Strengthening of Steel-Concrete Composite Girders Using Carbon Fiber Reinforced Polymers Sheets

M. Tavakkolizadeh, M.ASCE,[1] and H. Saadatmanesh, M.ASCE[2]

Abstract: The use of advanced composite materials for rehabilitation of deteriorating infrastructure has been embraced worldwide. The conventional techniques for strengthening of substandard bridges are costly, time consuming, and labor intensive. Many new techniques have used the lightweight, high strength, and the corrosion resistance of fiber reinforced polymers (FRP) laminates for repair and retrofit applications. The load-carrying capacity of a steel-concrete composite girder can be improved significantly by epoxy bonding carbon fiber reinforced polymers (CFRP) laminates to its tension flange. This paper presents the results of a study on the behavior of steel-concrete composite girders strengthened with CFRP sheets under static loading. A total of three large-scale composite girders made of W355×13.6 A36 steel beam and 75-mm thick by 910-mm wide concrete slab were prepared and tested. The thickness of the CFRP sheet was constant and a different number of layers of 1, 3, and 5 were used in the specimens. The test results showed that epoxy-bonded CFRP sheet increased the ultimate load-carrying capacity of steel-concrete composite girders and the behavior can be conservatively predicted by traditional methods.

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Introduction

During the past 35 years, the American Association of State Highway and Transportation Officials (AASHTO) and the Federal Highway Administration (FHWA) have developed programs to rate bridges through biannual inspection. As a result, it has been found that more than one third of the highway bridges in the United States are considered substandard. According to the latest National Bridge Inventory (NBI) update, the number of functionally obsolete highway bridges in this country is more than 81,000. (FHWA Bridge Program Group 2001).

More than 43% of these bridges are made of steel. Steel bridges were among the most recommended group for improvement based on the NBI report. Corrosion, lack of proper maintenance, and fatigue sensitive details are major problems in steel bridges. In addition, many of these bridges need to be upgraded to carry larger loads and more traffic. In the NBI report, it was also recommended that repair and a retrofit option be considered before a decision is made to replace a bridge. The cost for rehabilitation and repair in most cases is far less than the cost of replacement. In addition, repair and rehabilitation usually takes less time, reducing service interruption periods. Considering the limited resources available to mitigate the problems associated with steel bridges, the need for adopting new materials and cost-effective techniques is evident.

The superior mechanical and physical properties of fiber reinforced plastics (FRP) make them excellent candidates for repair and retrofit of structures. FRPs are made of high-strength filaments (tensile strength in excess of 2 GPa) such as glass, carbon, and kevlar placed in a resin matrix. Glass-based composite has been readily available and fairly inexpensive. They have been used in applications involving concrete and masonry structures. The low-tensile modulus of these composites made them ineffective for retrofitting steel structures. Carbon fiber reinforced plastics (CFRP) display outstanding mechanical properties with typical tensile strength and modulus of elasticity of more than 1,200 MPa and 140 GPa, respectively. In addition, the CFRP laminates weigh less than one fifth of the steel and are corrosion resistant.

CFRP plates or sheets with high-tensile modulus can be epoxy bonded to the tension face of the member to enhance the strength and stiffness of the steel girders. By addition of the CFRP sheet, the stress level in the original member will decrease, that in turn results in a longer fatigue life. During the past decade, there have been many studies on repair and retrofit of concrete girders with epoxy bonded FRP materials, however, very few studies have addressed the use of epoxy bonded plates or sheets for strengthening steel girders.

This paper discusses the effectiveness of epoxy bonding CFRP sheets to the tension flange of steel-concrete composite girders to improve their ultimate load-carrying capacity and stiffness after yielding.

Previous Work

The most commonly used techniques for rehabilitation of bridges are (1) strengthening of members; (2) addition of members; (3) developing composite action; (4) producing continuity at the support; and (5) post-tensioning. In general, all of the conventional

[1]Assistant Professor, Dept. of Civil Engineering, Jackson State Univ., Jackson, MS 39217. E-mail: reza.tavakoli@jsums.edu

[2]Professor, Dept. of Civil Engineering and Engineering Mechanics, Univ. of Arizona, Tucson, AZ 85721. E-mail: hamid@u.arizona.edu

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techniques mentioned above require heavy machinery, a long period of service interruption, and they are costly. In most cases, they do not eliminate the possibility of reoccurrence of the problem completely.

For instance, the use of welded steel cover plates to repair and strengthen existing structures has been a popular method for years. The first use of this method can be traced back to 1934 in France when a 73-year-old bridge was strengthened (Klaiber et al. 1987). The major shortcomings of welded coverplates are (1) the need for heavy machinery in order to place the heavy steel plates in place and welding them; (2) the sensitivity of the welded detail to fatigue (Fisher 1997); and (3) the possibility of galvanic corrosion between the plate and existing member and attachment materials (weld, bolt, rivet).

Several studies have concentrated on the use of epoxy-bonded steel plates for strengthening of steel and concrete structures. The first reported application dates as far back as 1964 in Durban South Africa where the reinforcements in a concrete beam were accidentally left out during construction (Dussek 1980). The beam was strengthened by epoxy bonding steel plates to the tension face. By 1975, in Japan, more than 200 defective elevated highway concrete slabs were strengthened with epoxy-bonded steel plates (Raithby 1980).

In a study conducted at the University of Maryland, adhesive bonding and end bolting of steel cover plates to steel girders provided a substantial improvement in the fatigue life of the system (Albrecht et al. 1984). They reported an increase in the fatigue life of more than 20 times, compared to the welded cover plates.

In another study conducted at the University of South Florida, the possibility of using CFRP in repair of steel-concrete composite bridges was investigated (Sen and Liby 1994). They tested a total of six 6.10-m-long beams made of W203×11 steel section attached to a 711-mm-wide by 115-mm-thick concrete slab. The CFRP sheets used in the study were 3.65-m long, 150-mm wide, and had two different thicknesses of 2 and 5 mm. It was reported that CFRP laminates could considerably improve the ultimate capacity of composite beams.

The advantages of using advanced composite materials in the rehabilitation of deteriorating bridges were investigated at the University of Delaware (Mertz and Gillespie 1996). As part of their small-scale tests, they retrofitted eight 1.52-m-long W203 ×4.5 steel beams using five different retrofitting schemes. They reported an average of 60% strength increase in CFRP retrofitted systems. They also tested and repaired two 6.4-m-long corroded steel girders. The girders were typical American Standard I shape with a depth of 610 mm and flange width of 230 mm. Their results showed an average of a 25% increase in stiffness and a 100% increase in the ultimate load-carrying capacity.

