Strengthening of Scaled Steel-Concrete Composite Girders and Steel Monopole Towers with CFRP

D. Schnerch & S. Rizkalla
North Carolina State University, Raleigh, NC, USA

ABSTRACT: Cost-effective rehabilitation and/or strengthening of steel structures currently demanded by the telecommunications industry and transportation departments. Rehabilitation is often required due to cross-section losses resulting from corrosion damage and strengthening may be required due to changes in the use of a structure. Current strengthening techniques, have several disadvantages including their cost, poor fatigue performance and the need for ongoing maintenance due to continued corrosion attack. The current research program makes use of new high modulus types of carbon fiber for strengthening steel structures. The research program, currently in progress, includes phases to determine the appropriate resin and adhesive for wet lay-up of carbon fiber reinforced polymer (CFRP) sheets and bonding of CFRP strips, respectively. Test results of three scaled monopoles showed significant stiffness increases prior to yield. A significant stiffness as well as ultimate strength increase was found for the first steel-concrete composite girder tested in the program.

1 INTRODUCTION

1.1 *Research Objective*

While FRP materials have been successfully used for flexural strengthening, shear strengthening and ductility enhancement of concrete bridge structures, far less research has been conducted in strengthening steel structures with fiber reinforced polymer (FRP) materials. Increasing number of cellular phone users and their requirement for improved service has required cellular phone companies to increase the number of antennas on monopole towers. This trend has been exasperated by the reluctance of communities to allow new monopoles to be built. Addition of new antennas increases the wind load acting on the monopole, therefore strengthening is required to match this demand. Existing techniques for strengthening monopoles with steel collars or with an additional lattice structure are costly and negatively affect the visual appearance of the structure. Repair and/or rehabilitation of steel bridge are also being demanded by transportation departments. The strengthening technique must be cost effective and should not cause major interruption of traffic. The purpose of this research is to determine the feasibility of using new types of high modulus carbon fiber reinforced polymer (CFRP) materials to provide stiffness and strength increases for steel-concrete composite girders and steel monopoles.

1.2 *Background*

1.2.1 *Advantages*

There are many advantages in favor of the use of FRP materials for repair and rehabilitation of bridges and structures. Cost savings may be realized through labor savings and reduced requirements for staging and lifting material. Due to the ease of application, disruption of service during construction may be reduced or eliminated. The dead weight added to a structure is minimal and there is typically little visual impact on the structure, such that good aesthetics can be maintained.

1.2.2 *Adhesive Selection and Surface Preparation*

Previous work has illustrated the importance of surface preparation, adhesive working time, curing methods and prevention of the formation of galvanic couples in selecting an appropriate resin/adhesive system (Rajagoplan et al. 1996). Different types of adhesives have been used to bond CFRP to steel, but generally room-temperature cured epoxies have been chosen due to their superior performance and ease of use. Adhesion promoters, such as silanes, have been shown to increase the durability of steel-epoxy bonds without affecting the bond strength (McKnight et al. 1994). Since the study at this stage is focused mainly on the structural performance, silanes were not used in this program, but would be recommended for use in field applications.

Surface preparation of the steel must be undertaken to enhance the formation of chemical bonds between the adherend surface and the adhesive. This requires a chemically active surface that is free from contaminants. The most effective means of achieving this is by grit blasting (Hollaway and Cadei, 2002). For the CFRP strip, it is usually desirable that the strip would be fabricated with a peel-ply on one or both surfaces. However, for the small amount of CFRP produced for this program it was not economical to manufacture the CFRP strips with peel plys. As such, the procedure recommended by Hollaway and Cadei (2002) was followed, whereby the strips were abraded on the side to be bonded with sandpaper and cleaned with a solvent, which was methanol in this study.

1.2.3 *Previous Work*

Previous work has shown the effectiveness of the technique in improving the ultimate strength of steel-concrete composite girders, although little enhancement to the stiffness has been shown. Sen et al. (2001) strengthened steel-concrete composite girders that were initially loaded past the yield strength of the tension flange. Ultimate strength increases were possible, however stiffness increases were small particularly for the thinner of the two types of CFRP laminate strips studied. It was noted that even for these specimens, the increase in the elastic region of the strengthened members might allow service load increases. Tavakkolizadeh and Saadatmanesh (2003a) also noted considerable ultimate strength increases and insignificant elastic stiffness increases. Potential to increase the elastic stiffness increase by strengthening with many plys of CFRP strips was discounted, since as the number of plys increase, the efficiency for utilizing the CFRP decreased. However, for girders that simulated corrosion damage with notches of the tension flange, Tavakkolizadeh and Saadatmanesh (2003c) found that elastic stiffness increases were possible.

Vatovec et al. (2002) performed tests on square tubular steel sections that were 152 mm in depth with a span of 3048 mm. After some early trials, the tubes were filled with concrete to prevent premature local buckling of the tubes. The reported increases of strength varied from 6 to 26 percent depending on the configuration and number of plys used. No meaningful difference in stiffnesses between the unstrengthened tubes and strengthened tubes could be found and it was claimed that strengthened steel elements could not develop the full ultimate tensile or compressive strength of the CFRP due to premature delamination.

Current techniques for strengthening and rehabilitation of steel structures often require bolting or welding steel plates to the existing structure. Welding is often not desirable due to the poor fatigue performance of welded connections. In contrast, the fatigue performance of repairs made to cracked steel cross-girders by bonding with CFRP has been shown to be effective up to 20 million cycles (Bassetti et al. 2000). For notched tensile specimens subjected to fatigue loading, Gillespie et al. (1997) has shown that CFRP patches applied across the notch have the effect of reducing the stress concentration at the notch, thereby substantially increasing the life of the specimen due to the slower rate of crack propagation. This finding was confirmed for notched flexural specimens subjected to fatigue loading (Tavakkolizadeh and Saadatmanesh, 2003b).

