STATIC AND FATIGUE PERFORMANCE OF 40 YEAR OLD PRESTRESSED CONCRETE GIRDERS STRENGTHENED WITH VARIOUS CFRP SYSTEMS

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ABSTRACT: The rapid growth in the volume and weight of heavy goods vehicles has resulted in a series of increases in the specified design loading for bridges in comparison to the standardized loadings adopted near the turn of the century. As a result, many bridges now in service do not conform to current design standards. Unless measurable reserves of strength exist, some form of strengthening or upgrade is required. This paper presents experimental results of a research program aimed at investigating the static and fatigue behavior of 40 year old prestressed concrete bridge girders strengthened with various carbon fiber reinforced polymers (CFRP) systems. Twelve 30 ft (9.14m) long, prestressed concrete C-Channel girders were tested: seven monotonically to failure and five under fatigue loading conditions. The girders were taken from a decommissioned bridge erected in 1961. Two girders were tested as control girders (one in static and one in fatigue), while the remaining ten girders were strengthened with various CFRP systems and tested in static and fatigue. The static test results show that the ultimate strength of prestressed girders can be increased by as much as 60% and that the crack width at ultimate and at the service load level can be reduced considerably using CFRP strengthening systems. Results from the fatigue tests show that prestressed girders strengthened with CFRP can withstand a 60% increase in live load for over 1 million cycles with little degradation. This paper provides a cost-effective analysis between the various CFRP strengthening techniques used in this investigation. Comparing the increase in the ultimate strength to the cost of the strengthening procedure, externally bonded CFRP sheets were found to be more cost-effective than the NSM technique.

Keywords: Prestressed concrete, CFRP, NSM, externally bonded, fatigue

INTRODUCTION

Research Objectives

The primary objective of the research described in this paper is to provide the prestressed/precast concrete industry as well as transportation agencies with a value-engineering and cost-effective analysis of strengthening techniques for prestressed concrete bridge girders using various FRP systems. The feasibility of using carbon fiber reinforced polymers (CFRP) strengthening systems to upgrade the load carrying capacity of 40 year old prestressed concrete bridges is investigated. Although there is an ever-expanding research database of reinforced concrete structures strengthened with different CFRP systems, information on various strengthening techniques for prestressed concrete structures is very limited. By conducting a cost-effectiveness analysis and examining the static and fatigue behavioral characteristics of strengthened girders and comparing them to girders tested as controls, a value engineering comparison can be made.

Background

Due to an aging bridge population and increasing volumes of traffic, departments of transportation across the United States have been faced with the difficult issue of bridge upgrading. Many bridges require improved pedestrian facilities or safer traffic barriers. Others require strength improvements to carry increased axle loads. Recent surveys have indicated that between 30 percent and 40 percent of all bridges in North America are either structurally or functionally deficient[1]. One of the biggest concerns to departments of transportation are short-span bridges in rural areas which have exceeded their design life but due to evolving industry demands may be required to carry loads above the initial design value. CFRP systems have the potential for cost-effective retrofitting of prestressed concrete bridges by increasing the load-carrying capacity thus extending their service life.

The use of externally bonded and NSM CFRP systems to repair or strengthen reinforced concrete beams in flexure has been well researched[2-5]. Takacs and Kanstad[6] showed that prestressed concrete girders could be strengthened with externally bonded CFRP plates to increase their ultimate flexural capacity. Reed and Peterman[7] showed that both flexural and shear capacities of 30 year-old damaged prestressed concrete girders could be substantially increased with externally bonded CFRP sheets. The investigated strengthening schemes were limited to one type of externally bonded CFRP sheets. Reed and Peterman also encouraged the use of CFRP U-wraps as shear reinforcing along the length of the girder in externally bonded systems to delay debonding failure. The use of NSM CFRP in prestressed concrete bridge decks was explored by Hassan and Rizkalla[3] and found to be a viable alternative to externally bonded systems. Nevertheless, the strengthening procedures were performed in the laboratory and did not simulate field conditions. In addition, the fatigue behavior was not studied. This paper provides a cost effectiveness and value engineering comparison of externally bonded and NSM strengthening techniques of 40 year old prestressed concrete girders under simulated field conditions.

Debonding of externally bonded FRP systems has been noted by many researchers often at the termination point of the FRP plate/sheet for members with a short span, and at the midspan section for long span members. Many models have been proposed to predict the failure loads of FRP strengthened reinforced concrete members due to plate-end debonding[8,9], yet the midspan debonding mechanism has not been as extensively researched[10]. Although one of the benefits of NSM FRP strengthening is to reduce the propensity for debonding failure, models to predict the debonding load have been characterized from earlier plate-based work[2].

The fatigue behavior of reinforced concrete beams strengthened with externally bonded CFRP systems has been investigated[11,12], yet no work has been done on prestressed concrete members strengthened with CFRP and tested in fatigue. The fatigue behavior of prestressed concrete is a topic that has been well researched, yet most of the studies were performed over 20 years ago[13,14]. Some of the main conclusions from this research is that (1) the failure of a prestressed concrete girder in fatigue will be due to rupture of the prestressing strand and (2) if the girder remains uncracked throughout its service life fatigue failure is very unlikely. The reason that failure is unlikely to occur for the uncracked girder is because the stress range, SR, in the prestressing is kept low, that is,

$$SR = \frac{f_{ps2} - f_{ps1}}{f_{pu}}$$

where

f_{ps2}, f_{ps1}	upper and lower stress range in prestressing subjected to cyclic loading conditions
f_{pu}	ultimate strength of the prestressing

