10.0%

As seen from the results, the damage and cutting of one of the prestressing strands (Girder PSC-8) resulted in 18.44% loss in flexural capacity compared to the undamaged control girder (PSC-7). The CFRP repair of a damaged girder, as shown in girders PSC-10 to PSC-15, 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 PSC-10 to PSC-14) 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.

Figures 8 – 10 show the fatigue behavior for half scale girders. The range of fatigue loading was much higher than that required by AASHTO LRFD to simulate overloading conditions. The ½ scale girders only survived less than 1 million cycles of fatigue loading at 3 Hz, with a fatigue load range of 10 kips to 35 kips.

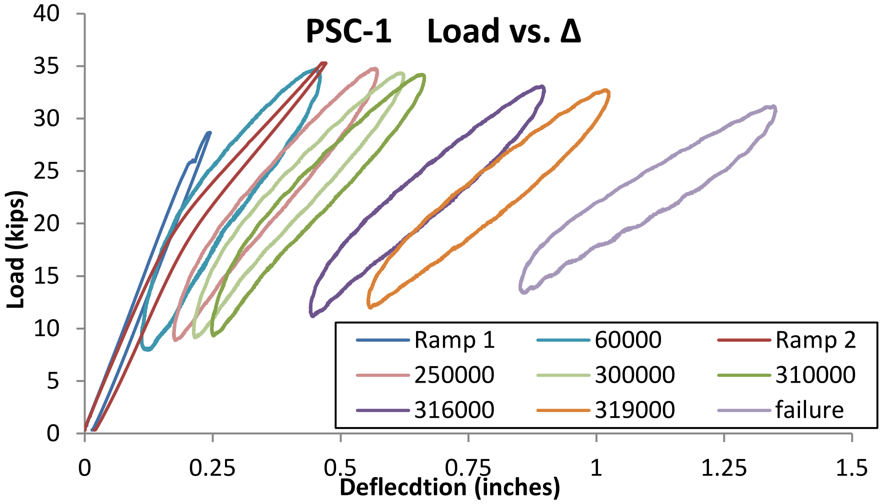

Fig. 8: Fatigue Behavior and Degradation until Failure for Girder PSC-1

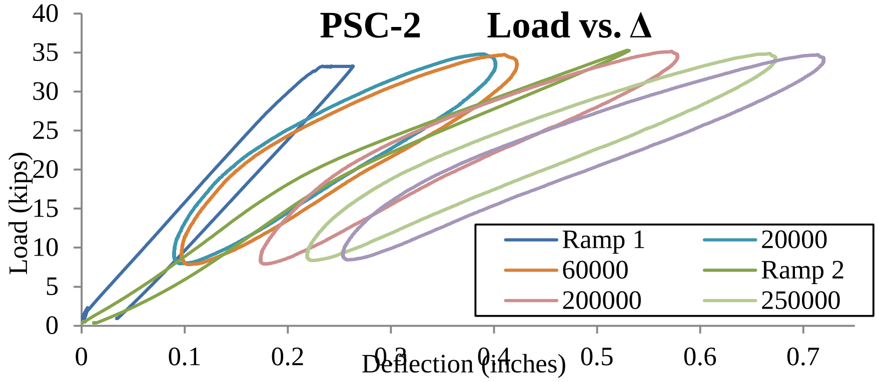

Fig. 9: Fatigue Behavior and Degradation until Failure for Girder PSC-2

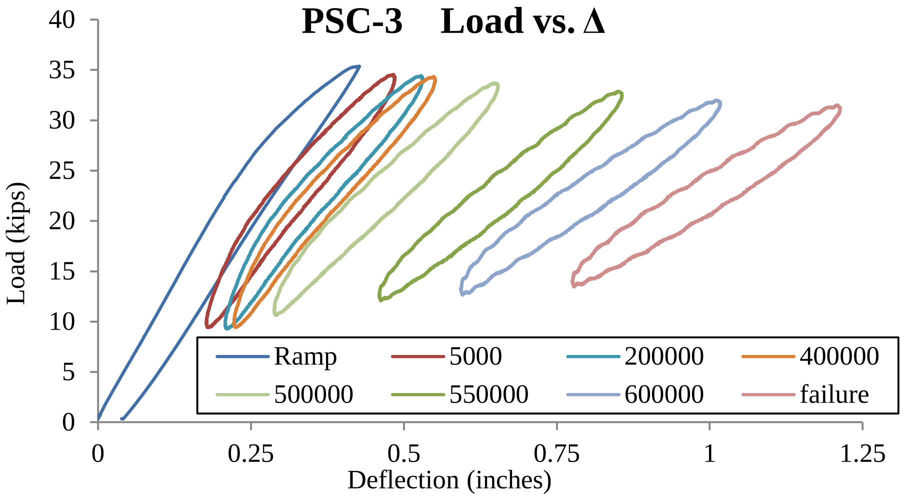

Fig. 10: Fatigue Behavior and Degradation until Failure for Girder PSC-3

Figures 11 – 22 represent the behavior of the full scale girders under fatigue loading. The girders survived 2 million cycles with 2 Hz without showing a noticeable degradation. The load range was 20 kips to 45 kips, according to AASHTO LRFD. However, PSC-3 was subjected to higher load range of 25 kips to 50 kips. It also survived the 2 million cycles.