The durability of CFRP materials bonded to metallic surface has to be carefully considered due to the potential for galvanic corrosion to occur if three conditions are met: an electrolyte (such as salt water) must bridge the two materials, there must be electrical connection between the materials and there must be a sustained cathodic reaction on the carbon (Francis, 2000). Brown (1974) studied the corrosion of different metals connected to CFRP by adhesive bonding or bolting. For the specimens connected by adhesive bonding there was no accelerated corrosion attack. This behavior was claimed to be due to the insulating behavior of most structural adhesives in not allowing electrical contact between the two materials. Tavakkolizadeh and Saadatmanesh (2001) provided the most comprehensive study of galvanic corrosion between steel and CFRP to date. Thicker epoxy films between the steel and CFRP surfaces were shown to significantly slow the corrosion rate of steel. The proceeding study proposed placing a layer of non-conductive GFRP as an insulating layer between the steel and CFRP interface. However, Tucker and Brown (1989) have found that glass fibers placed within a carbon fiber composite result in the blistering of the composite by creating conditions favorable for the development of a strong osmotic pressure within the composite. Clearly, water being drawn within the bond line by osmotic pressure is not favorable for maintaining a durable bond.

1.3 *Carbon Fiber Material*

The work presented in this paper makes use of two types of carbon fiber with properties given in Table 1. The high modulus carbon fiber used, was in the form of unidirectional tow sheets or CFRP laminate strips. These sheets had a width of 330 mm and are suitable when a wet lay-up process is necessary to conform to the exact surface configuration of the structure. The same fiber was also pultruded into unidirectional CFRP laminate strips using Resolution Performance Products Epon 9310 epoxy resin with Ancamine 9360 curing agent at a fiber volume content of 55 percent. These strips were expected to be more suitable for field applications where a greater degree of strengthening is required and flat uniform surfaces are available for bonding. An intermediate modulus fiber was also pultruded using the same epoxy and to the same fiber volume fraction. The properties of the CFRP strips, as determined by the manufacturer, are provided in Table 2.

Table 1. Fiber properties for two types of fiber used in the experimental program.

Fiber type	High modulus	Intermediate modulus
Tensile modulus (GPa)	640	438
Tensile strength (MPa)	2450	2550

Table 2. CFRP strip properties for two types of strips used in the experimental program.

Strip type	High modulus	Intermediate modulus
Width (mm)	75	75
Thickness (mm)	1.4	3.2
Tensile modulus (GPa)	340	230
Compressive modulus (GPa)	317	177
Tensile strength (MPa)	1190	1230
Compressive strength (MPa)	355	470
Tensile rupture strain (millistrain)	3.32	5.08

2 EXPERIMENTAL PROGRAM

2.1 *Outline*

The main objective of the experimental program is to develop a system to increase the elastic stiffness and ultimate strength of steel structures. The program consists of three phases. The first phase was conducted to determine a suitable resin for the wet lay-up of unidirectional carbon fiber sheets bonded to steel. For applications requiring more strengthening material, adhesive bonding of CFRP laminate strips is more practical. As such, the second phase focused on selection of adhesives and continued to determine development lengths for both materials. The third phase of the program was conducted to investigate the performance of the strengthening technique using larger scale specimens, including steel monopoles and steel-concrete composite girders.

2.2 *Phase I: Resin Selection*

Resin selection for the wet lay-up process was determined through testing of double lap shear coupons using ten different resins. Additional variables included different cure temperatures, use of a wetting agent and resin hybridization resulting in a total of twenty-two different trials. This work is presented in greater detail elsewhere, Schnerch et al. (2004).

2.3 *Phase II: Adhesive Selection and Development Length Study*

2.3.1 *Test Specimens*

Test specimens for adhesive selection and determining the development length for bonded laminate strips were Super Light Beams, designated SLB 100 x 4.8, as shown in Figure 1. The same beam specimens were also used to determine the development length for the CFRP sheets. An additional 6.4 mm thick, grade A36 steel plate was welded to the compression flange to simulate the strain profile of a bridge girder that acts compositely with a concrete deck, such that the neutral axis is located closer to the compression flange. The beams were then strengthened on the tension flange with either the CFRP laminate strips described previously, or by bonding one layer of the CFRP sheets. For this study, the width of the laminate strip was reduced to 35.8 mm to fit within the tensile flange of the test specimens. The width of the CFRP sheets used was 53.7 mm and its effective thickness was 0.19 mm. Different laminate strip lengths, varied from 51 mm to 203 mm, were used to determine the approximate development length and most suitable adhesive for bonded laminate strips. A length of 51 mm was used for the CFRP sheet and shown to be conservative in developing the full strength of the fibers.

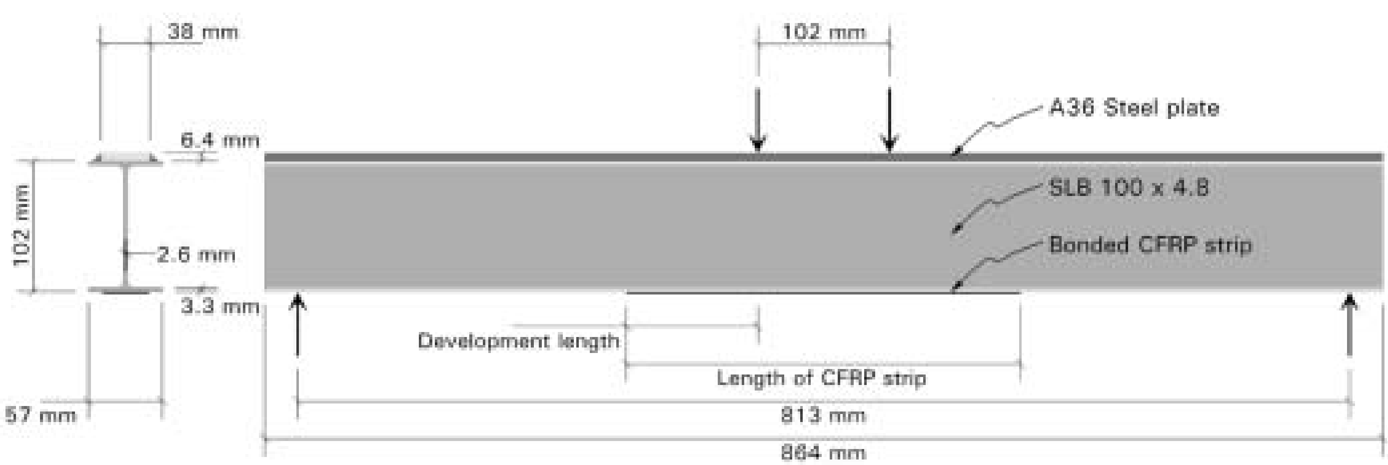

Figure 1. Cross-section dimensions and loading configuration of typical adhesive selection and development length specimen.