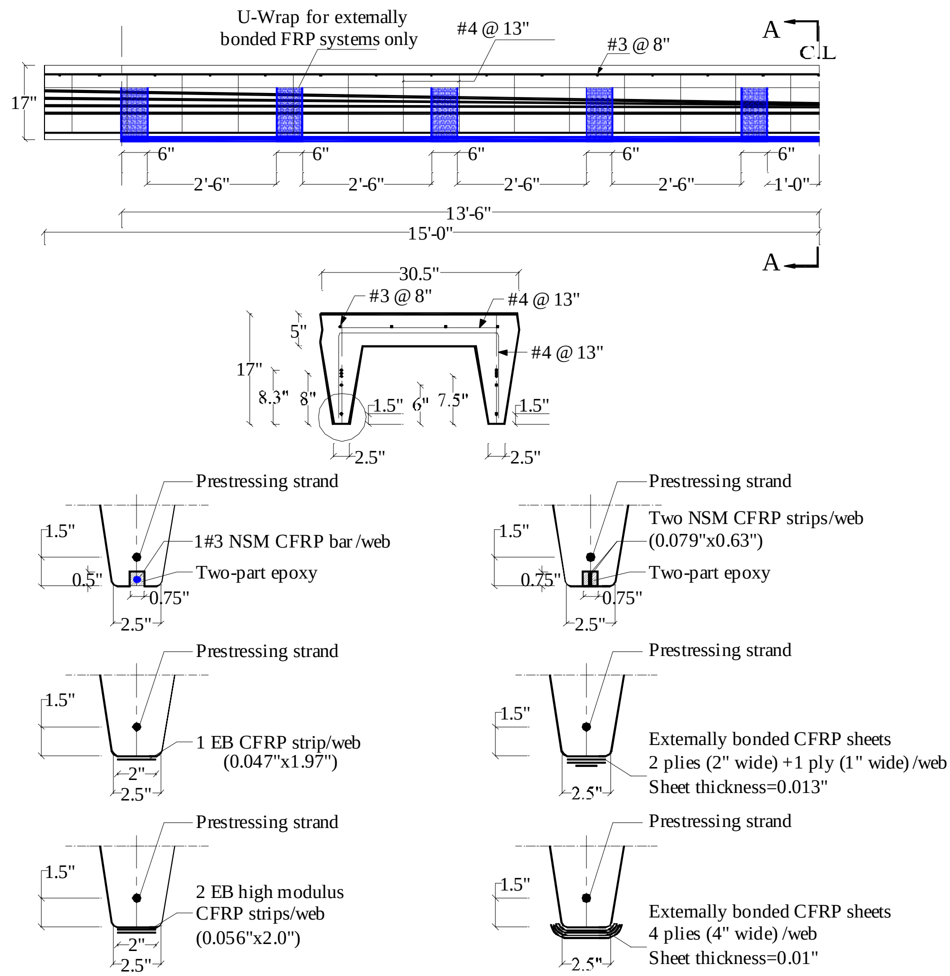
One of the most recent studies of prestressed concrete in fatigue, performed on decommissioned bridge girders, found that the stress range in the prestressing should be limited to 6 percent of f_{pu}[15].

The girders used in the research described in this paper were designed to remain uncracked during their service life and therefore immune to fatigue failures. Since the addition of CFRP strengthening systems could increase the loads above the cracking load, the fatigue behavior needs to be examined closely.

EXPERIMENTAL PROGRAM

Test Specimens

As part of an extensive research program sponsored by the North Carolina Department of Transportation, twelve 30 ft (9.14m) long prestressed concrete bridge girders were tested at the Constructed Facilities Laboratory at North Carolina State University. Two girders were tested as controls (without any strengthening) in static and fatigue (S0 and F0), while the remaining ten girders were strengthened with NSM bars, NSM strips and externally bonded strips and externally bonded sheets of different types. All twelve girders were C-Channel type prestressed concrete bridge girders (see Fig. 1). The girders were taken from a decommissioned bridge in Carteret County, NC, USA, which was erected in 1961. The girders were in fair to good condition upon delivery, with no visible flexural cracks. According to core samples tested per ASTM C42[16], the average compression strength of the concrete ranged from 8105 psi (55.9 MPa) to 11590 psi (80.8 MPa). Each girder was prestressed with ten 250 ksi (1725 MPa) seven-wire stress relieved, 7/16 in (11 mm) prestressing strands (five in each web) and had a 5 in (125 mm) deck with minimal reinforcing. The wearing surface was removed prior to delivery. The measured camber of the girders at midspan, due to prestressing, ranged from 1.25 to 1.5 in (40 mm).



Six different types of CFRP systems were applied to the two webs in each strengthened girder: two were near surface mounted (NSM) and four were externally bonded (EB). Four of these systems were tested in static and fatigue, and two were tested only in static. Eight girders were designed to achieve a 20 percent increase in ultimate strength with respect to the control girder. The ninth and tenth girders were designed to achieve a 60 percent increase in capacity. The first EB system, tested in static and fatigue, (S3 and F3) used one 2 in (50 mm) wide CFRP strip per web bonded using two-part epoxy. The second EB system (S4) used two 2 in (50 mm) plies and one 1 in (25 mm) ply of CFRP sheets per web bonded using a two-part epoxy matrix and a wet lay-up installation procedure. The third EB system (S5) used two high modulus CFRP strips per web bonded using two-part epoxy. The final EB system, tested in static and fatigue (S6 and F4), which was designed to achieve a 60 percent increase in strength, used four 4 in (100 mm) plies of CFRP sheets per web bonded using a two-part epoxy matrix and wet lay-up installation procedures. For all of the EB systems, 6 in (150 mm) wide U-wraps were provided at 3 ft (900 mm) spacing along the length of the girder to control the debonding mechanism. The first NSM system, tested in static and fatigue (S1 and F1) used one 3/8 in (10 mm) CFRP bar per web bonded using a two-part epoxy. The second NSM system, tested in static and fatigue (S2 and F2) used two 0.047 in x 1.97 in (2 mm x 16 mm) strips per web bonded using a two-part epoxy. The strips were bonded together prior to embedment using the same adhesive. The groove dimensions were 0.75 in by 0.75 in (19 mm by 19 mm) in both cases.