Considering the limited research on the effectiveness of the epoxy bonding of CFRP sheets to the tension flange of steel-concrete composite girders, this study was conducted to investigate the effectiveness of this method. The experimental results were also compared with the conventional analytical methods. The authors in a separate article have addressed the concern of galvanic corrosion, when CFRP is used in conjunction with steel (Tavakkolizadeh and Saadatmanesh 2001). The results of this study indicated that galvanic corrosion was not significant and it could be further reduced by providing a thin layer of adhesive or a nonmetallic composite layer between the steel and CFRP.

Experimental Study

The effectiveness of the epoxy bonding of CFRP sheets on improving the ultimate load-carrying capacity of composite girders was examined by testing three large-scale girders strengthened with pultruded carbon fiber sheets. In order to observe the effectiveness of this technique, three different thicknesses of CFRP laminates were considered. Identical girders were strengthened with one, three, and five layers of CFRP sheets. The overall lengths of CFRP sheets were identical and cut-off points for each layer were staggered to prevent premature failure at termination points due to stress concentration (Schwartz 1992).

Materials

Tack Coat

A two-component viscous epoxy was used for bonding the laminate to the steel flange surface. The mixing ratio of the epoxy was one part resin (bisphenal A based) to one part hardener (polyethylenepolyamin) by volume. The epoxy had a pot life of 30 min at room temperature and was fully cured after two days at 25 °C. This epoxy immediately reached high-tack consistency and was ideal for over-head applications.

Epoxy

A two-component less viscous epoxy was used for bonding the laminates to each other. The mixing ratio of the epoxy was two parts resin (bisphenal A based) to one part hardener (polyamide) by volume. The epoxy had a pot life of 1 h at room temperature and was fully cured after seven days at 25 °C. This epoxy had a longer gel time and much lower viscosity and was used in between CFRP sheets to insure the least-entrapped voids.

CFRP

A unidirectional pultruded carbon fiber sheet with a width of 76 mm and thickness of 1.27 mm was used. After testing a total of 16 straight strips (coupons) with a length of 400 mm and width of 25 mm, an average tensile strength of 2,137 MPa, tensile modulus of elasticity of 144.0 GPa, and Poisson's ratio of 0.34 were obtained. A typical stress-strain plot for CFRP coupons tested in uniaxial tension is shown in Fig. 1(a).

Steel

W355×13.6 A36 hot rolled sections were used for the experiments. A uniaxial tension test was performed on seven dogbone specimens with a gage length of 125 mm, a gage width of 25 mm, and a thickness of 9.5 and 6.4 mm cut from flanges and the web, respectively. Average yield strengths of 354.9 and 381.9 MPa, a moduli of elasticity of 198.3 and 177.5 GPa, and Poisson's ratios of 0.305 and 0.299 were obtained from the specimens cut from the flange and web, respectively. A typical stress-strain plot for the flange in uniaxial tension test is shown in Fig. 1(b). The minimum reinforcement in the concrete slab for temperature and shrinkage was provided by using a 150×150×6.4 mm welded smooth wire mesh.

Concrete

Concrete was ordered from a ready mix plant with the nominal compressive strength of 15.5 MPa, a slump of 100 mm, and a maximum aggregate size of 10 mm. Twelve 75×150 mm cylinders were made at the time of casting and were kept with the girders during curing. They were tested under uniaxial compression right before the beam tests. The compressive strength and

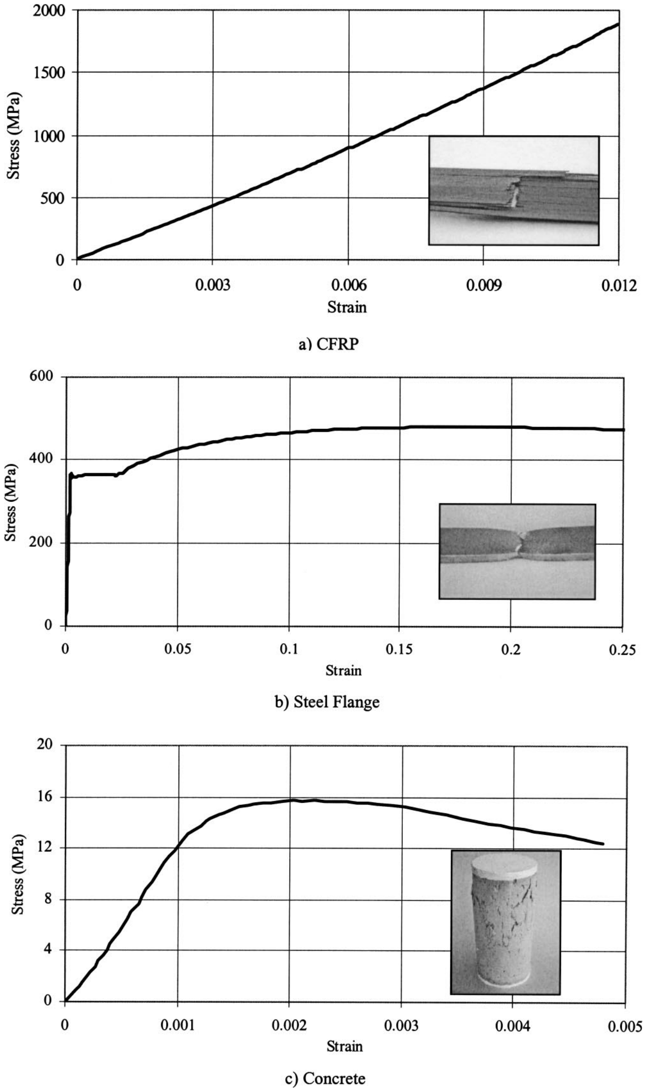

Fig. 1. Typical stress-strain behavior: (a) CFRP; (b) steel flange; and (c) concrete

modulus of elasticity of concrete was 16.6 MPa and 13.8 GPa, respectively. A typical stress-strain plot of concrete under uniaxial compression test is shown in Fig. 1(c).

Specimen Preparation and Instrumentation

The steel sections were first cut into 4.9-m-long beams. Then, shear studs with a diameter of 13 mm and a height of 51 mm were welded to the compression flange in two rows 125 mm on center along two shear spans. After constructing the forms and securing the edges of the forms, the wire mesh was placed in the midheight of the slab by using 38-mm-thick chairs as shown in Fig. 2. Two hooks made of #4 rebar were welded to the top flange at quarter lengths for transportation of the girders after casting. All slabs were cast at the same time and a hand-held vibrator was used for compaction. Several 75×150 mm cylinders were cast at the same time for compression testing. The girders and cylinders were kept moist under a plastic cover for one week.