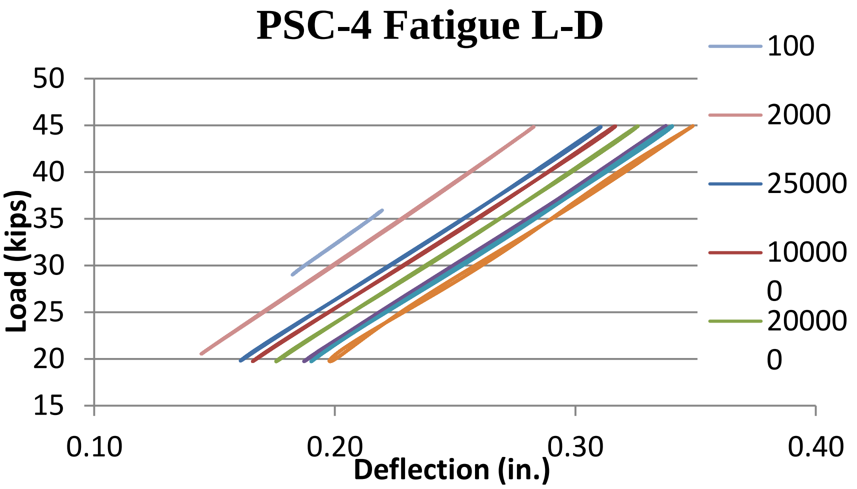

Fig. 11: Load Deflection of PSC-4

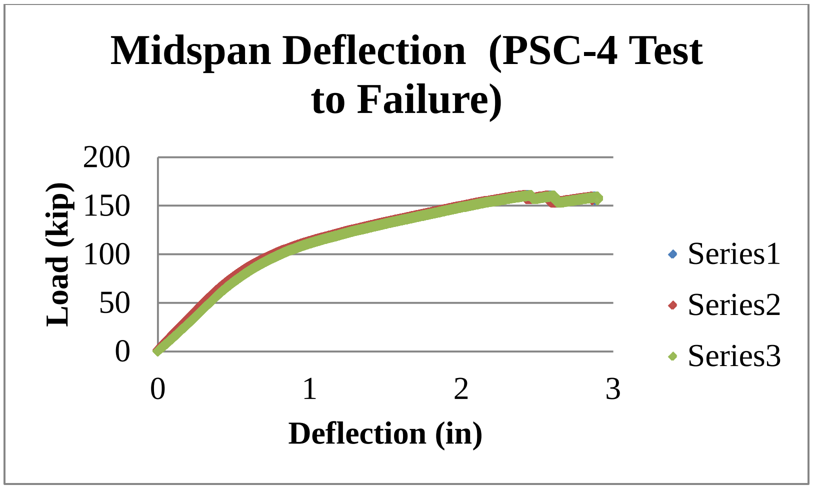

Fig. 12: Load Deflection of PSC-4 at static failure

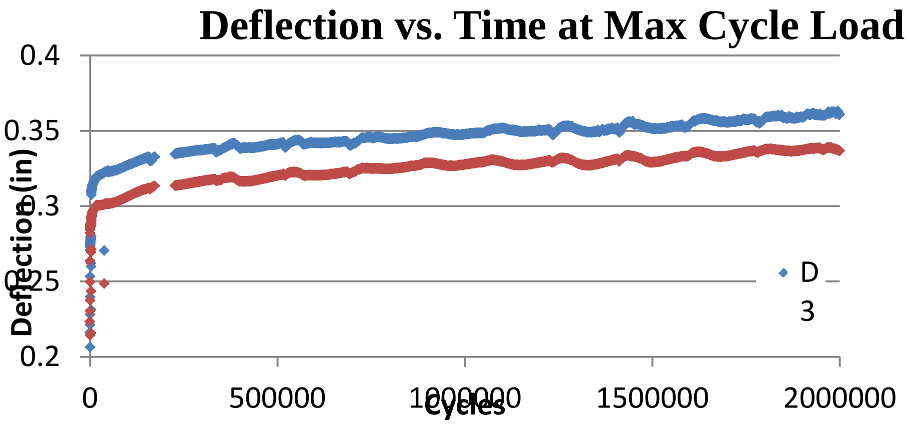

Fig. 13: Deflection cycles of PSC-4 at max cycle load

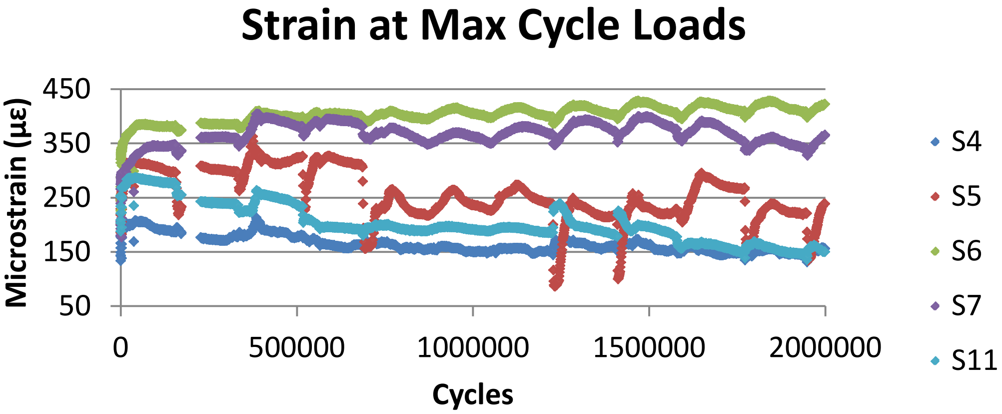

Fig. 14: Strain cycles of PSC-4 at max cycle load

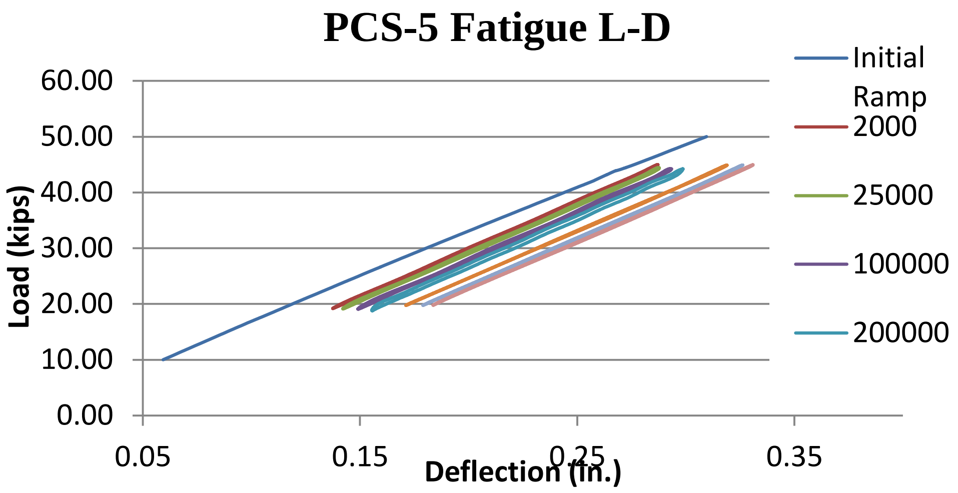

Fig. 15: Load Deflection of PSC-5

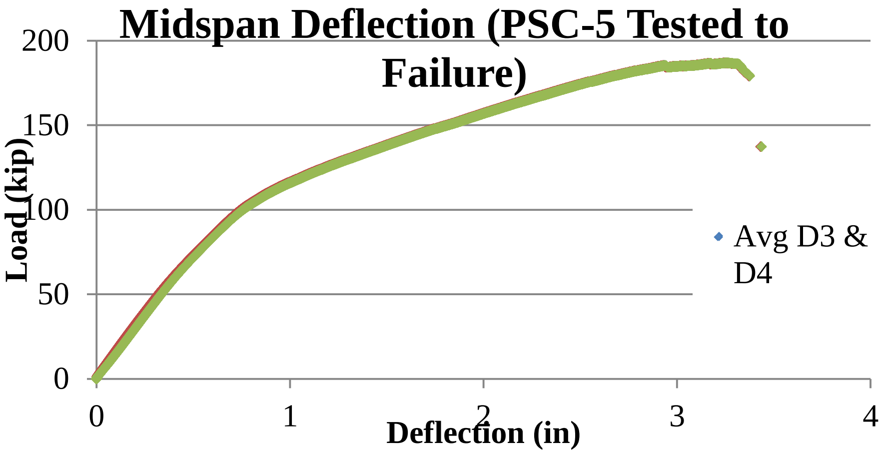

Fig. 16: Load Deflection of PSC-5 at static failure

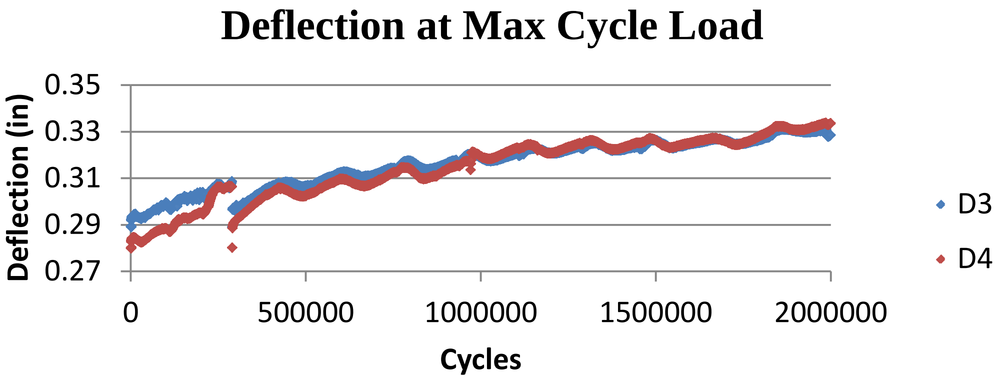

Fig. 17: Deflection cycles of PSC-5 at max cycle load

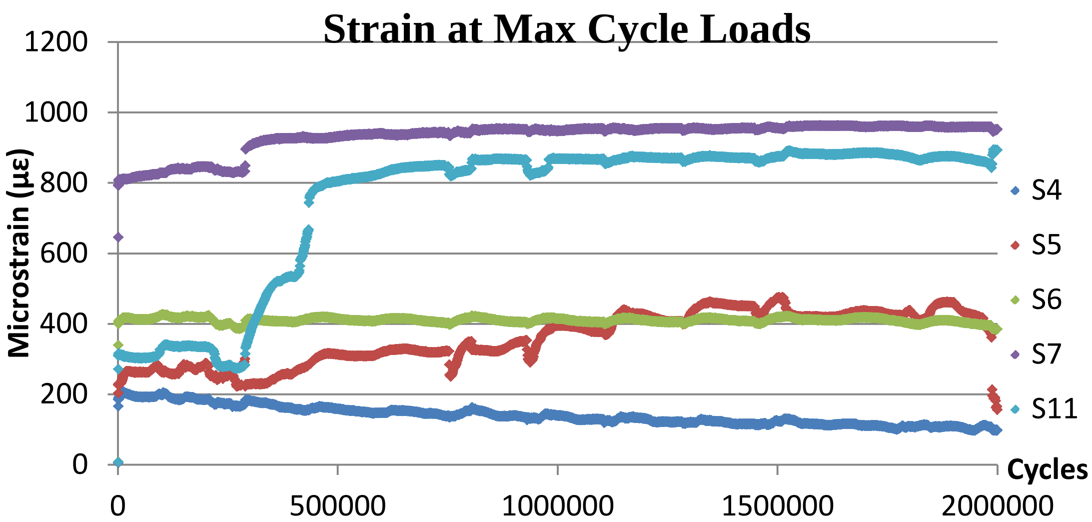