Strengthening was carried out according to the guidelines proposed by the National Cooperative Highway Research Program[17] for the repair and retrofit of concrete structures using FRP composites. To simulate field conditions, the girders were placed side-by-side, as they would be on a bridge, and placed on top of an outdoor steel substructure approximately 7 ft off the ground. The tensile strength, modulus of elasticity, and rupture strain for the CFRP materials were determined from coupon tests per ASTM D3039[18] and are reported in Table 1. Fig. 1 shows the reinforcement and strengthening details of the test specimens.

Table 1. Summarized static test results and material properties

Girder Designation	S0	S1	S2	S3	S4	S5	S6
Strengthening Technique	none	NSM Bars	NSM Strips	EB Strips	EB Sheets	EB Strips	EB Sheets
FRP Details	--	1 #3 per web	2 strips per web	1 strip per web	2.5 plies per web	2 strips per web	4 plies per web
FRP Shape	--	bar diameter = 0.362”	strips are 0.079” x 0.63”	strips are 0.047” x 1.97”	sheets are 0.057” x 2”	strips are 0.056” x 2”	sheets are 0.095” x 4”
A_{FRP} (in^2)	--	0.202	0.2	0.185	0.568	0.448	3.04
Ultimate Tensile Strength of FRP (ksi)	--	300	300	406	97	141.5	49.5
E_{FRP} (Msi)	--	18	19	23.9	7.18	55.25	6.74
FRP rupture strain	--	0.0167	0.0158	0.0170	0.0135	0.00256	0.00734
f'_c (psi)	10724	9667	6958	8901	7402	8347	9838
Ultimate load (kips)	33.2	40.8	40.7	39.6	36.7	29.1	53.1
% increase in capacity	--	22.9	22.6	19.3	10.5	none	60
Failure mode*	C	C	C	D	R	R	R

1 kip = 4.448 kN, 1in = 25.4 mm

* C = crushing of concrete, D = FRP debonding, R = rupture of FRP

Design of the Strengthened Girders

The design of the strengthened girders proceeded after testing the control girder under static loading conditions. The objective of the strengthening was to achieve a 20 percent increase in the ultimate load carrying capacity with respect to the control girder, except for S6 and F4 which were designed for a 60 percent increase in the ultimate load carrying capacity for further comparison. Each strengthened girder was designed using a cracked section analysis program, Response 2000[19]. For the design, the manufacturer's properties were used to model the FRP materials. The prestressing steel and concrete material properties were taken from the provided specifications.

Flexural failure, defined as rupture of the FRP or crushing of the concrete in compression, was the desired mode of failure. It was recognized that the externally bonded systems are more prone to debonding failures than the near surface mounted systems. According to Malek, et al. 1998[20], shear stresses developed at the FRP cut-off point were significantly lower than the shear strength of the concrete. Therefore, plate-end debonding should not be a concern. To delay FRP delamination-type failures along the length of the girder, 6 in (150 mm) wide U-wraps were installed at 3 ft (900 mm) spacing for all externally bonded strengthened girders. This arrangement was selected to simulate typical anchorage details commonly used by the construction industry for reinforced concrete members strengthened with FRP.

Test Setup & Instrumentation

All girders were tested using a 110 kips (490 kN) MTS hydraulic actuator, except for the control girder under static loading which was tested using a 450 kips (2000 kN) MTS hydraulic actuator. The actuator was mounted to a steel frame placed at the midspan of the girder. For the static tests, in order to simulate loading conditions on an actual bridge, a set of truck tires filled with silicon rubber were used to apply the load from the actuator. The footprint of the two tires was approximately 10 in x 20 in (150 mm x 250 mm), the same area as the AASHTO specified design loading area[21]. Due to stability concerns these tires could not be used during the fatigue tests and a steel plate of the same dimensions was utilized. To simulate small displacements at the supports, the girder was supported at both ends on a 2.5 in thick (64mm) neoprene pad which in turn rested on a 1 in thick (25mm) steel plate. The width of the neoprene pad was 8.5 in (216mm) which yielded a clear span of 28.58 ft (8710 mm).

The behavior of the girders during the static testing was monitored using three sets of string potentiometers, placed at quarter spans, and two linear potentiometers to measure vertical displacement over the supports. During the fatigue tests, two linear potentiometers were placed at midspan along with two more over the supports. The compressive strain in the concrete was measured during all the tests using a combination of PI gauges (a strain gauge mounted to a spring plate) and electrical resistance strain gauges located beside and between the loading tires (or plate, for the fatigue tests). PI gauges were also placed at the level of the lowest prestressing strand to measure the crack width and to determine the strain profile through the section. The tensile strain in the CFRP reinforcement was measured using six strain gauges: two at midspan, two at 6 in (150 mm) from midspan and two at 12 in (305 mm) from midspan. A schematic of the test setup used for the static tests is shown in Figure 2.

