Flexural Strengthening of Steel Bridges and Towers Using High Modulus CFRP Materials

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ABSTRACT: Many steel structures are in need of strengthening due to requirements to increase carrying capacity and/or rehabilitation due to corrosion degradation. This paper summarizes the results of an investigation that has been conducted to evaluate the performance of carbon fiber reinforced polymer (CFRP) materials to increase the strength and stiffness of steel structures. Two types of specimens were used: monopole towers and steel-concrete composite girders representative of typical steel bridges. Three steel monopoles, scaled from those typically used by the telecommunications industry, have been strengthened using different schemes. The first monopole was strengthened by wet lay-up of high modulus unidirectional CFRP sheets that conformed to its dodecagonal cross-section. The remaining monopoles were strengthened using CFRP strips bonded to the flat sides of the monopoles using two different types of fiber. This paper also includes the results of a steel-concrete composite girder strengthened on the tension side using externally bonded intermediate modulus CFRP strips. The remaining girders will be strengthened using high modulus CFRP strips in two different configurations including one prestressed configuration. The results of these two girders will be presented at the conference

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. 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*

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.

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).

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
Tensile strength (MPa)	1190	1230
Tensile rupture strain (millistrain)	3.32	5.08

2 EXPERIMENTAL PROGRAM

2.1 *Outline*

Testing to evaluate the performance of CFRP materials bonded to steel structures 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 work of the first two phases is presented in greater detail elsewhere, Schnerch et al. (2004). The third phase of the program, described in this paper, was conducted to investigate the performance of the strengthening technique using larger scale specimens, including steel monopoles and steel-concrete composite girders.

2.2 *Testing of scaled steel monopoles*

2.2.1 *Test specimens*

Three scaled steel monopoles were fabricated from A572 60 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 cross-section. 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. 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

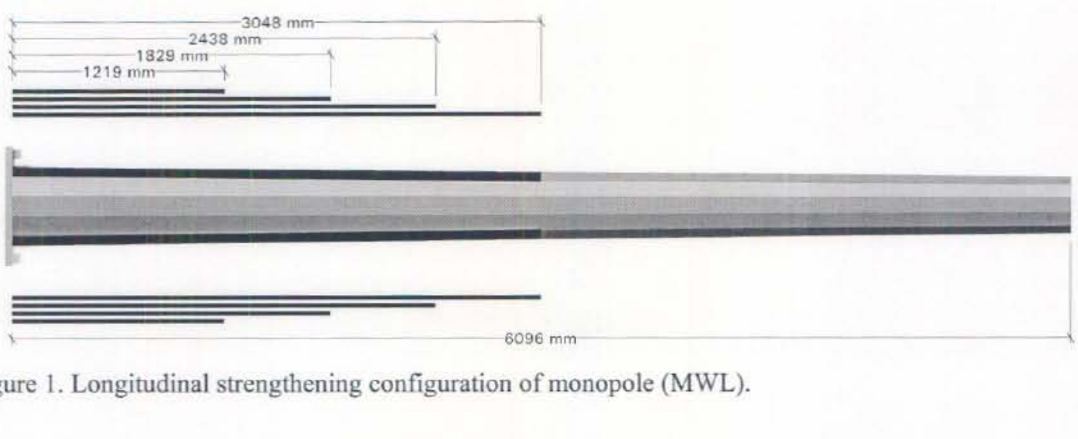


Figure 1. Longitudinal strengthening configuration of monopole (MWL).

one ply terminating at mid-length of the monopole as shown in Figure 1. 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, Spabond 345 adhesive with a fast hardener was used, manufactured by SP Systems. Similar to monopole (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 2. 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.

2.2.2 *Test procedure*

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, mounted horizontally to a rigid structural wall by bolting to a steel fixture. Load was applied with nylon straps at 305 mm from 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.

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.