CFRP sheets were cut to the proper length with a band saw. For the specimen retrofitted with one layer, a pair of 75-mm-wide, 3.95-m-long CFRP sheets was placed side by side and bonded to the steel girder. For the specimen retrofitted with three layers, three pairs of CFRP sheets were cut to 3.95, 3.65, and 3.35-m long (150-mm staggers) and placed side by side on the steel girder. For the specimen retrofitted with five layers, CFRP sheets were cut to the lengths of 3.95, 3.80, 3.65, 3.50, and 3.35 m (75-mm staggers). The ends of the sheets were finished smoothly using a grid 150 sandpaper. The surfaces of the sheets were sand blasted with No. 30 sand and then washed with saline solution and rinsed with fresh water. After the drying of the sheets, for a multiple layer system, the surfaces of the sheets were covered with thick layers of epoxy and were squeezed together with a small pressure to force the air bubbles and the excess epoxy out. Binder clips were used in close intervals for securing the edges of the sheets together.

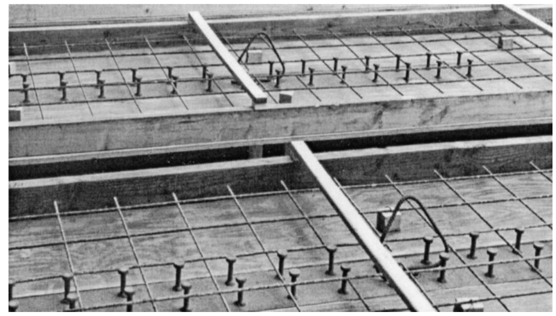

Fig. 2. Formwork for concrete slab

After the concrete slabs were completely cured, and just before applying the CFRP sheets, the tension flange of each girder was sand blasted using No. 30 sand, washed with saline solution, and rinsed with fresh water.

Upon drying of the steel beam and the CFRP sheets, the tack coat was mixed and applied to the tension flange surface and the sheets. All pieces were covered with uniform and thin layers of tack coat and were pressed together firmly to force the air pockets out with excess epoxy. The CFRP sheets were secured throughout their lengths using binder clips and 40×40×3-mm aluminum angle bars, while the tack coat was curing. After two h, the extra epoxy around the bond area was scraped off. A typical retrofitted specimen is shown in Fig. 3.

After one week, strain gages with resistance of 120 Ω were mounted on the surfaces of a steel beam, a CFRP sheet, and a concrete slab. In the midspan, the strain gages were mounted on top and bottom of the concrete slab, top, and bottom flanges of the steel beam and on CFRP sheets. In addition, strain gages were mounted at the end and the quarter length of CFRP sheets on the tension flange and CFRP sheet. The locations of the strain gages at midspan are shown in Fig. 4.

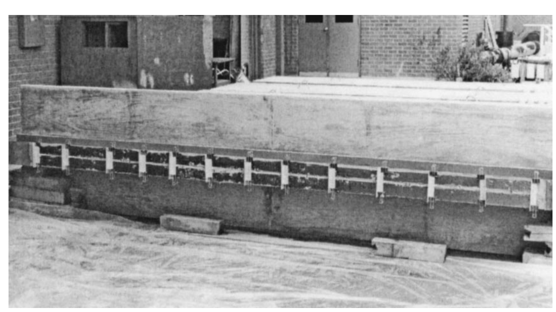

Fig. 3. Underside view of typical retrofitted girder

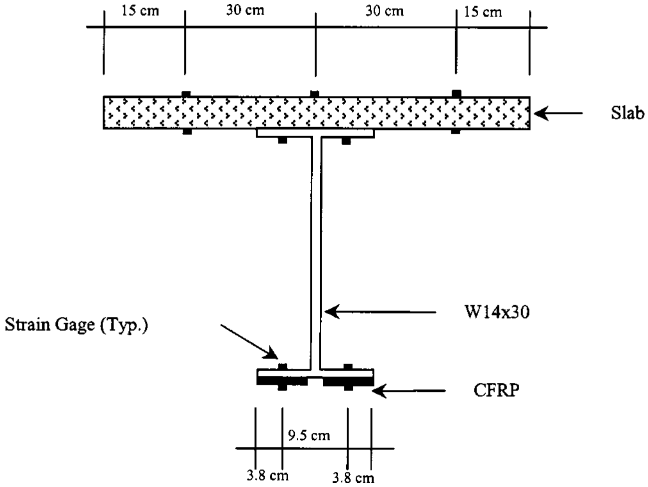

Fig. 4. Schematic of strain gage locations at midspan

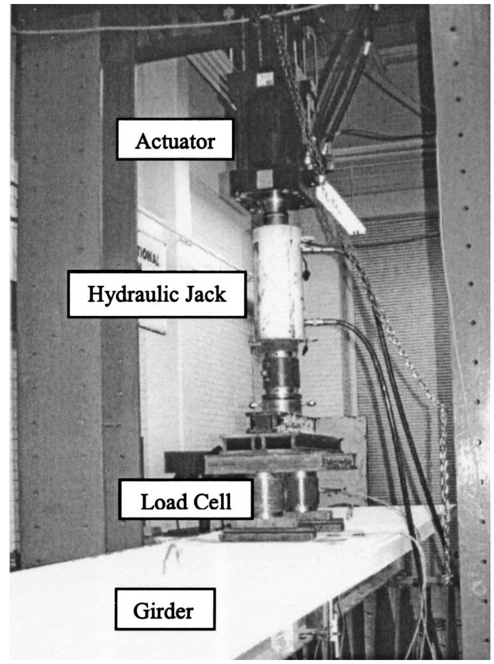

Fig. 6. Test setup

Eight pieces of a 100×100-mm wooden block were cut and tightly fit between the flanges using cider wedges at the supports and under loading blocks to prevent the web crippling. The loading platen on top of the slab was prepared by casting two 100 ×400×5-mm block using anchoring cement (Pour-Stone). The blocks were 500-mm apart and placed symmetrically on both sides of the midspan along the center of the slab.

Experimental Setup

Four-point bending tests were performed using a 2,200-kN test frame. Loading was applied by an MTS-244.41 hydraulic actuator and Enerpac-RRH10011 hydraulic jack with capacities of 500 and 1,000 kN, respectively. The Enerpac had to be used since the capacity of the MTS actuator was limited to 500 kN. The load was measured by two MTS-661.23A-02 load cells with a capacity of 500 kN and the deflection was measured by a DUNCAN 600 series transducer with a range of ±75 mm. Monotonic loading was performed under actuator displacement control with a rate of 0.025 mm/s. A total of three unloadings were carried out during each test; before steel yielded, after steel yielded, and at a 500 kN load level (switching from the actuator to the jack). The load, midspan deflection, and strains at different points were recorded with a Daytronic System 10 data acquisition system interfacing with PC through Microsoft Excel software.