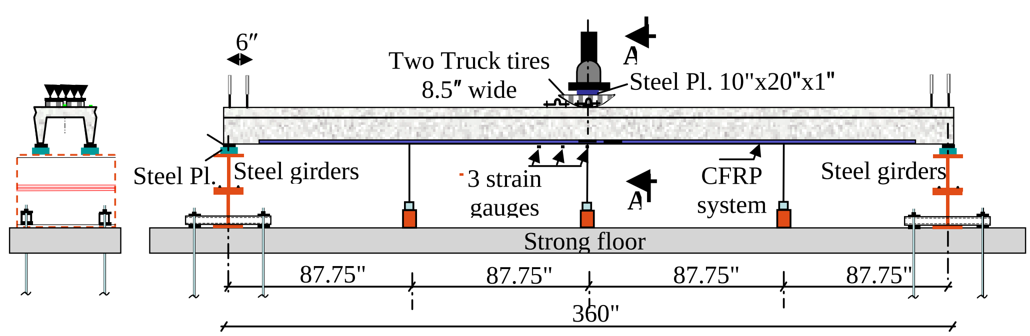


Figure 2. Test setup for strengthened girders tested in static

The loading sequence of all the tested girders started by increasing the applied load to a load level slightly higher than the cracking load, unloading, and then reloading again at a rate of 0.1 in/min (2.5 mm/min) up to the load at which the crack at midspan reopens. This loading sequence was selected to determine the effective prestressing force in the girders by observing the re-opening of the flexural cracks[22]. After this loading sequence, the static test specimens were loaded up to a load level equivalent to yielding of the prestressing strands, after which the rate of the applied load was increased to 0.2 in/min (5 mm/min) up to failure.

The above method was used for the fatigue test girders to simulate an overload condition and to determine the effective prestressing force. After this, the girders to be tested in fatigue were cycled between two load values at a frequency of 2 Hz. The two load values were chosen to simulate typical service loads encountered on a bridge such as this, that is, the lower load was the dead load and the upper load was the dead load plus the live load. The dead load, which corresponds to the lower stress range in the prestressing strands, for all the girders tested in fatigue was the same, 2 kips (8.9 kN), and taken as approximately the equivalent load (based on our test setup) generated by a moment from a 2 in (50 mm) wearing surface, or pavement overlay. The live load used for the control specimen was 9 kips. This was based on the service load the original girder was designed for (HS15 type loading) and includes the appropriate distribution and impact factors. For three of the strengthened girders tested in fatigue (F1, F2, F3), this was increased 20% to 11 kips, and for F4 it was increased 20% for one million cycles and 60% for the next one million cycles.

STATIC TEST RESULTS AND DISCUSSION

This section presents test results of the C-channel girders tested under static loading conditions. The general behavior of each girder is summarized in the following subsections. Test results for static girders are summarized in Table 1.

Control Girder – Static Test

All the tested girders were uncracked prior to testing. Cracking of the control girder (S0) occurred at a load of 12.6 kips (55.8 kN). Flexural cracks were evenly distributed along the length of the girder, approximately at the location of mild steel stirrups. Yielding of the bottom prestressing strands took place at a load level of 25.8 kips (115 kN) measured by the PI gauge readings. Failure was due to crushing of concrete at a load level of 33.2 kips (148 kN) which corresponded to a deflection of 9in (228 mm). The average crack width at failure, calculated from PI gauge readings at the tension face, was 0.08 in (1.9 mm). Due to the confining effect induced by the loading tires, crushing of the concrete occurred first at the edge of the girder at midspan before it occurred underneath the loading area. The maximum measured concrete compressive strain at failure was 0.0029. The load-deflection behavior is shown in Figure 3. This plot does not include the measured camber for each girder. The load-crack width behavior of all statically tested girders is shown in Fig. 4.

Near Surface Mounted FRP Systems – Static Tests

The two girders, S1 and S2, strengthened with a NSM system using either FRP bars or strips respectively, performed similarly during testing. Flexural cracking was observed at a load level of 12.4 kips (55 kN) for both girders. The initial stiffness of both girders was similar to that of the control girder, as was the post-cracking stiffness. After yielding of the prestressing strands, the presence of the CFRP reinforcement constrained opening of the cracks and consequently reduced the midspan deflection compared to the control girder. The mild steel stirrups again acted as crack initiators, but other cracks between the stirrups were also observed. The failure of both girders strengthened with NSM CFRP reinforcement was due to concrete crushing followed by debonding of the NSM CFRP reinforcement at a load of 40.8 kips (181 kN) for the NSM bars and 40.7 kips (180 kN) for the NSM strips as shown in Fig. 5. Since the ultimate strain in the concrete controlled the failure mode of both girders, the curvature and consequently, the deflection at ultimate were very similar. Test results showed that strengthening of the prestressed girders using NSM CFRP bars and strips increased the ultimate load carrying capacity of the girder by 22.9 and 22.6 percent with respect to the control girder.