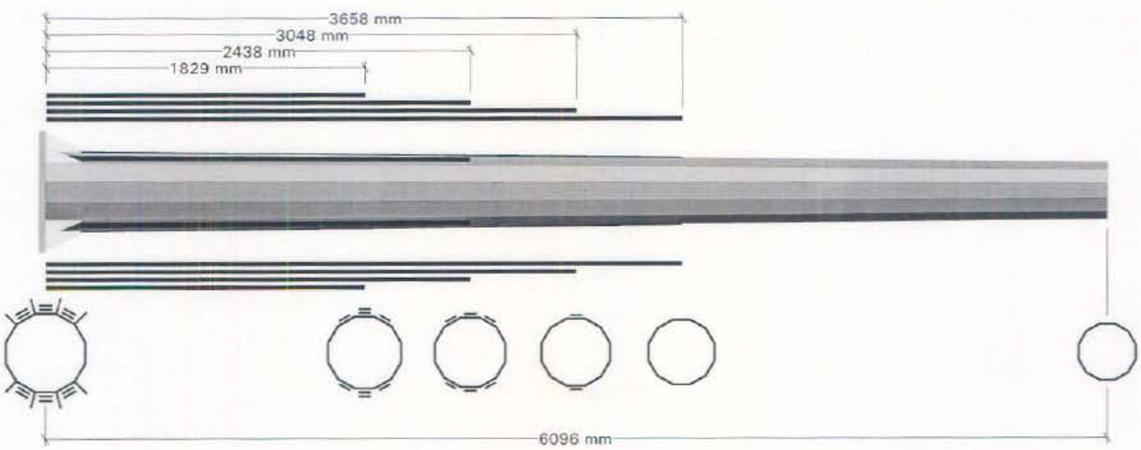


Figure 2. Longitudinal strengthening configuration of monopoles (MAB-H) and (MAB-I).

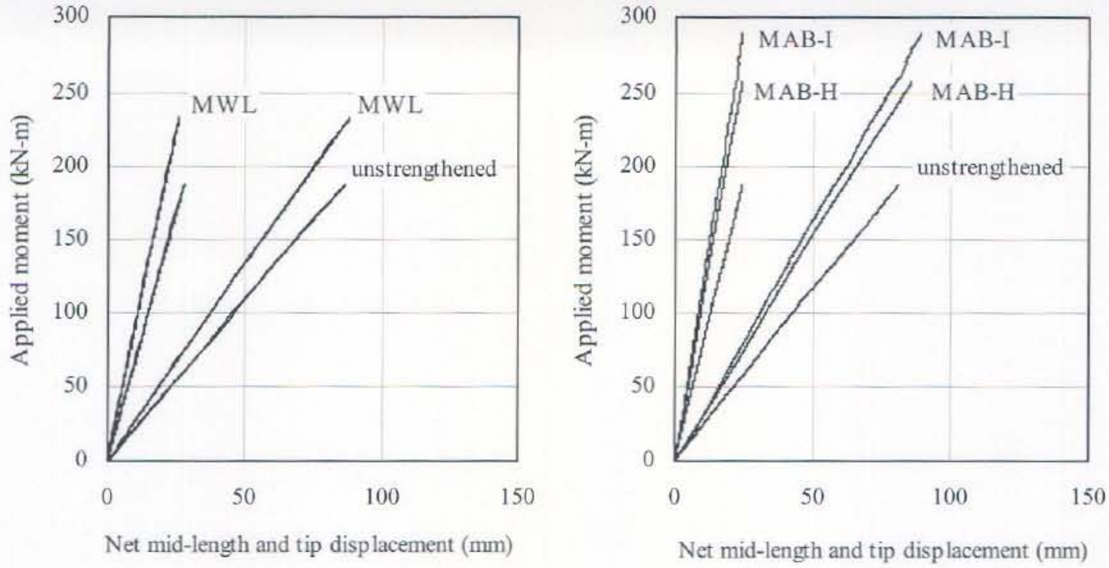
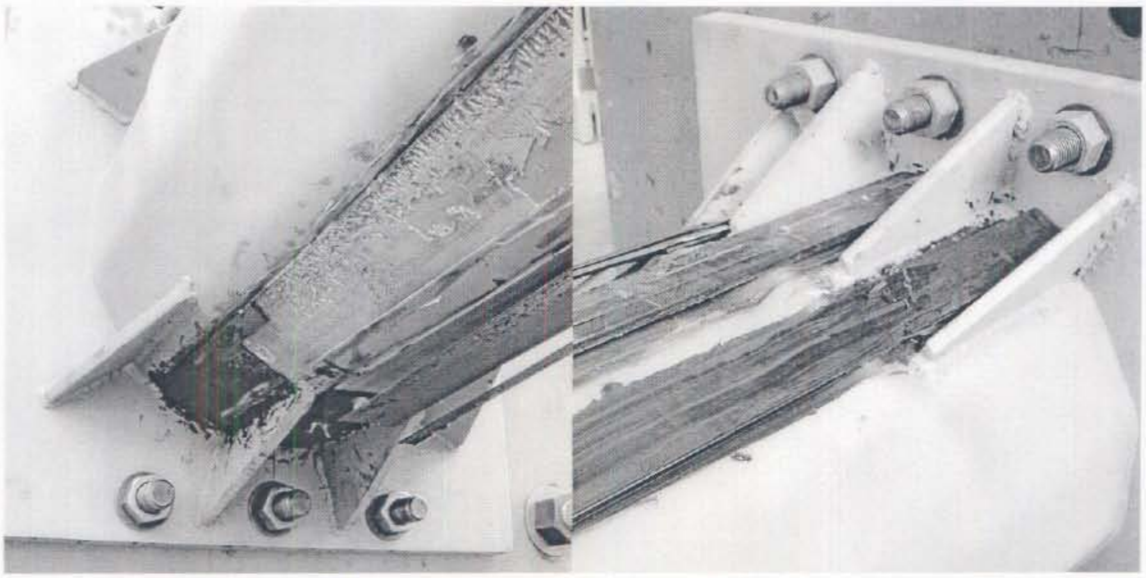


Figure 3a & b. Comparison of net mid-length and tip displacements for monopoles after strengthening.