The clear span was 4.78 m and the loading points were 0.5 m apart as shown in Fig. 5. The loading points and supports were made using rolling blocks, and one sphere blockhead was used to transfer the load from the hydraulic jack to the spreader beam. The test setup is shown in Fig. 6.

Analytical Modeling

An incremental deformation method insuring compatibility of deformations and equilibrium of forces was used in analysis. In addition, the ultimate load-carrying capacities of the girders were compared with the ultimate strength design method adopted by AASHTO.

Load-Deflection Behavior

In order to predict the load-deflection behavior of the steel-concrete-CFRP girder the following assumptions were made:

1. Strains vary linearly across the depth of cross section;
2. No slippage between steel, concrete, and CFRP, i.e., perfect composite action;
3. Elastic-perfectly plastic stress-strain relationship for steel;
4. Hognestad's Parabola for concrete behavior (Park and Paulay 1975);

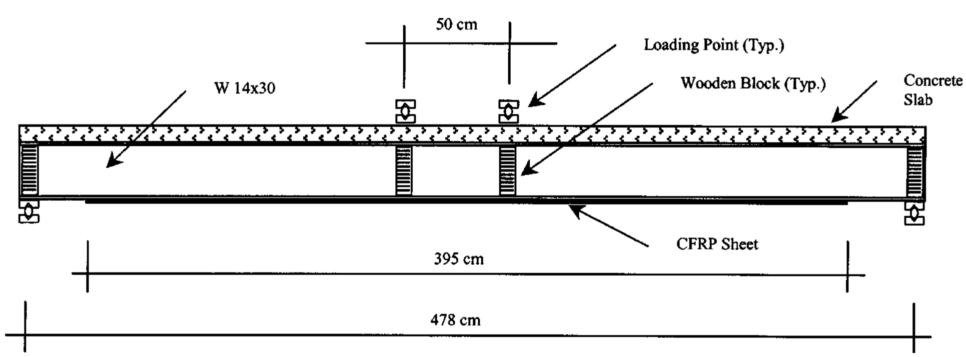

Fig. 5. Schematic of loading setup

Table 1. Constitutive Properties of Materials

Concrete		Steel			CFRP	
Strength (MPa)	16.6	Flange	Yield strength (MPa)	354.9	Strength (MPa)	2137
Modulus (GPa)	13.8		Modulus (GPa)	198.3	Modulus (GPa)	144.0
Peak strain	0.00197	Web	Yield strength (MPa)	381.9	Poisson's ratio	0.34
Failure strain	0.00380		Modulus (GPa)	177.5		
			Poisson's ratio	0.31		

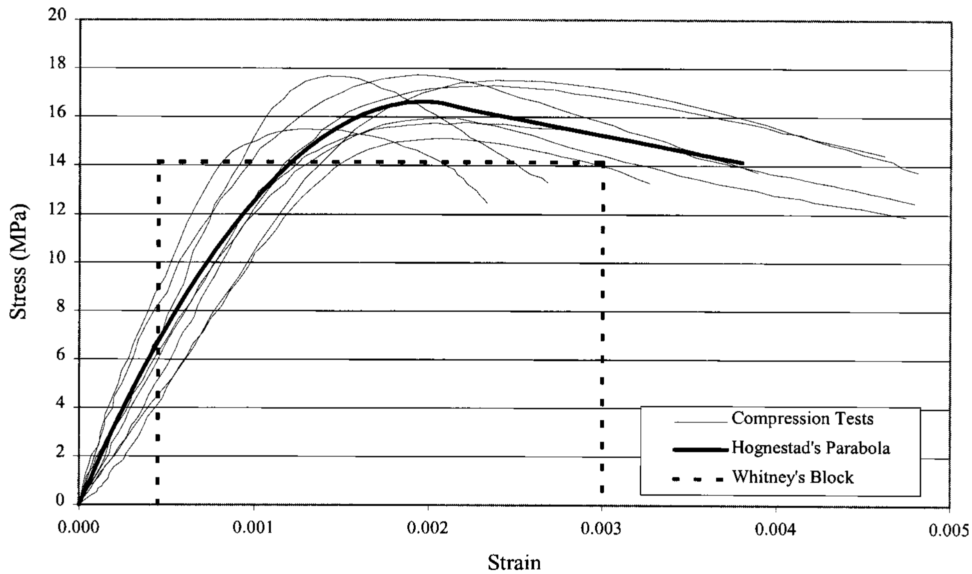

Fig. 7. Constitutive modeling of concrete

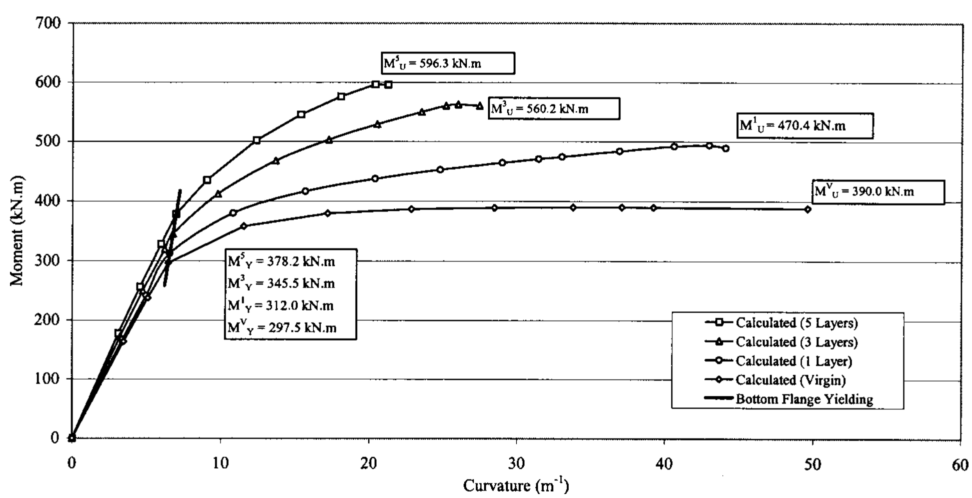

Fig. 8. Theoretical moment-curvature plot (Hognestad)

5. Linear elastic behavior for CFRP; and
6. No shear deformation.

Based on the properties obtained in uniaxial tension and compression tests, the constitutive idealizations of the materials are listed in Table 1. The stress-strain plots for concrete cylinders, the Hognestad's parabola, and the Whitney's rectangular stress block are shown in Fig. 7.