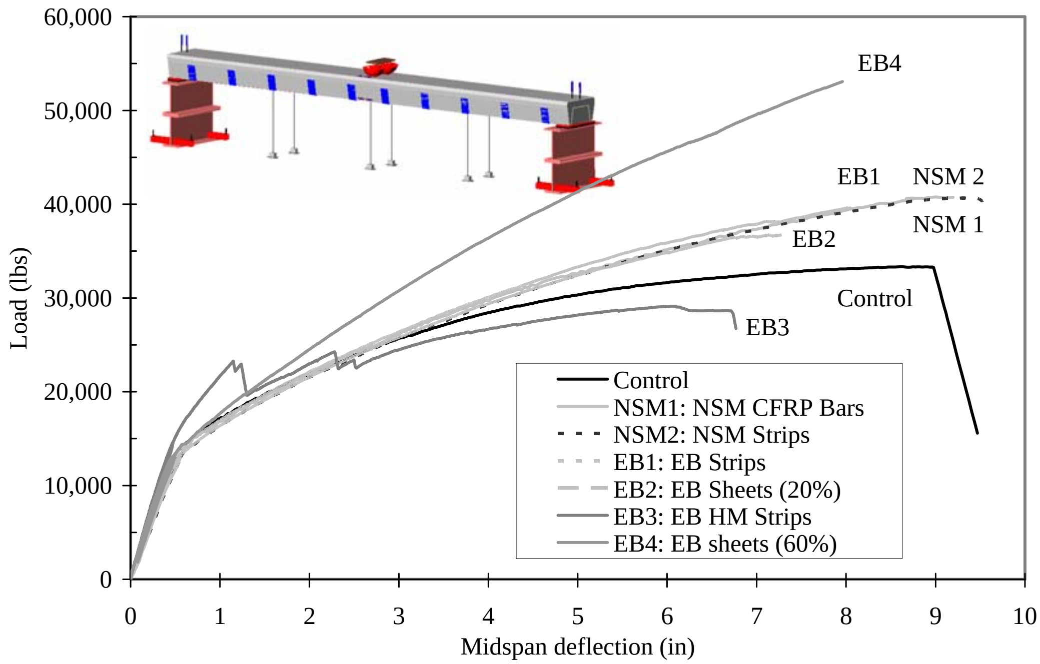


Figure 3. Load-deflection behavior of static test specimens

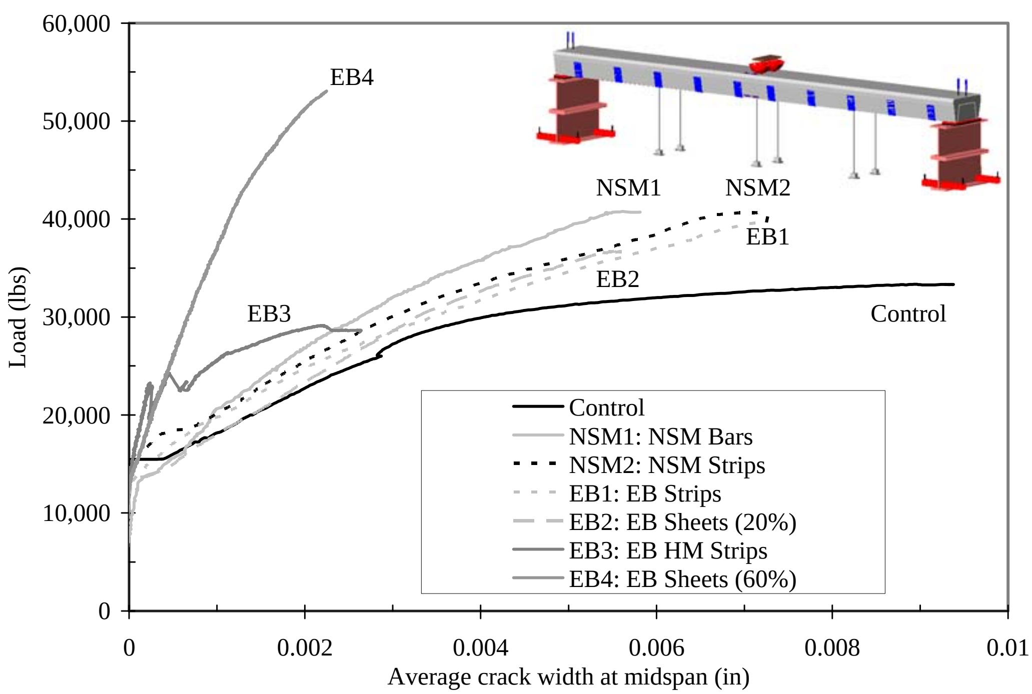


Figure 4. Load versus crack width for statically tested specimens

Figure 5. Typical failure due to concrete crushing for the NSM strengthened specimens

Externally Bonded FRP Systems – Static Tests

A total of four girders were strengthened with externally bonded CFRP systems. The behavior of the prestressed concrete girder strengthened with externally bonded CFRP strips (S3) and the girder strengthened with 2.5 plies of externally bonded sheets (S4) matched those of the NSM strengthened girders before and after cracking as shown in Figure 3. Flexural cracking at midspan was observed at a load level of 11.9 kips (57 kN) for both specimens. The cracking pattern and average crack width at midspan was similar to that of the NSM-strengthened girders. For girder S3, failure occurred due to debonding of the strips at midspan at a load level of 39.6 kips (176 kN) as shown in Fig. 6. The majority of the debonding occurred within the concrete substrate, although in several places debonding was noticed within the CFRP strip (interlaminate debonding). Compared to the control girder, S3 achieved an increase in the ultimate load carrying capacity of 19 percent.

Figure 6. Debonding failure of externally bonded strips in girder S3

For girder S4, failure was due to rupture of the CFRP sheets at midspan at a load of 36.7 kips (163 kN); an increase of 10 percent compared to the measured ultimate load of the control girder. The maximum recorded tensile strain in the CFRP sheets was 1.29 percent, similar to results obtained from the tension coupon tests. It should be mentioned that the design of the girder was performed using the dry fiber properties provided by the manufacturer at an ultimate strain of 1.7 percent. Therefore, the desired percent increase in ultimate capacity was not achieved.