2.3.2 *Test Procedure and Instrumentation*

Six adhesives for bonding the laminate strips and two resins for bonding carbon fiber sheets were evaluated in the development length study. The six adhesives were: Fyfe Tyfo MB2, Jeffco 121, Sika Sikadur 30, SP Systems Spabond 345, Vantico Araldite 2015, and Weld-On SS620. The two resins were Degussa MBrace Saturant and Sika Sikadur 330. These two resins were selected from the ten resins evaluated using double lap-shear coupons in the first phase.

The beams were simply supported and loaded under four-point loading using a spherically seated bearing block over two 25 mm diameter steel rollers. Lateral bracing was provided by supporting the top flange of the beam over the supports with two steel angles fixed to each support. Load was applied at a constant displacement rate of 0.75 mm/min.

Strain and displacement were measured at midspan of the beam. Strain was measured using foil strain gauges bonded on the inside of the compression flange, inside of the tension flange, and outside of the tension flange for the unstrengthened specimen and outside on the laminate strip, or carbon fiber sheet, for the strengthened beams. Displacement was measured using two linear voltage displacement transducers.

2.3.3 *Results and Observations*

The results of the tests are shown in Tables 3 and 4 for the laminate strips and carbon fiber sheets respectively, indicating the ultimate strain at failure, either by rupture or debonding from the steel surface. The specimens fabricated using SP Spabond 345 and Weld-On SS620 adhesives had the shortest development lengths, developing the ultimate CFRP strip strain at a length of approximately 102 mm. Specimens tested with both Sika Sikadur 330 and Degussa MBrace Saturant achieved ultimate strain of the carbon fiber sheet at the development length of 51 mm. Certain development lengths were replicated to validate results. The average strains from these tests are underlined in both tables.

Of the six adhesives tested, SP Spabond 345 and Weld-On SS620 achieved a development length of 102 mm, Jeffco 121 and Vantico Araldite 2015 reached 152 mm, Fyfe Tyfo MB2 was 203 mm, and Sika Sikadur 30 was greater than 203 mm. The development lengths were based on ultimate strain of the laminate strip and failure by rupture.

Two tests were completed to verify a linear relationship assumption between the thickness of the laminate strips and the development length. Using the laminate strips provided, thickness was doubled by bonding two layers with the adhesives SP Spabond 345 and Jeffco 121 at the development lengths of 203 mm and 254 mm respectively. As shown in Table 3, the ultimate strain for the beam bonded with SP Spabond 345 achieved 3.09 millistrain and a rupture failure mode, which was the same failure mode and almost the same strain measurement, 3.11 millistrain, as the 102 mm development length tested beam. Ultimate strain for the beam bonded with Jeffco 121 was 2.02 millistrain; which was a little less than the ultimate strain of the laminate strip at half the development length of 127 mm, which was 2.66 millistrain. No conclusive decisions can be drawn from these

Table 4. CFRP sheet ultimate strain (millistrain) and failure mode for various development lengths.

		Development length		
Resin	Plys	102 mm	76 mm	51 mm
Degussa MBrace Saturant	1	-	-	3.57 rupture
Sika Sikadur 330	1	-	-	3.41 rupture
Degussa MBrace Saturant	2	3.24 rupture	2.04 debond	2.58 debond
Sika Sikadur 330	2	3.19 rupture	2.84 rupture	2.33 debond

* note that underlined strain values are the average from repeated tests and dashed indicate that no tests were conducted

Table 3. CFRP laminate strip ultimate strain (millistrain) and failure mode for various development lengths.

		Development length						
Adhesive	Plys	254 mm	203 mm	152 mm	127 mm	102 mm	76 mm	51 mm
Weld-On SS620	1	-	3.08 rupture	2.96 rupture	-	3.16 rupture	2.80 rupture	1.55 debond
SP Spabond 345	1	-	2.88 rupture	2.93 rupture	-	3.11 rupture	2.43 rupture	1.83 debond
Vantico Araldite 2015	1	-	3.09 rupture	2.98 rupture	-	2.82 rupture	2.18 debond	-
Jeffco 121	1	-	2.98 rupture	3.28 rupture	2.66 rupture	3.01 debond	-	-
Fyfe Tyfo MB2	1	-	3.47 rupture	3.06 debond	-	2.10 debond	-	-
Sika Sikadur 30	1	-	2.81 debond	-	-	-	-	-
SP Spabond 345	2	-	3.09 rupture	-	-	-	-	-
Jeffco 121	2	2.02 debond	-	-	-	-	-	-

* note that underlined strain values are the average from repeated tests and dashed indicate that no tests were conducted

few tests, though the results did show that doubling the thickness of the laminate strip increased the development length by at least twice the length for one layer of CFRP strip.

The same study was completed using carbon fiber sheets for 51, 73, and 102 mm development lengths. The results from these tests showed that doubling the thickness of the carbon fiber sheets required at least twice the development length needed for one layer. At 102 mm development length, the ultimate strain of the carbon fiber sheet was achieved.

2.4 *Phase III: Testing of Scaled Members*

2.4.1 *Scaled Steel Monopoles*

2.4.1.1 Test Specimens

Three scaled steel monopoles were fabricated from A572 grade 65 steel with similar proportions to monopoles that are typically used as cellular phone towers. The length of the monopole was 6096 mm with a dodecagonal, or twelve-sided, cross-section. The cross-section depth as measured from outside to outside of two opposing sides was tapered uniformly along the length, with a maximum depth of 457 mm at the base and a minimum depth of 330 mm at the tip. Cold forming was used to fabricate the monopole from two equal halves, which were then welded together along their length near mid-depth of the monopole. The monopole was welded to a base plate that was 38 mm in thickness to allow mounting of the monopole to a fixed structural wall. Three different strengthening configurations were examined in this series of testing. Monopole (MWL) was strengthened by wet lay-up of the high modulus CFRP sheets. Monopole (MAB-H) was strengthened by bonding CFRP laminate strips pultruded using the same high modulus fibers. The final monopole (MAB-I) was strengthened by bonding pultruded strips using an intermediate modulus carbon fiber.