The relationship between moment and curvature of a section was developed by discretizing the concrete slab, top flange, web, and bottom flange into ten strips each with equal thickness and considering the CFRP as one layer. The strain at top of the concrete slab was the primary parameter that varied at each step and the depth of the neutral axis was calculated by trial and error. As a result, at each point a pair of values for moment and curvature was obtained. The ultimate strain for concrete was assumed to be 0.38%. The theoretical moment-curvature plots for the steel-concrete composite girder with different layers of CFRP are shown in Fig. 8. It is evident from these plots that the retrofitting technique can increase the ultimate moment capacity of the sec-

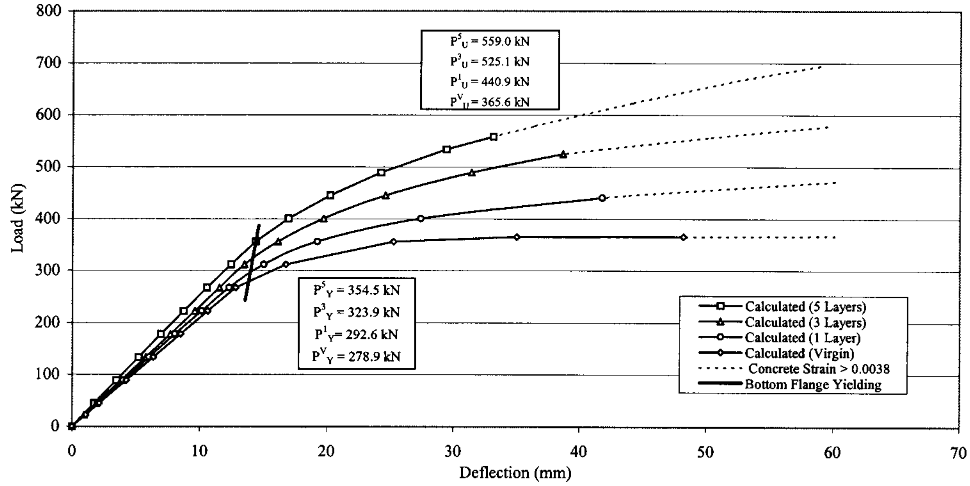

Fig. 9. Theoretical load-deflection plot (Hognestad)

tions from 390.0 kN m for virgin section to 470.4, 560.2, and 596.3 kN m for a one-, three- and five-layer system, respectively. The presence of the CFRP sheets increased the yielding moment of the section from 297.5 kN m for virgin section to 312.0, 345.5, and 378.2 kN m for one-, three- and five-layer systems, respectively. The significant increase in the stiffness of the sections after yielding of steel could be another significant improvement resulting from utilization of this technique.

The load-deflection relationship was then obtained by the moment of area method. The beam was cut into 100 segments with equal lengths. The results of the load-deflection calculations for one-, three-, and five-layer retrofitting systems are shown in Fig. 9. Similar to the moment-curvature plots, all curves were terminated when concrete reached the strain of 0.0038. Parallel to what obtained from moment-curvature relationships, the ultimate load-carrying capacity of the girders improved significantly. The load-deflection relationships can be idealized as bilinear with two distinct stiffnesses of elastic and post-elastic. The elastic stiffness of girders increased from 19.2 MN/m for a virgin girder to 21.8, 23.1, and 24.8 MN/m for one-, three-, and five-layer systems, respectively. Meanwhile, the postelastic stiffness of girders increased from 0.25 MN/m for virgin girder to 2.99, 3.75, and 5.51 for one-, three-, and five-layer systems, respectively. The above procedures were deployed with a macro program in an Excel spreadsheet.

Ultimate Moment Capacity (AASHTO)

AASHTO code uses the Whitney's block approximation to estimate the compression stress in concrete at failure. The ultimate strain in concrete assumed to be 0.003, which was fairly conservative. Using this method and considering different properties for web and flanges, the nominal moment capacity and the ultimate curvature of the sections were obtained. The results are shown in Table 2. By adding CFRP sheets to steel girder, the moment capacity increased significantly and the neutral axis lowered, which reduced the ultimate curvature and ductility of the section. The results of the AASHTO method compared well with the values obtained by the iterative numerical method.

Experimental Analysis

Three composite girders tested in the present study were retrofitted by epoxy bonding of one, three, and five layers of CFRP sheets to the entire length of their tension flanges. They were subjected to monotonic loading with a constant rate of loading of around 2 mm/min with few unloadings in the elastic and the postelastic regions. In the elastic region, data were collected at specified load levels, and after yielding, they were collected at specified deflection levels. The concrete slab supported the compression flange of the steel beam and there was not any local or lateral buckling in the flange of girders before compression failure. The shear studs provided a good composite action between concrete slabs and steel beams without any slippage. The wooden blocks that supported the web under the loading points and at the reactions, prevented any web crippling. The load-deflection plots for the retrofitted composite girders are shown in Fig. 10.

Retrofitting with One Layer of CFRP Sheets

The load-deflection behavior of the retrofitted composite girder is shown in Fig. 11 with a heavy solid line. At the beginning of the experiment, there was a slight nonlinearity in the response due to transverse shrinkage cracks in the slab. At the 75-kN load level,

Table 2. Calculated Ultimate Moment and Curvature of Sections

		Virgin	One layer	Three layer	Five layer
Hognestad	Moment (kN m)	390.0	470.4	560.2	596.3
(theory)	Neutral axis (mm)	102.6	120.9	151.1	179.1
	Curvature ($1/\text{mm}\times 10^{-6}$)	37.1	31.4	25.2	21.2
Whitney	Moment (kN m)	393.7	459.9	537.6	585.6
(AASHTO)	Neutral axis (mm)	98.2	116.4	143.3	163.6
	Curvature ($1/\text{mm}\times 10^{-6}$)	30.5	25.8	20.9	18.3

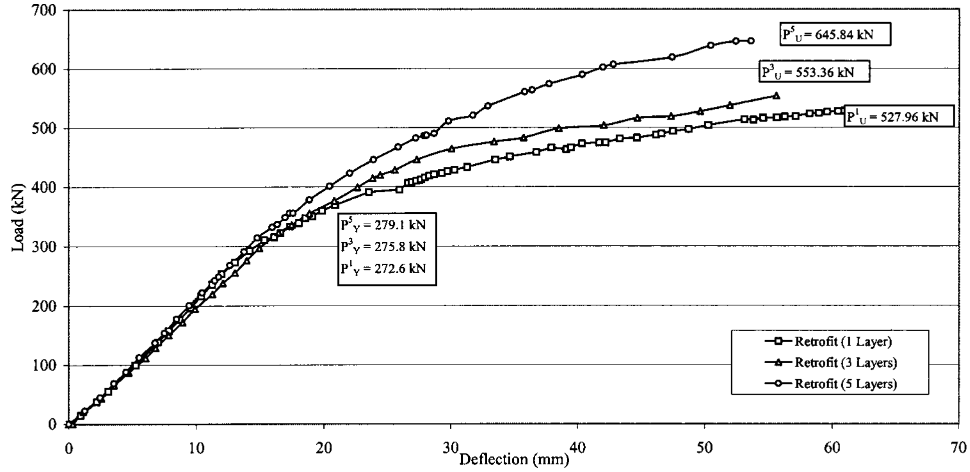

Fig. 10. Load versus deflection of retrofitted composite girders

the cracks were closed and the girder response became fairly linear with the stiffness of 22.77 MN/m, significantly higher than the 17.81 MN/m at the beginning of the test. An unloading at the 200-kN load level exhibited no permanent deformation or nonlinear behavior and traced back the initial plot closely. The extreme fiber of the tension flange yielded at a load of 272.6 kN. At this load level, the stress in the CFRP sheet was 202.3 MPa. Thereafter, the stiffness of the girders gradually decreased and the curve became progressively nonlinear while yielding proceeded through the flange and into the web.