Fig. 18: Strain cycles of PSC-5 at min cycle load

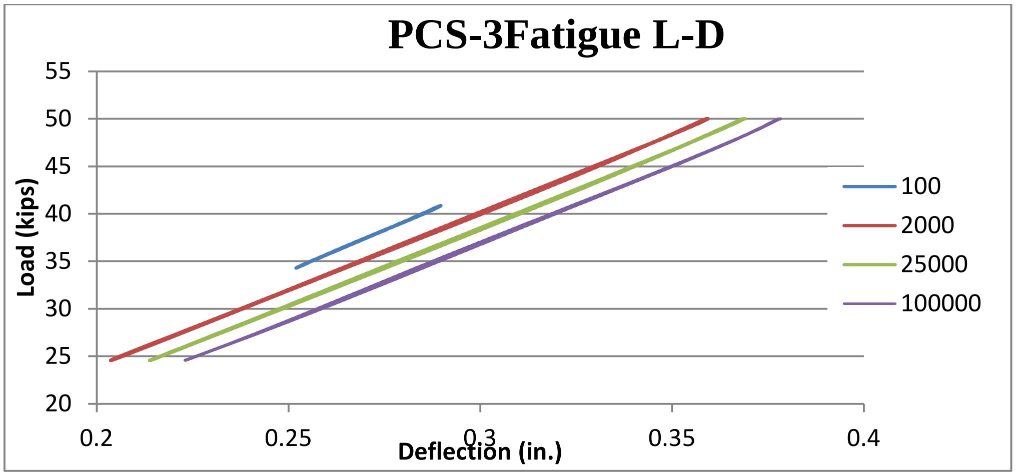

Fig. 19: Load Deflection of PSC-6

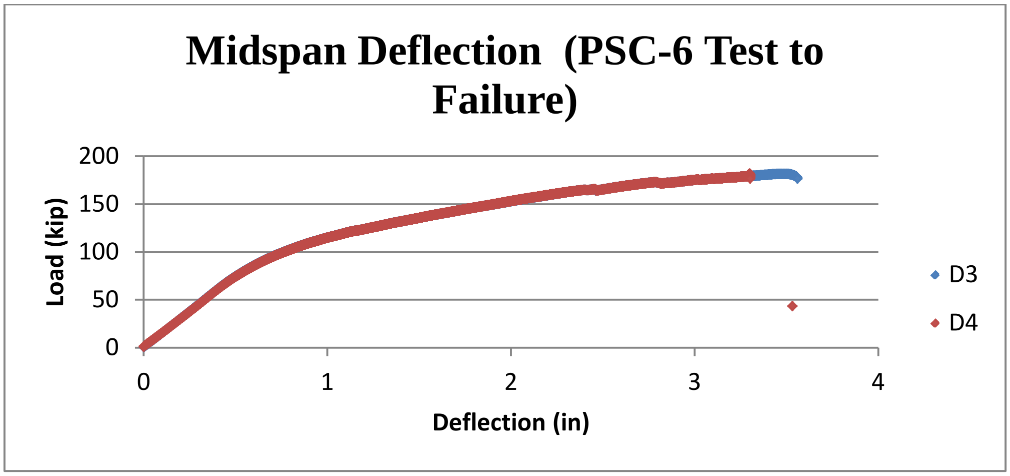

Fig. 20: Load Deflection of PSC-6 at static failure

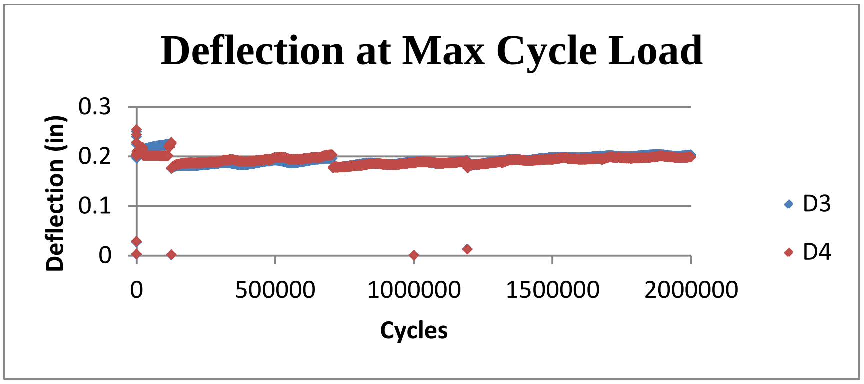

Fig. 21**:** Deflection cycles of PSC-6 at max cycle load

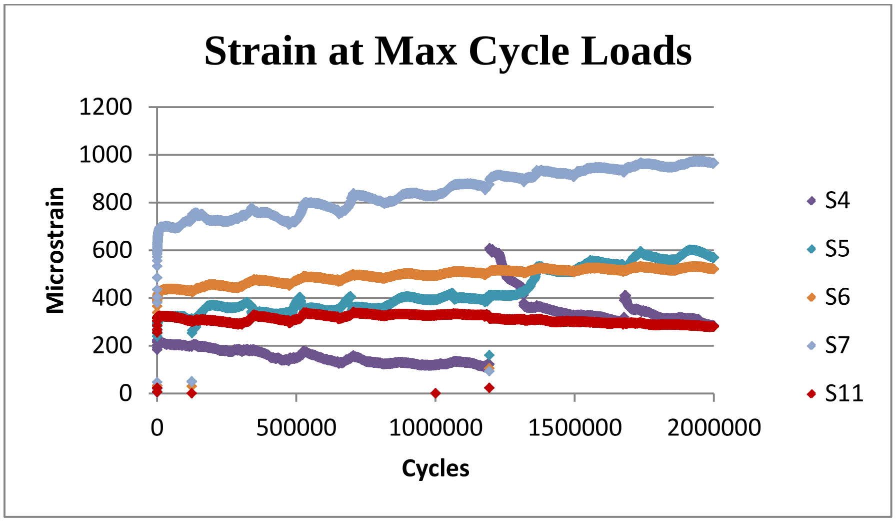

Fig. 22: Strain cycles of PSC-6 at min cycle load

Figures 23 to 26 show the full scale girders, the simulated damage, CFRP repair, and testing. Also, Figure 27 shows the girders after the final static failure load following fatigue cycles.

Fig. 23: Damaged full scale PSC beam

Fig. 24: Repair preparation