Monopole (MWL) was strengthened by wet lay-up of 330 mm wide unidirectional, CFRP sheets in both the longitudinal and transverse directions using Sika Sikadur 330 resin. This process allowed the composite material to exactly conform to the surface configuration of the monopole. Strengthening was performed to match the demand placed on the monopole due to the cantilever loading condition. From a preliminary analysis, it was found that most of the strengthening is required at the base of the monopole and no strengthening was required from mid-span to the tip. As such, the thickness of the applied CFRP sheets was tapered from four plys of the sheets at the base to one ply terminating at mid-length of the monopole as shown in Figure 2. Anchorage was provided for the sheets by continuing the fibers past the shaft of the monopole and bending the fibers up onto the base plate. More resin was applied to the surface of the fibers and several steel angles were used to mechanically anchor the fibers to the base plate.

Half-width sheets were used to wrap the longitudinal sheets transversely to prevent possible premature delamination of the strengthening applied to the compression side of the monopole. These sheets were wrapped around the cross-section in two halves such that they overlapped by 100 mm at mid-depth of the monopole. The transversely oriented sheets were applied continuously from the base to 1200 mm along the length to also delay the onset of local buckling of the steel on the compression side. From this point to the mid span, the transversely oriented sheets were spaced apart from each other.

The two remaining monopoles were strengthened by adhesive bonding of unidirectional CFRP laminate strips. Monopole (MAB-H) used high modulus CFRP strips and monopole (MAB-I) used intermediate modulus strips. For both monopoles, the adhesive used was Spabond 345 with a fast hardener, manufactured by SP Systems. Similar to monopole

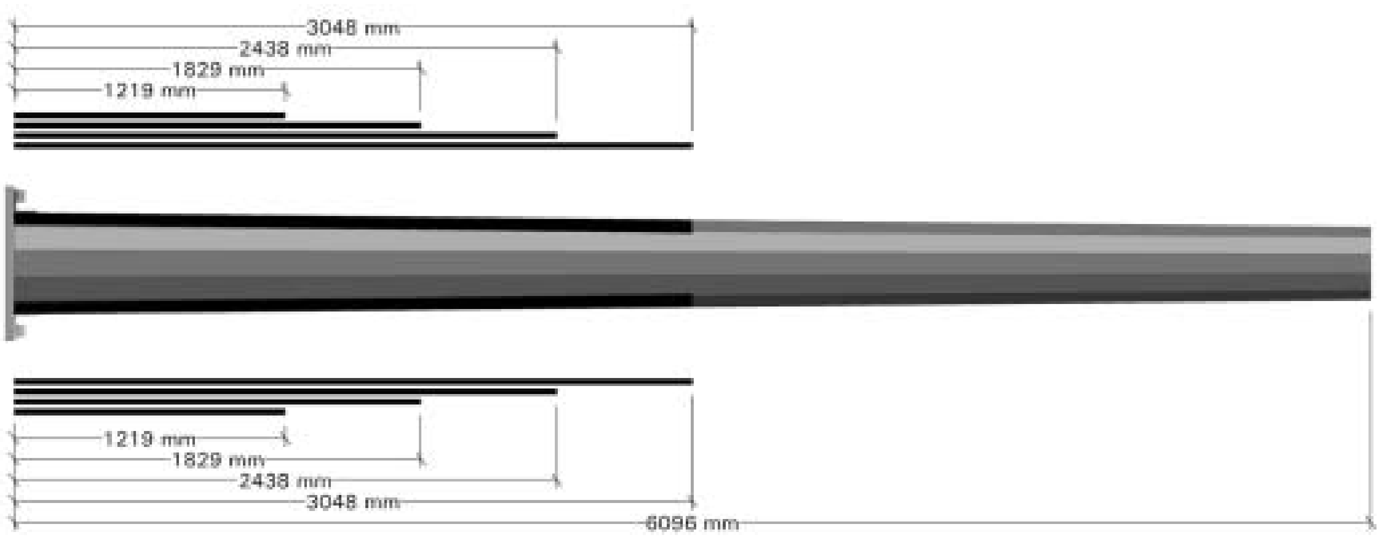

Figure 2. Longitudinal strengthening configuration of monopole (MWL).

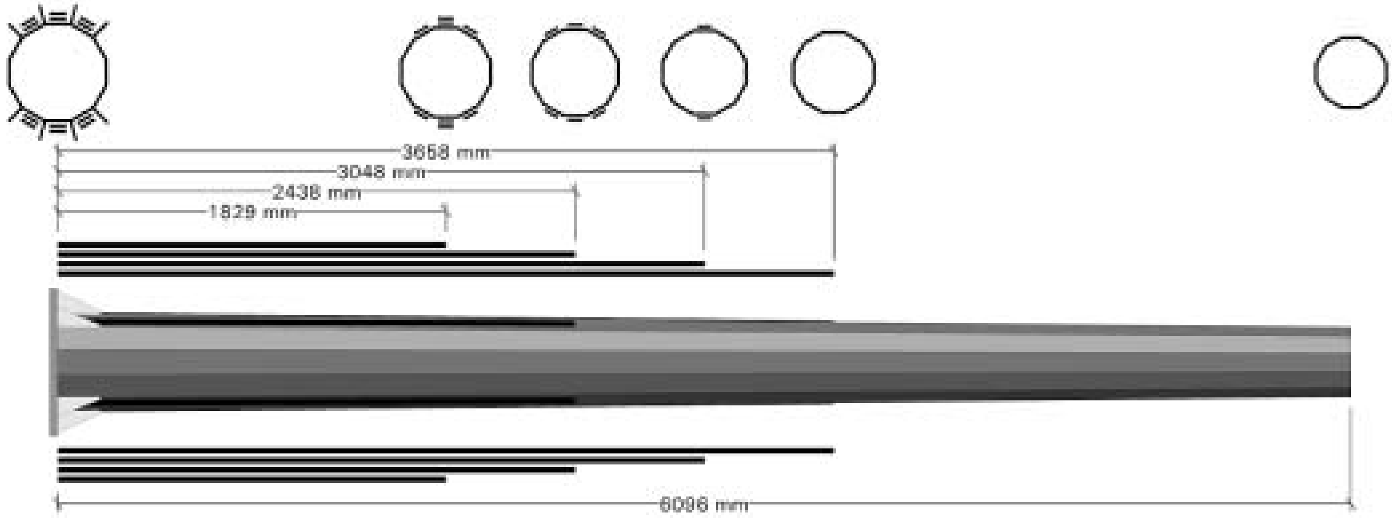

Figure 3. Longitudinal strengthening configuration of monopoles (MAB-H) and (MAB-I).