Beyond the 355-kN load level, several longitudinal cracks in the concrete slab and along the edges of the flange started to appear. These cracks tended to increase as load increased and could have been prevented if more transverse reinforcement in the slab had been provided. At load levels of 400 kN, and 500 kN, the girder was completely unloaded and then reloaded. The unloading loops displayed a slight hysteresis especially for the second unloading. In both cases, the loop closed out at the same point at which the girder had been unloaded. The stiffness of the girder in the reloading segments was 20.7 MN/m, slightly lower but comparable to the stiffness at the latter stages of the initial loading. The reason was the complete closure of cracks in the slab after plastic deformation of the specimen.

The girder failed with the crushing of the concrete slab at 528.0 kN. The CFRP stress at the failure was about 1589.8 MPa the values of about 75% of CFRP ultimate strength. Just before failure, the edges of the CFRP sheet started to show signs of rupture as a few strands snapped.

Retrofitting with Three Layers of CFRP Sheets

For this specimen, the tack coat was used to bond three CFRP sheets together and to bond the three-layer laminate to the steel flange. The heavy solid line represents the load-deflection behavior of this retrofitted girder in Fig. 12. The load-deflection behavior of the beam started fairly linear and the unloading in the elastic region at the 200-kN load level displayed no permanent deformation. The measured stiffness of the beam, 20.19 MN/m, was clearly lower than the theoretical values calculated. This is perhaps due to an incomplete composite action between the CFRP layers and steel since the tack coat was more flexible than the epoxy. At a load of 275.8 kN, the tension flange started to yield and the stress in the CFRP sheet reached 192.5 MPa. Afterward, the stiffness of the girders constantly decreased and the curve became continuously nonlinear.

Several longitudinal cracks in the concrete slab along the edges of the top flange of the steel beam started to appear at the

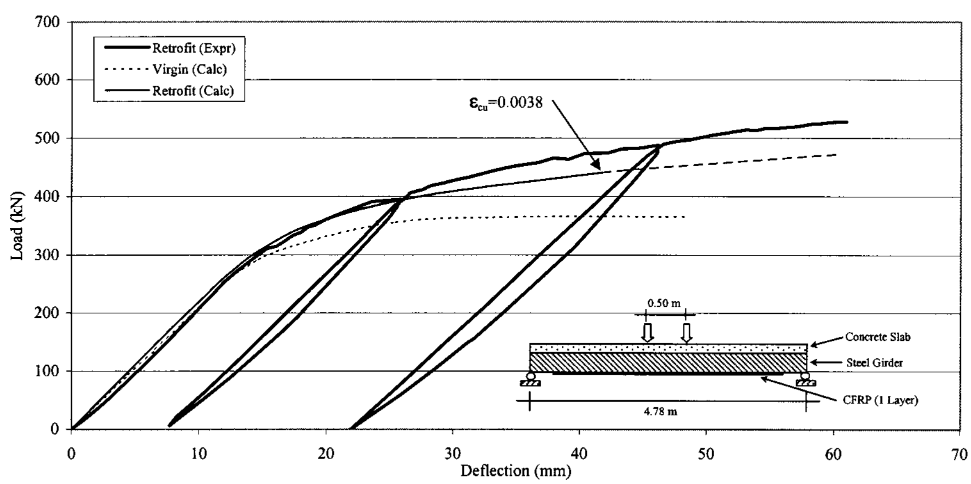

Fig. 11. Load versus deflection of girder retrofitted with one layer of CFRP sheet

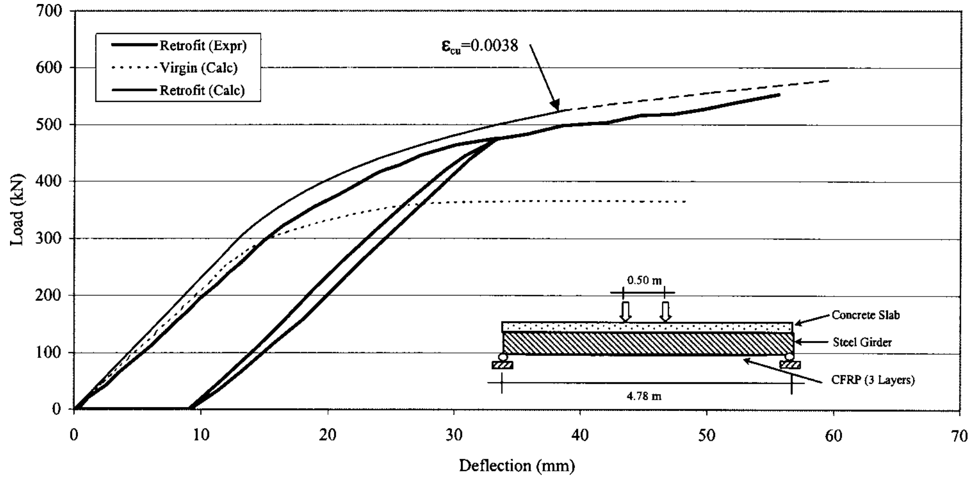

Fig. 12. Load versus deflection of girder retrofitted with three layers of CFRP sheet

400-kN load level. By providing more transverse reinforcements in the slab, these cracks could have been prevented. At a load level of 475 kN a complete unloading and reloading was performed. A slight hysteresis was observed during the loop, but it closed out at the same point at which the girder had been unloaded. The girder in the reloading segments displayed a stiffness of 20.41 MN/m similar to that at the beginning of the initial loading and then showed a slightly higher stiffness of 21.17 MN/m. The shrinkage cracks for this beam perhaps caused the lower stiffness, which even after near 1 cm, permanent deflection had not been completely closed.