For the girder strengthened with externally bonded high modulus CFRP strips (S5), flexural cracking occurred at an applied load of 14 kips (67 kN). Post-cracking stiffness higher than that of the control girder, yet the girder failed due to rupture of the CFRP strips at a load of 23.1 kips (103.7 kN) just after yielding of the prestressing strands. The maximum measured strain observed in the CFRP strips at failure was 0.17 percent, which is 41 percent of the specified manufacturer's rupture strain and 74 percent of the rupture strain measured from the tension coupon tests. It is believed that the reason early rupture of the high modulus CFRP strips occurred was due to flaws in the manufacturing process as reported by the manufacturer of this relatively new material. After rupture of the CFRP strips, the test was continued until concrete crushing occurred at a load of 29.1kips (129kN). The lower level of maximum measured load in comparison to the control girder could be due to the sudden failure of the CFRP material imparting an impact load on the girder.

The fourth girder strengthened with an externally bonded system (S6) was designed to achieve an increase of 60 percent in the ultimate load carrying capacity compared with the control girder. The girder was designed using the laminate properties as provided by the manufacturer. Flexural cracking occurred at a load of 13 kips (57.6 kN). Failure was due to rupture of CFRP sheets at a load of 53.1kips (236kN) providing an increase of 60 percent over the ultimate load of the control girder. Rupture of the CFRP sheets occurred throughout the four layers at the same location. No debonding was noticed before rupture of the CFRP sheets.

FATIGUE TEST RESULTS AND DISCUSSION

This section presents test results of the C-channel girders tested under fatigue loading conditions. The general behavior of each girder is summarized in the following subsections. Test results for fatigue girders are summarized in Table 2.

Table 2. Summarized fatigue test results and material properties

Girder Designation	F0	F1	F2	F3	F4
Strengthening Technique	none	NSM Bars	NSM Strips	EB Strips	EB Sheets
FRP Details	--	see S1	see S2	see S3	see S6
f'_c (psi)	8105	10150	9880	10695	11950
# of cycles achieved	1,076,000	2,000,000	2,000,000	625,000	2,000,000*
Failure mode**	RP	C	C	RP	R
% decrease in ultimate compared to virgin girder	--	1.01	10.17	--	4.19 (increase)

1 kip = 4.448 kN, 1in = 25.4 mm

* one million cycles at 20% increase in LL, one million at a 60% increase

** RP = rupture of prestressing, C = crushing of concrete, R = rupture of FRP

Control Girder – Fatigue Test

Two errors occurred in the testing of the control girder in fatigue (F0). Therefore the results should not be used as representative of the control girder. The first experimental error that occurred was due to problems in the tuning of the actuator that led to a sudden load being applied to the girder, prior to testing, whose magnitude is not known. Also, before the fatigue loading began, the girder was loaded up to approximately 50% of the ultimate load of the control static girder, higher than the initial load used for the strengthened girders. A future test regimen (already underway and the results expected to be ready for the conference) includes the retesting of the control girder in fatigue, to validate the results of this test.

The first error described above makes determination of the cracking load impossible. During the initial loading sequence, crack reopening occurred at a value of 9.3 k (41.2 kN). After this the fatigue loading began at a frequency of 3 Hz between 2 kips and 11 kips (8.9 kN to 48.7 kN). The rationale for this loading is described earlier. Static tests were performed regularly throughout the testing sequence, and little or no degradation in stiffness was observed before failure of the girder, which occurred at 1,076,000 cycles due to fatigue rupture of one of the lower prestressing strands. Since there was no noticeable corrosion around the location of the ruptured prestressing strand, it is believed errors in the loading sequence prior to testing may damaged the lower prestressing strands, leading to a premature failure.

Near Surface Mounted FRP Systems – Fatigue Tests

One girder strengthened with NSM bars and another strengthened with NSM strips (identical to the two girders tested under static loading conditions, S1 and S2) were tested in fatigue, F1 and F2 respectively. As mentioned earlier, after the initial loading the strengthened girders were tested at a frequency of 2 Hz between 2 kips and 13 kips (8.9 kN to 57.6 kN). The cracking load of the NSM bars and strips strengthened girders occurred at loads of 12.2k (54.3 kN) and 11.5k (51.2 kN) respectively. For both girders, the largest degradation in stiffness occurred between the secondary loading sequence (used to determine the prestress losses) and 5,000 cycles. Between these static loading sequences, the girders show a marked secondary stiffness zone as the cracks near midspan extends upwards. This behavior stabilizes after 5,000 cycles; however, gradual stiffness degradation was observed up to 2 million cycles for both girders, most likely due to the formation of other small cracks along the length of the girder. The load-deflection behavior of the NSM bar strengthened girder at 2 million cycles is shown in Figure 7.