(MWL), due to the requirement for greatest strength increase at the base, six strips were applied to the three flat sides on both the tension and compression sides of the monopole and the amount of strengthening provided decreased with increasing distance from the base as shown in Figure 3. To allow comparison of the effectiveness between the two adhesive bonded systems, the applied strengthening had approximately equal values of the Young's Modulus multiplied by the cross-sectional area of the applied strips. To accomplish this, the width of the strips applied to the flats on the sides was reduced to 50 mm for monopole (MAB-I). Anchorage was provided by the addition of steel gusset plates welded from the base plate along the shaft at the corners of the cross-section. The length of the gusset plate was 200 mm to allow full development of two plys of the high modulus CFRP strips.

2.4.1.2 Test Procedure and Instrumentation

Each of the three monopoles was statically loaded to 60 percent of the specified yield stress and unloaded to determine the initial stiffness of each monopole. For monopole (MWL) this was completed before strengthening and for monopoles (MAB-H) and (MAB-I) this was completed after the addition of the gusset plates, but before strengthening with the CFRP strips. The monopoles were tested as cantilevers using the test setup shown in Figure 4. The monopole was mounted horizontally to a rigid structural wall by bolting to a steel fixture. Load was applied with nylon straps near the tip of the monopole, pulling the tip of the monopole upward. This type of loading was used to most closely represent the loading condition of a field monopole, whereby most of the loading from wind pressure is concentrated at the location of the antennas.

Figure 4. Test setup for scaled monopoles.

After strengthening, each monopole was reloaded to the same mid-pole displacement as the same monopole before strengthening to determine the stiffness increase resulting from the applied strengthening. After unloading, nylon straps were exchanged for steel chains and the monopoles were loaded to failure.

Measurements were taken of the deflection and extreme fiber strains at quarter points of the monopoles in addition to the actuator load and displacement. Deflection was also recorded at the base to determine the uplift as well as the rotation of the base plate. From these displacements, the net deflection of the monopole was calculated.

2.4.1.3 Results and Observations

Monopole (MWL): The result of strengthening by wet lay-up of CFRP sheets was that the net deflection of the monopole was reduced by 25 percent at the mid-length and the tip deflection was reduced by 17 percent compared to the unstrengthened monopole. The load-deflection behavior for both the strengthened monopole (MWL) and the same monopole before strengthening are given in Figure 5. The monopole was then reloaded to failure, which occurred due to rupture of the sheets on the tension side underneath the anchorage. Following rupture,

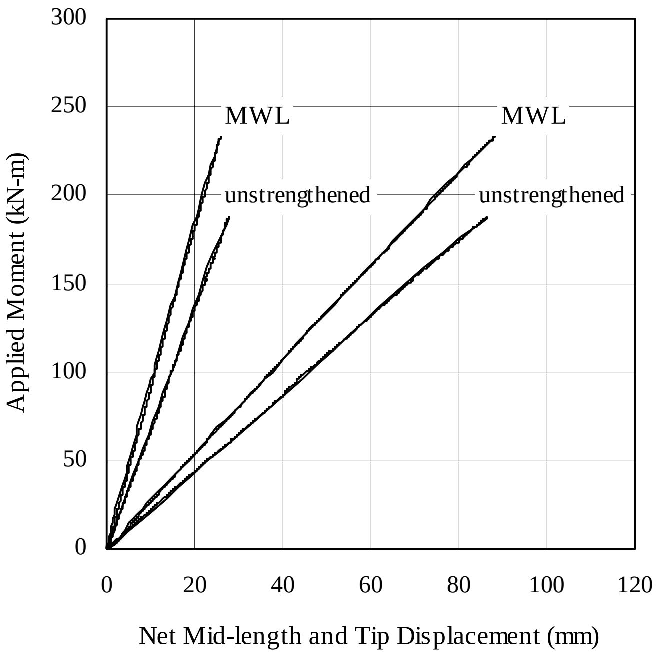

Figure 5. Comparison of net mid-length and tip displacements before and after strengthening of monopole (MWL).

Figure 6. Detail of local buckling at base showing anchorage angles for sheets.

redistribution of the stresses in the monopole resulted in local buckling of the monopole on the compression side about 150 mm from the base, as shown in Figure 6. This buckling ruptured the longitudinal and transverse fibers surrounding the buckled region. The ultimate moment capacity of the monopole was 548 kN-m with a maximum net deflection at the tip of 129 mm.

Monopole (MAB-H): The monopole was reloaded to the same mid-length displacement as before strengthening, but after welding of gusset plates. Figure 7 shows the load-deflection behavior at mid-length and at the tip. Net deflection of the monopole was reduced by 39 percent at mid-length and by 30 percent at the tip. Upon loading to failure, crushing of the laminate strips on the compression side preceded their debonding. Two of the strips on the tension side ruptured, while one debonded, as shown in Figure 8. Failure of the monopole was by local buckling near the base.

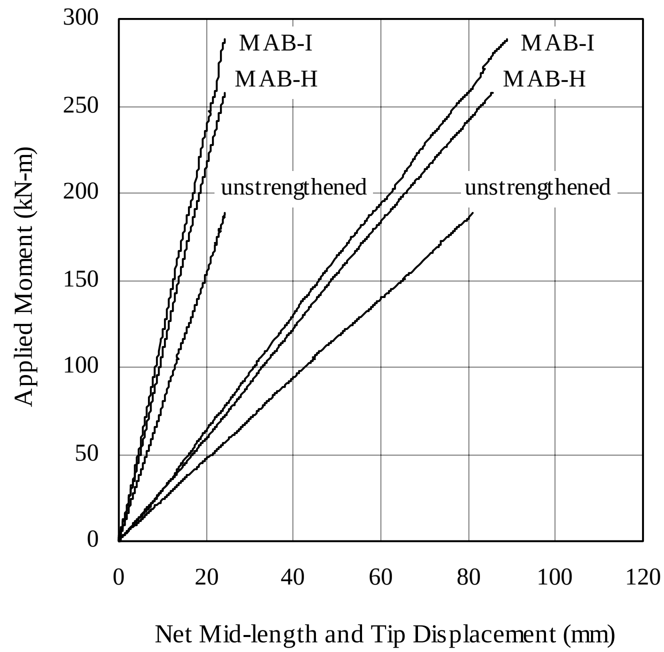

Figure 7. Comparison of net mid-length and tip displacements before and after strengthening monopoles (MAB-H) and (MAB-I).