The girder failed prematurely by the debonding of the last two layers of the CFRP sheets from the first sheet at the 553.4-kN load level. The stress at the CFRP sheet was about 901.2 MPa, well below its ultimate strength (42%). A tack coat was used in between the CFRP sheets as well as between the CFRP laminate and steel flange. After examining the debonded faces, the lack of complete curing of the epoxy was observed (strong smell of partially cured adhesive). This was due to the improper mixing of the adhesive during construction of specimens. This problem was corrected in the last specimen.

Retrofitting with Five Layers of CFRP Sheets

As was mentioned before, the issue of the premature failure of the three-layer retrofitted girder was corrected for this specimen. The CFRP sheets were sand blasted harsher to expose the carbon fibers and the lower viscosity epoxy was mixed well and used in between layers. The tack coat was mixed thoroughly and only used to attach the five-layer CFRP laminate to the steel flange.

The load-deflection behavior of the retrofitted girder is shown in Fig. 13 with a heavy solid line. The initial nonlinearity due to the transverse cracks in the concrete slab existed in this girder as well. The cracks were closed at the 100-kN load level and the beam started to show a higher stiffness of 21.99 MN/m compared to 19.61 MN/m as for the beginning. An elastic unloading after reaching the 200-kN load level showed no permanent deformation or hysteresis effects. The yielding of the tension flange started after reaching a load of 279.1 kN. The CFRP sheet was experiencing a tensile stress of 184.9 MPa at that point. After yielding of the steel started, the stiffness of the girder gradually decreased and yielding extended to the web as well as to the rest of the flange.

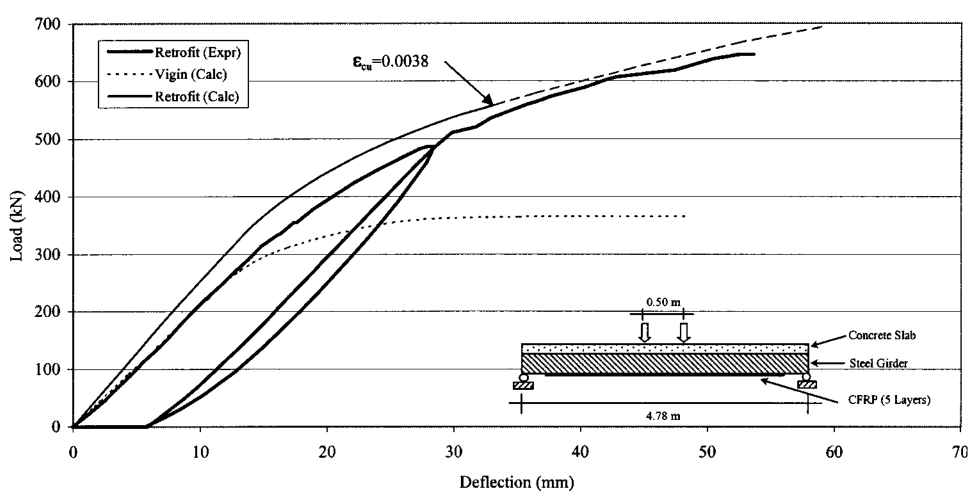

Fig. 13. Load versus deflection of girder retrofitted with five layers of CFRP sheet

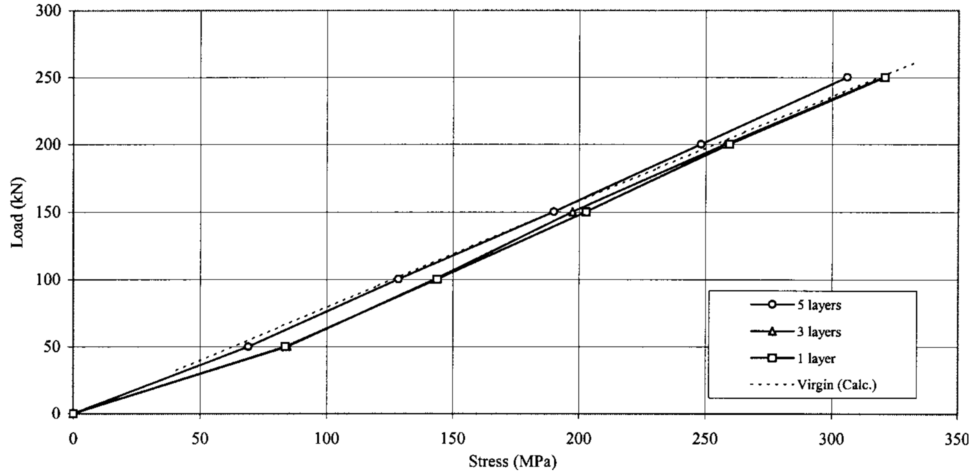

Fig. 14. Load versus stress in flange for different retrofitted girder (prior to yielding)

Several longitudinal cracks similar to those in previous beams appeared after reaching the 425-kN load level. These cracks started to grow and widen significantly as loading increased. A complete unloading and reloading was conducted around the 475-kN load level. This displayed a modest hysteresis very similar to that in previous girders, and like others, the loop closed out at the unloading point. A slightly higher stiffness of 22.76 MN/m was observed after unloading.

The girder failed with the crushing of the concrete slab and progressive debonding of the slab from the steel beam at one end due to the excessive widening of longitudinal cracks at 645.8 kN. The CFRP stress at the failure was about 903.7 MPa, which is far below its ultimate strength (42%). The shear plains in the tension flange and web of the steel beams were clearly visible toward the end of the experiments. After concrete failure, the compression flange of the steel beam started to buckle due to disintegration of supporting slab.

Effect of Retrofitting on Stress and Strain Reduction

Increasing the cross-sectional area of the section could reduce the stress level in the tension flange of a girder. The tensile modulus of the CFRP sheets was about 2/3 of the modulus of steel and the thicknesses of these sheets were moderately small. The adhesive layer that connected the two materials was relatively soft. Therefore, not a significant improvement in the stiffness of the girder in the elastic region was anticipated. Fig. 14 shows the variation of the stress level in the tension flange as a function of the load before yielding for different retrofitting system. The theoretical values of stiffnesses and strains did not conform to the experimental results in the elastic region due to shrinkage cracks. Therefore, the results were compared to each other rather than to the virgin girder. The five-layer system displayed the most reduction in stress with an average reduction of 13.7 MPa compared to the other two retrofitting systems. One- and three-layer systems displayed an insignificant stress reduction in the elastic region.

After yielding, the effectiveness of the CFRP laminates was much more profound as shown in Fig. 15. Tensile strains in the tension flanges of the retrofitted composite girders decreased significantly. The tensile strain in the flange decreased with an average of 20.6, 39.2, and 52.6% for one-, three-, and five-layer systems compared to the virgin girder at 350 kN. For higher-load levels, the effectiveness of the retrofitting was apparent. The significant improvement could limit the permanent deformation of the girders experiencing unexpected and sudden overload.