Figures 4a & b. Rupture of CFRP strips after loading monopole (MAB-H) and local buckling failure of monopole (MAB-I).

2.2.3 *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 3a. The monopole was then reloaded to failure, which occurred due to rupture of the sheets on the tension side underneath the anchorage. Following rupture, redistribution of the stresses in the monopole resulted in local buckling of the monopole on the compression side about 150 mm from the base. 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 3b 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 4a. Failure of the monopole was by local buckling near the base.

Monopole (MAB-I): Figure 3b 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 sagging during adhesive curing. Near the ultimate strength of the monopole, but prior to local buckling, shown in Figure 4b, the strips on the compression side first crushed, and then began to debond.

2.3 *Testing of scaled steel-concrete composite girders*

2.3.1 *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 welded shear studs, staggered along the length of the compression flange. The 100 mm by 835 mm concrete deck 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. End blocks were cast with the deck, fully encasing the steel girder at the ends to provide lateral stability. Concrete for all the girders was cast simultaneously from a single batch to minimize differences among the three girders. Test results of four standard concrete cylinders after 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 significant differnece 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). 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, such that the entire length of the applied strengthening was 6000 mm. 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.

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 girders will be presented at the conference

2.3.2 *Test procedure*

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, with a constant moment region that was 1000 mm in length. Load was applied across the entire width of the girder by means of two hollow square steel tubular sections with 25 mm thick neoprene pads placed between the tubes and the concrete surface. The girders spanned 6400 mm, center-to-center of the 75 mm thick 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.

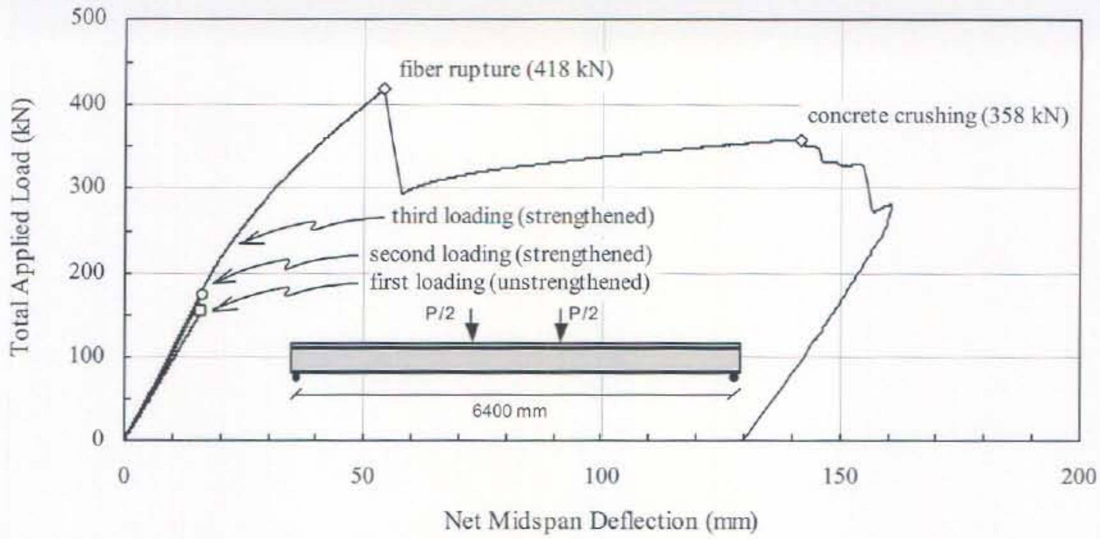


Figure 5. Load deflection behavior of steel-concrete composite girder (CGAB-I) for three loadings.

Figures 6a & b. Deflection of girder (CGAB-I) after CFRP strip rupture and girder soffit after failure.

2.3.3 *Results and observations*

The load deflection behavior for the three loadings of composite girder (CGAB-I) is shown in Figure 5. Loading was first completed to 155 kN with a net mid-span deflection of 16.1 mm. After strengthening, the second loading resulted in a load of 173 kN at the same mid-span deflection or a 12 percent increase. No degradation of the stiffness was apparent for the third loading, which became only slightly nonlinear after yielding of the steel. The ultimate load, 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. Figure 6a shows the girder deflection immediately after the strip ruptured. This was 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. 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 than anticipated concrete failure strain.

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 6b. 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.

3 CONCLUSIONS

A strengthening system for steel structures has been developed using high modulus CFRP sheets and high or intermediate modulus CFRP strips. 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 girder showed a strength increase of 42 percent before rupture of the fibers.

4 ACKNOWLEDGEMENTS

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