Figure 8. Rupture of CFRP strips at end of gusset plates for monopole (MAB-H).

Monopole (MAB-I): Figure 7 also shows the behavior of the monopole strengthened with the intermediate modulus CFRP strips. This monopole showed the greatest stiffness increase, with a reduction of the net mid-length and tip deflection by 53 percent and 39 percent, respectively. Debonding of all the strips on the tension side occurred just after yielding of the monopole. Examination of the failure surface after testing showed that the strips were not fully wetted by the adhesive. This was likely due to their application being completed from underneath, together with their greater thickness in comparison to the previous strips resulting in possible sagging during adhesive curing. Near the ultimate strength of the monopole, but prior to local buckling, the strips on the compression side first crushed, and then began to debond.

2.4.2 *Scaled Steel-Concrete Composite Girders*

2.4.2.1 Test Specimens

Three identical steel-concrete composite girders were fabricated, simulating the geometry of steel-concrete girders commonly used for bridge structures. These girders used grade A36 steel W310 x 45 sections, with shear studs welded along the length of the compression flange to ensure full composite action between the concrete deck and the steel section as shown in Figure 9. These studs were staggered to prevent longitudinal cracking of the deck. The concrete deck was 100 mm thick and 835 mm in width and was reinforced with grade 60 steel reinforcing bars with a diameter of 12.7 mm at a spacing of 100 mm in the longitudinal direction and 152 mm in the transverse direction. End blocks were cast with the deck and fully encasing the steel girder at the ends to provide lateral stability and to eliminate the possibility of web crippling at the supports during loading. The distance from inside to inside of the end blocks was 6250 mm.

Concrete for all the girders was cast simultaneously from a single batch to minimize differences among the three girders. A nominal concrete strength of 31 MPa was used in the design; however test results of four standard concrete cylinders after

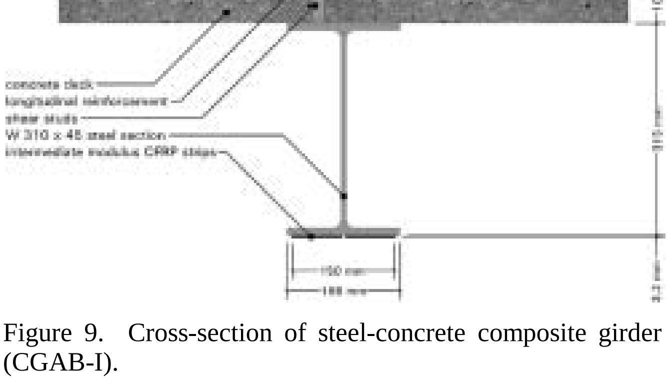

Figure 9. Cross-section of steel-concrete composite girder (CGAB-I).

28-days provided a compressive strength of 44 MPa with a standard deviation of 2 MPa. Results of an additional four concrete cylinders at the time of testing indicated no change in the concrete strength from the 28-day strength.

The first steel-concrete composite girder (CGAB-I) represents one of the three different strengthening configurations considered in this investigation. The strengthening technique for this girder was achieved using the same intermediate modulus CFRP strips used for strengthening monopole (MAB-I). As shown in Figure 10, two strips with a length of 4000 mm were placed side by side to strengthen the middle of the girder. The effectiveness of a spliced connection was investigated by bonding additional 1000 mm strips on either side of the main longitudinal strips and spliced with 400 mm long pieces of the same type of strip. This allowed a development length of 200 mm (or twice the development length determined from the second phase of the study) to transfer the forces from the 4000 mm long strips into the 1000 mm long strips. As a precaution against debonding due to peeling, the strips were wrapped around the flange by wet lay-up of 330 mm unidirectional carbon fiber sheets at the splice locations and at the ends of the 1000 mm long pieces.

Strengthening of composite girder (CGAB-I) was completed immediately following grit blasting the tension flange with an abrasive consisting mainly of angular iron and aluminum silicates produced as a by-product of coal fired power-generating stations. Surface preparation of the strips followed the same procedure used for the monopoles. All the strip ends were detailed with a 20-degree reverse bevel. This type of end treatment has been shown by Adams (2001) to improve the bond performance of the CFRP to steel due to reduction of the stress concentration at these critical locations.

The second composite girder (CGAB-H) will be strengthened using three plys of the high modulus carbon fiber strips similar to the ones used for strengthening monopole (MAB-H). The third composite girder (CGAB-H-PS) will also make use of the high modulus strips, however using one prestressed ply to improve the initial stiffness of the girder. Test results of the last two girderswill be presented at the conference.

2.4.2.2 Test Procedure and Instrumentation

All composite girders were initially subjected to a four-point flexural bending test with two equal loads applied symmetrically about the center of the girder as shown in Figure 11. The constant moment region was 1000 mm in length and load was applied across the entire width of the composite girder by means of two hollow square steel tubular sections with 25 mm thick neoprene pads placed between the tubes and the finished concrete surface. The girders spanned 6400 mm, center-to-center of the 75 mm thick