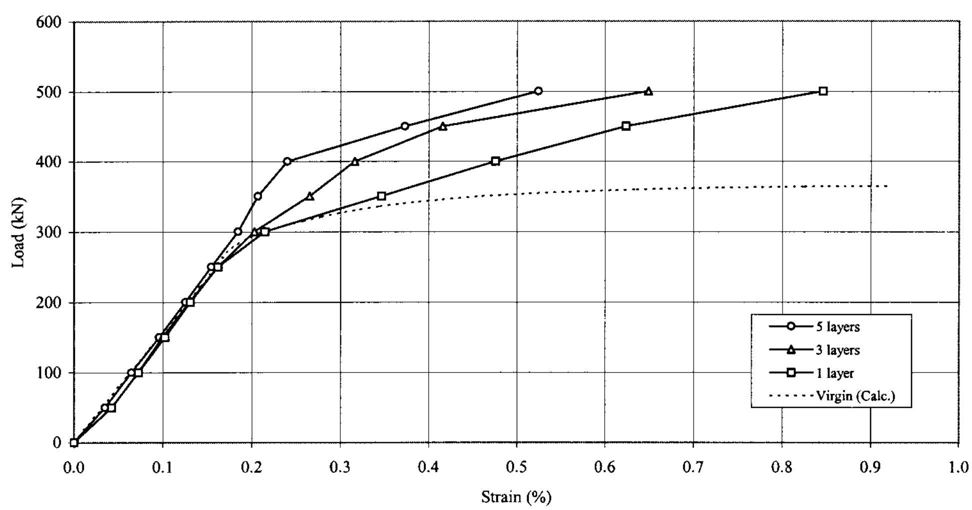

Fig. 15. Load versus strain in the flange for different retrofitted girder

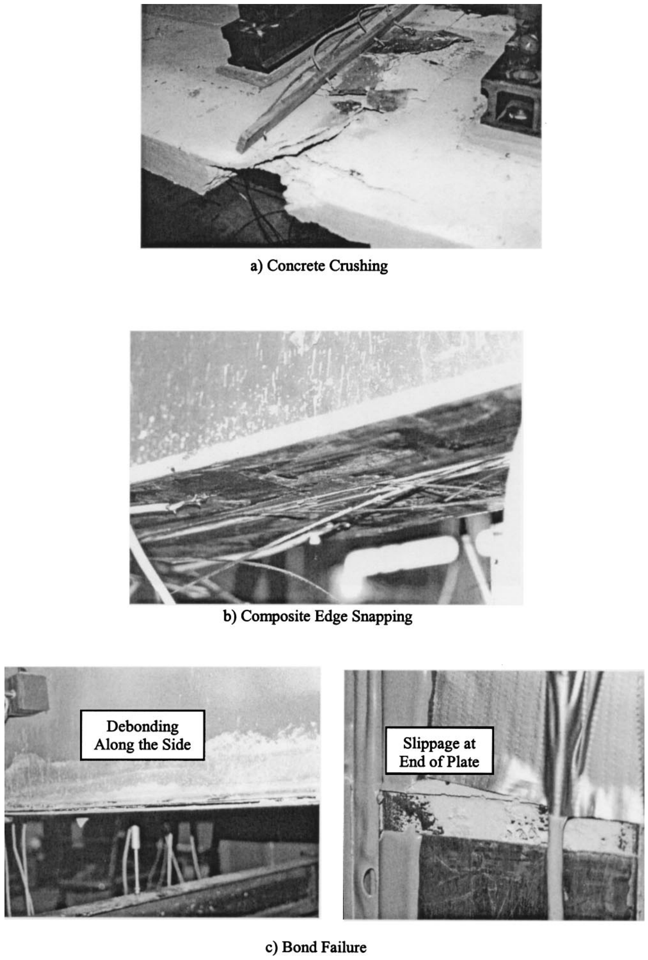

Fig. 16. Failure modes: (a) concrete crushing; (b) composite edge snapping; and (c) bond failure

Failure Modes

As indicated before, the steel-concrete-CFRP system could display several distinct failure modes including: concrete crushing; CFRP debonding; CFRP rupture; web crippling; and shear stud failure. Meanwhile, the wooden blocks placed between the flanges at support, and under loading points prevented web crippling. Shear studs were designed for a slab with 35-MPa compressive strength, well above the measured strength of the concrete used in this study.

Compression crushing of concrete, as shown in Fig. 16(a), was the dominating failure mode in all three retrofitted girders. In the one-layer system, the concrete failure happened when the CFRP sheets started to show signs of failure by snapping along their edges as shown in Fig. 16(b). In the three-layer system, the debonding of the CFRP sheet caused a premature failure as shown in Fig. 16(c). The partially cured adhesive layer, due to an improper mixing ratio, was believed to be the main contributor of that. After corrections were made, the five-layer system did not display any premature failure and, again the compression crushing of concrete became the failure mode of that girder.

Conclusions

Test results of steel-concrete composite girders retrofitted with epoxy-bonded CFRP laminates are very promising. For all three retrofitted girders considered in this study, this technique improved the ultimate load-carrying capacity significantly. The effect of CFRP laminates on the plastic stiffness was also notable. Based on the results of the experimental investigation, the following conclusions are drawn:

1. Ultimate load-carrying capacities of the girders significantly increased by 44, 51, and 76% for one-, three-, and five-layer retrofitting systems. In addition, the yield load of the girders continuously increased as a result of retrofitting (272.6, 275.8, and 279.1% kN for one-, three-, and five-layer systems);
2. The effect of CFRP bonding on the elastic stiffness of the girders was not significant, due to the flexibility of the adhesive;
3. As the number of CFRP layers increased, the efficiency for utilizing the CFRP sheet decreased. Stress in the CFRP laminate for the one-layer system was 75% of its ultimate strength while in the five-layer system, it dropped to 42%. This indicates that a balanced design should be considered to effectively utilize the strength of CFRP laminates;
4. The analytical models using the Hognestad's parabola as well as the AASHTO's method both provided conservative results for predicting the ultimate capacity, by 13 and 17%, respectively;
5. The theoretical values of both elastic and postelastic stiffnesses were also conservative but fairly accurate;
6. While the analytical models indicated that the ductility of the retrofitted system was less than the virgin girders, all three retrofitted girders deflected between 50 and 60 mm, which is about 1/100–1/80 of the clear span; and
7. The effect of retrofitting on the stress level in the tension flange in the elastic region was insignificant (5% difference between the one- and five-layer systems at 200 kN). In the postelastic region, the significant reduction in tensile strain was observed (21, 39, and 53% for the one-, three-, and five-layer systems, respectively).

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