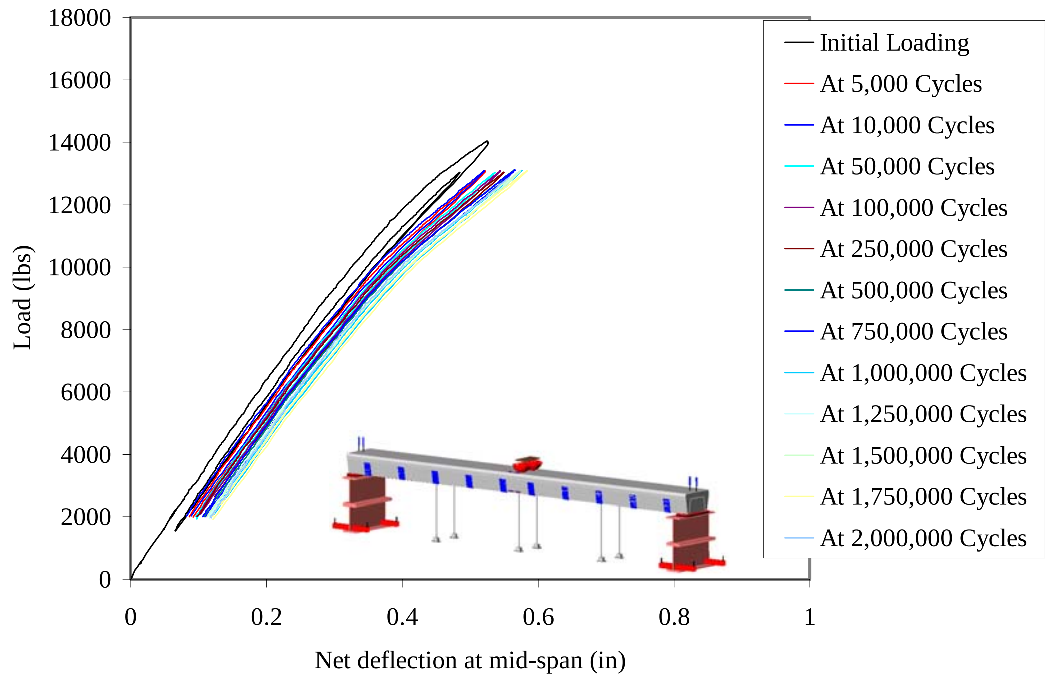


Figure 7. Fatigue load-deflection behavior of girder F1

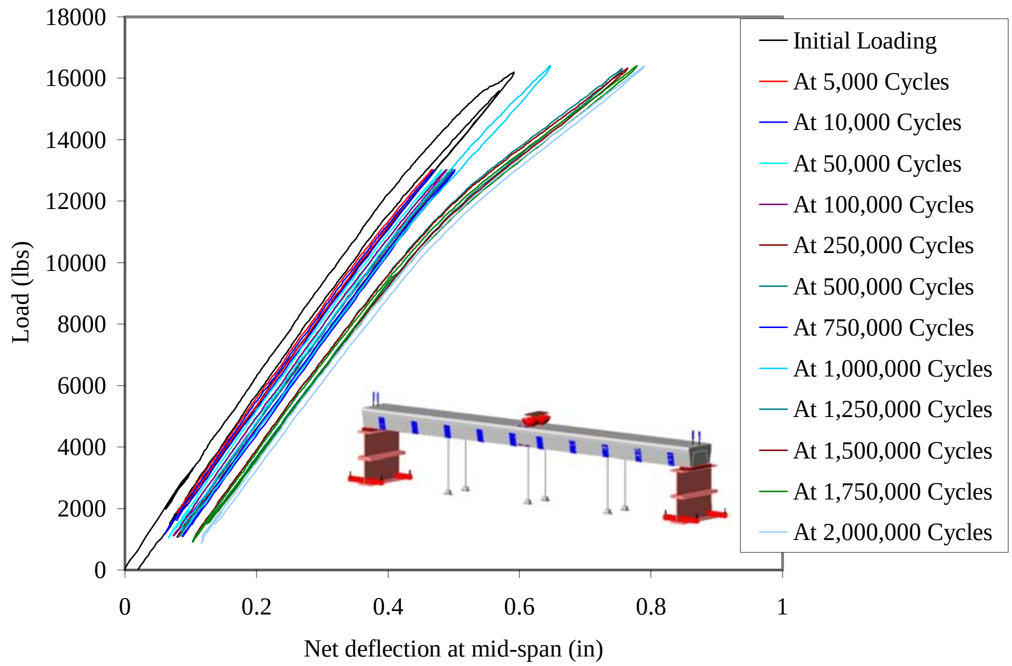
After 2 million cycles the girders were tested up to failure and showed little difference between the girders tested under static loading conditions. For the NSM bars strengthened girder tested in fatigue, failure was due to crushing of the concrete at a load of 40.2 kips (178.1 kN), compared to the statically tested girder which failed at a load of 40.8 kips (180.8 kN). The NSM strip strengthened girder tested in fatigue also failed due to crushing of concrete at a load of 36.9 kips (163.5 kN), compared to the statically tested girder which failed at a load of 40.6 kips (179.9 kN).

Externally Bonded FRP Systems – Fatigue Tests

Two girders strengthened with externally bonded systems were tested in fatigue: one girder strengthened with externally bonded strips (F3) and another strengthened with externally bonded sheets designed for a 60% increase in capacity compared to the control (F4). These were identical to two girders tested under static loading conditions (S3 and S6).

The girder strengthened with EB strips had a cracking load of 12.3 kips (54.5 kN). After the initial loading and reloading, the girder was tested at a frequency of 2 Hz between 2 kips and 13 kips (8.9 kN to 57.6 kN). Up to 625,000 cycles, the behavior of the girder was similar to the NSM strengthened girders: a large amount of stiffness degradation occurring early on and then stabilizing. At 625,000 cycles, however, a very large crack was noticed near midspan which led to localized delamination of the CFRP sheets from the concrete substrate. This crack was due to rupture of a prestressing strand (it is unclear if it was the lower strand or one further up the web). The next static test which was performed, at 750,000 cycles, showed a large degradation of stiffness and residual displacement. The test was continued, with the frequency changed to a lower value to accommodate the large displacements encountered, and catastrophic failure occurred at 908,000 cycles due to progressive rupture of the prestressing strands followed by debonding of the CFRP strips. The rupture of the first prestressing strand constitutes failure of the girder, since the deflection under service loading exceeded the limits specified for this type of girder.