Fig. 25: CFRP pattern

Fig. 26: Test preparation

Fig. 27-a: failure shape mode of full scale PSC beam by static testing

Fig. 27-b: failure shape mode of full scale PSC beam by static testing

CONCLUSIONS

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. The U-wrapping had a comparable performance to full wrapping.
3. Evenly spaced transverse U-wrappings provide very efficient configuration for CFRP flexural enhancement repairs to mitigate debonding.
4. The original capacity of a damaged full scale bridge girder was restored and enhanced using CFRP repair applications.
5. If CFRP shear enhancements are not needed, the configuration of transverse U-wraps with spacing between them has shown to provide comparable flexural benefits when compared to a fully wrapped beam.
6. Without consideration for shear enhancements, the optimum spacing for transverse anchoring is theorized to be between a distance of ½ to 2/3d, where d is the height of the AASHTO beam (or ½ to 1 times the height of entire composite cross-section).
7. When repairing laterally damaged girders having a loss of steel reinforcements it is necessary to cover the damaged section with longitudinal and transverse strips to reduce the crack propagation in the critical region which initiates early debonding.
8. A comparison between the failure load of control ½ scale 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).
9. 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).
10. 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.
11. The damage and cutting of one of the prestressing strands (Girder PSC-8) 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%.
12. 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.
13. 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.
14. Damaged prestressed bridge girders repaired using non-prestressed fabric CFRP laminates can withstand over 2 million cycles of fatigue loading simulating service load conditions, with little degradation

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