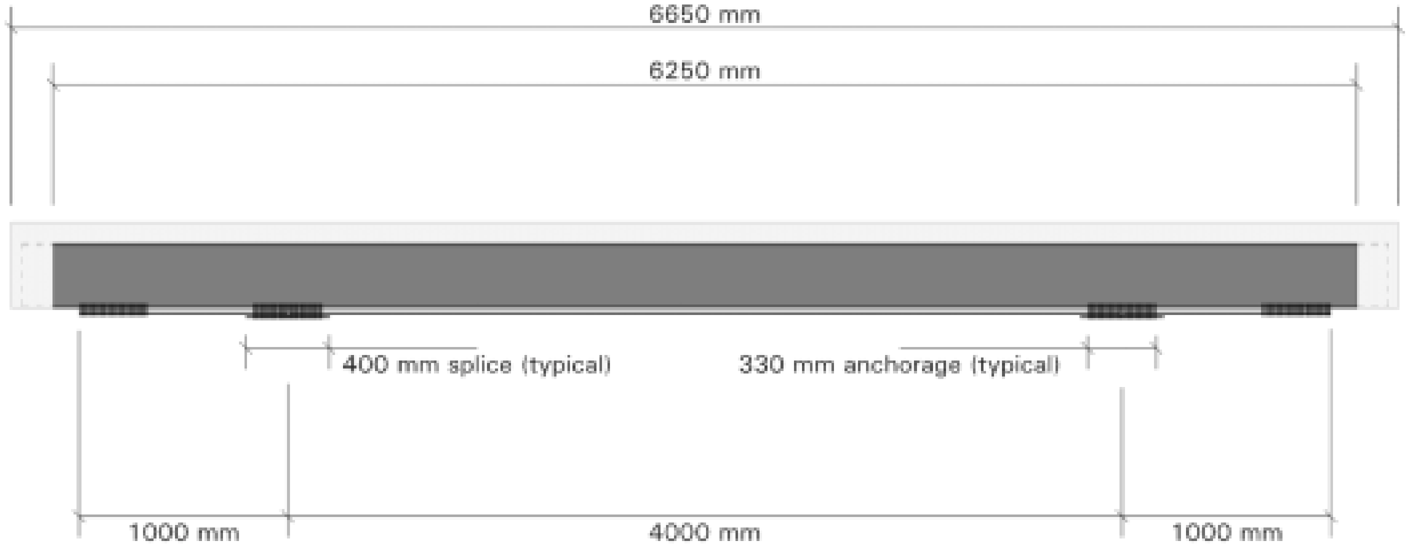

Figure 10. Reinforcement scheme for steel-concrete composite girder (CGAB-I).


Figure 11. Test setup for steel-concrete composite beam (CGAB-I) before strengthening.

neoprene support pads. Each girder was loaded to 60 percent of the specified yield stress and unloaded to determine the initial stiffness properties before strengthening. The girder was loaded under displacement control of the actuator at a rate of 0.5 mm/min.

After strengthening, the girder was reloaded to the same mid-span displacement using the same displacement rate to determine the stiffness increase. The girder was reloaded to failure using the same rate before yield, and then increased to 2.0 mm/min after yield.

Instrumentation consisted of wire potentiometers to measure the deflection at the quarter points of the girder and under the load points. Two potentiometers were also positioned at each end of the girder to measure the support settlement and the girder rotation. Strain gauge type displacement transducers were positioned above the top surface of the concrete and underneath the tension flange at each quarter point to determine the extreme fiber strains at these locations. For the strengthened girder, additional foil strain gauges were applied to the surface of the FRP at additional locations near the splice to determine its effectiveness.

2.4.2.3 Results and Observations

The load deflection behavior for the three loadings of composite girder (CGAB-I) is shown in Figure 12. The first loading was completed to a load of 155 kN at a net mid-span deflection of 16.1 mm. The second loading, showed the effectiveness in providing stiffness increase to the girder, resulting in a load of 173 kN at the same mid-span deflection or a 12 percent increase compared to the predicted value of 19 percent using moment curvature analysis. No degradation of the stiffness was apparent for the third loading, which became only slightly

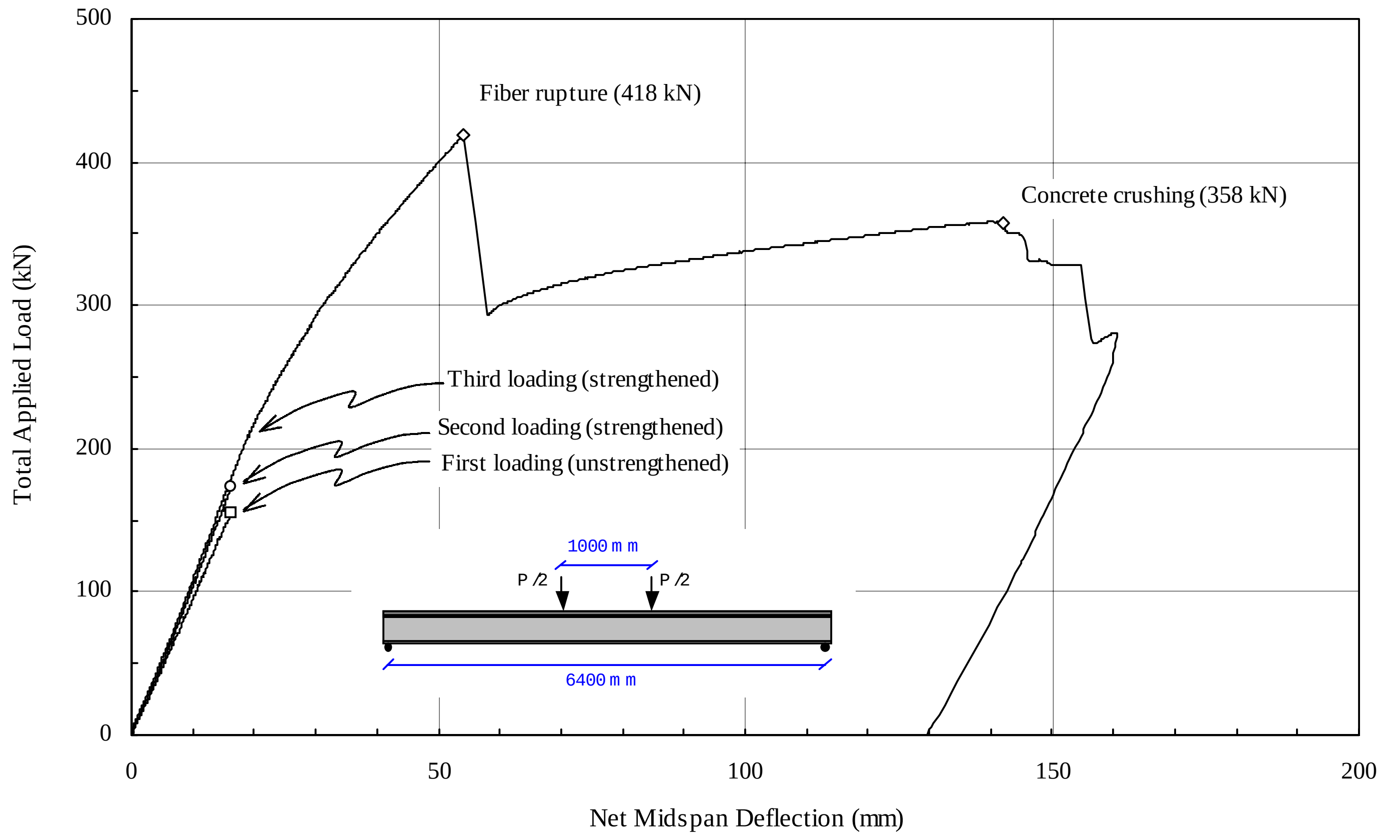

Figure 12. Load-deflection behavior of steel concrete composite beam (CGAB-I) for three loadings.