A girder strengthened with externally bonded CFRP sheets designed to achieve a 60% increase in the ultimate load carrying capacity of the control girder was tested in fatigue. The cracking load of this girder was measured to be 15.8 kips (70.0 kN), much greater than any of the previously tested girders due to a higher effective prestressing force. Due to the uncertainty in calculating the stress range in the lower prestressing strand for the girder, it was decided to test this girder identically to the other strengthened girders: at 2 Hz between 2 kips and 13 kips (8.9 kN to 57.6 kN). Very little degradation was noticed between the initial loading sequences up until 1 million cycles. At this stage it was decided to cycle between the load range of 2 k to 16.4 k (8.9 kN to 72.7 kN), representing a 60 percent increase in live load, comparable to the amount of ultimate capacity designed for. After this loading started, the crack opening and secondary stiffness became apparent, but between 1.25 million cycles and 2 million cycles very little change was observed in the cracking pattern or the load-deflection behavior. After 2 million cycles the girder was tested to failure. Like the girder tested under static loading conditions (S6), failure was due to rupture of the CFRP sheets. Due to the higher prestressing force observed in this girder, the girder tested in fatigue failed at a load of 55.3 kips (245.0 kN), greater than the ultimate load of 53.1 kips (235.3 kN) observed for girder S6. The load-deflection behavior of this girder is shown in Figure 8.



COST - EFFECTIVENESS ANALYSIS

One of the goals of the research is to provide departments of transportation and the prestressed concrete industry a complete evaluation including a cost-effectiveness analysis of different FRP strengthening techniques. The cost-effectiveness of each FRP system used in this study was thoroughly analyzed to provide a comparison among different techniques.

To closely resemble field conditions, the girders were placed side by side, as they would be on a bridge, on top of a steel substructure approximately eight feet off the ground. The strengthening started in the winter when temperatures at night dropped below those recommended for curing of the epoxy by the manufacturer. Therefore a plastic enclosure in addition to a propane heater was provided. This cost was assumed to be similar for all types of bridges repaired under these conditions and was not included in the analysis.

To determine the cost-effectiveness of each strengthening technique the following items were considered: (1) labor costs of the professional FRP applicators, (2) time taken to complete all tasks, (3) material costs, (4) equipment used for strengthening. The material costs include all primers, adhesives and CFRP required for field application. Equipment items include the rental of sandblasting pot, compressor, sand, and diamond bit saw blades used to cut the grooves for the NSM strengthened girders. All other equipment (such as grinders, mixers, safety equipment, etc) is either assumed to be provided by the contractor or used equally in each of the strengthening systems. The results of the cost-effectiveness analysis are shown in Table 3. Cost-effectiveness in the table is defined as the increase in ultimate capacity per unit cost of the strengthening. Due to the fact that the girder strengthened with high modulus strips (S5) did not achieve an increase in strength over the control girder, it was not included in the analysis.

The cost analysis indicates that the most cost-effective system, when comparing the variables described above and the percent increase in strength, was the one which utilized four layers of CFRP sheets (S6 and F4). The NSM systems also performed well using these criteria. The only system with poor cost-effectiveness was the externally bonded CFRP strip used in girders S3 and F3, due to the high costs of the material.

Table 3. Cost-effectiveness of various FRP strengthening techniques

Girder Designation	S1 and F1	S2 and F2	S3	S4	S6 and F4
Strengthening Technique	NSM Bars	NSM Strips	EB Strips	EB Sheets	EB Sheets
FRP cost / foot	8.26	6.28	67.27	6.61	9.68
Adhesive cost / foot	7.26	7.26	5.58	2.89	
Equipment cost / foot	2.67	2.67	2.52	2.52	2.52
Labor cost / foot	30.60	29.70	22.95	22.50	31.05
Total cost / foot	48.78	45.90	98.32	34.52	43.25
% increase in strength	22.9	22.6	19.3	10.5	60
Cost effectiveness	0.47	0.49	0.20	0.30	1.39

1 ft = 0.3048 m

CONCLUSION

Twelve 40-year-old, 30 ft (9.14m) long prestressed concrete girders have been tested under static and fatigue loading. Two girders were tested as control specimens and ten girders were strengthened with various FRP systems. Based on the results, the following conclusions can be drawn:

1) The ultimate strength of prestressed concrete girders can be substantially increased using FRP strengthening systems. The ultimate load carrying capacity of the aged prestressed girders tested in this research increased by as much as 60 percent in comparison to the control girder.

2) The placement of CFRP U-wraps along the length of the girder for the externally bonded systems enhanced the performance of the girders.
3) NSM or EB FRP systems reduced the crack width of prestressed concrete girders at ultimate by 20-500 percent compared to the unstrengthened girder.
4) Girders strengthened using NSM CFRP reinforcement can withstand over 2 million cycles of a loading equivalent to a 20% increase in live load with little degradation.
5) Girders strengthened with externally bonded CFRP sheets can withstand over 1 million cycles of a loading equivalent to a 60% increase in live load with little degradation.
6) Defining the cost-effectiveness as the increase in ultimate capacity per unit cost of the strengthening, the most cost-effective systems are those which use externally bonded sheets. Externally bonded strips are the least cost-effective due to high material costs.

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