Figure 13. Concrete crushing failure and CFRP strip rupture of steel-concrete composite girder (CGAB-I).

nonlinear after yielding of the steel. The ultimate load, which was recorded prior to rupture of the CFRP strips, was 418 kN. The measured maximum CFRP strip strain, occurring at mid-span of the girder, was 4.50 millistrain. This is slightly less than the ultimate strain of the laminate subjected to pure tension conditions, determined by the manufacturer to be 5.08 millistrain. The load drop at failure was measured to be 42 percent, compared to a predicted value of 38 percent. Failure of the girder occurred at a load of 358 kN due to crushing of the concrete near one of the load points, as shown in Figure 13. This load represents the ultimate load of an unstrengthened girder, since at this point most of the strengthening material was no longer effective. As such, the ultimate load increase was 18 percent compared to an expected value of 24 percent. This difference was due to the higher measured concrete strain than the anticipated value for the girder.

Rupture of the fiber occurred within the constant moment region. Away from the rupture location, the strips partially debonded. The debonding surface was partially between the adhesive and the strip and partially within the strip itself, leaving a thin layer of fiber still bonded to the girder in some locations, as shown in Figure 14. No debonding or other distress was observed from the location of the splice to the ends of the girder, where the splice was wrapped transversely with sheets around the top of the tension flange.

Figure 14. Soffit of girder (CGAB-I) showing interlaminar shear failure of CFRP strip away from midspan.

3 CONCLUSIONS

A strengthening system for steel structures has been developed using high modulus CFRP sheets and high or intermediate modulus CFRP strips for providing stiffness increases and strength increases. Resins and adhesives for these materials have been selected. Typically development lengths for the CFRP sheets used are 50 mm, whereas for the strips, the development length varies between 100-200 mm based on the adhesive used. Substantial stiffness increases up to 39 percent at the tip have been shown for the three strengthened monopoles that were tested. The first steel-concrete composite girder tested in this program showed a stiffness increase of 12 percent and an ultimate strength increase of 18 percent. This strengthening technique achieved up to a 42 percent strength increase before rupture of the fibers.

4 ACKNOWLEDGEMENTS

The authors would like to acknowledge the financial support provided by the National Science Foundation Industry/University Cooperative Research Center and Mitsubishi Chemical for sponsoring this project. The assistance of Mr. Bryan Lanier in completing testing of the steel monopoles is greatly appreciated. Mr. Kirk Stanford and Mr. Mina Dawood are also acknowledged for their assistance in strengthening the first of the steel-concrete composite girders tested in this program.

5 REFRENCES

Adams, R.D. 2001. The design of adhesively-bonded lap joints: Modelling considerations. In, *A materials and processes odyssey, Proc. 46th intern. SAMPE symp. and exhib. Long Beach, California, 6-10 May 2001*.

Bassetti, Andrea, Alain Nussbaumer, and Manfred A. Hirt. 2000. Fatigue life extension of riveted bridge members using prestressed carbon fiber composites. In, *Steel structures of the 2000's, ECCS, Istanbul, 11-13 September 2000*.

Brown, A.R.G. 1974. The corrosion of CFRP-to-metal couples in saline environments. In *Proc. 2nd intern. conference on Carbon Fibers, London, 18-20 February, 1974.*

Francis, R. 2000. *Bimetallic corrosion: Guides to good practice in corrosion control*. London: National Physical Laboratory.

Gillespie, John W., Jr., Dennis R. Mertz, William Edberg, and Nouredine Ammar. 1997. Steel girder rehabilitation through adhesive bonding of composite materials. In, *Annual Technical Conference - ANTEC conference proc. Toronto, 27 April-2 May 1997*.

Hollaway, L.C. and J. Cadei. 2002. Progress in the technique of upgrading metallic structures with advanced polymer composites. *Progress in Structural Engineering Materials* 4 (2): 131-148.

McKnight, Steven H., Pierre E. Bourban, John W. Gillespie, Jr., and Vistap Kharbari. 1994. Surface preparation of steel for adhesive bonding in rehabilitation applications. In Kim D. Bashim (ed.), *Infrastructure: New materials and methods of repair, proc. ASCE materials engineering conference*, New York: 13-16 November 1994.

Rajagoplan, G., K.M. Immordino, and J.W. Gillespie, Jr. 1996. Adhesive selection methodology for rehabilitation of steel bridges with composite materials. In, proc. 11th technical conference of the American Society for Composites, Atlanta, Georgia, 7-9 October 1996.

Schnerch, David, Kirk Stanford, Emmett A. Sumner, and Sami Rizkalla. 2004. Strengthening steel structures and bridges with high modulus carbon fiber reinforced polymers: Resin selection and scaled monopole behavior, accepted for publication in, *Transportation Research Record*.

Sen, Rajan, Larry Liby, and Gray Mullins. 2001. Strengthening steel bridge sections using CFRP laminates. *Composites: Part B* 32 (4): 309-322.

Tavakkolizadeh, Mohammadreza. and Hamid. Saadatmanesh. 2001. Galvanic corrosion of carbon and steel in aggressive environments. *Journal of Composites for Construction* 5 (3): 200-210.

Tavakkolizadeh, M. and H. Saadatmanesh. 2003a. Strengthening of steel-concrete composite girders using carbon fiber reinforced polymer sheets. *Journal of Structural Engineering*. 129 (2): 30-40.

Tavakkolizadeh, M. and H. Saadatmanesh. 2003b. Fatigue strength of steel girders strengthened with carbon fiber reinforced polymer patch. *Journal of Structural Engineering*. 129 (2): 186-196.

Tavakkolizadeh, M. and H. Saadatmanesh. 2003c. Repair of damaged steel-concrete composite girders using carbon fiber-reinforced polymer sheets. *Journal of Composites for Construction*. 7 (4): 311-322.

Tucker, Wayne C. and Richard Brown. 1989. Blister formation on graphite/polymer composites galvanically coupled with steel in seawater. *Journal of Composite Materials* 23 (4): 389-395.

Vatovec, M., P.L. Kelley, M.L. Brainerd, and J.B. Kivela. 2002. Post strengthening of steel members with CFRP. In, *Proc. intern. SAMPE symp. and exhibition, v. 47 II, Long Beach, California, 12-16 May 2002*.