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STRENGTHENING STEEL-CONCRETE COMPOSITE BRIDGES WITH HIGH MODULUS CARBON FIBER REINFORCED POLYMER (CFRP) LAMINATES

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ABSTRACT: The deterioration of steel bridges is a severe problem that often results in restrictive load ratings, costly repairs of the steel structural elements, and early replacement of entire structures. Strengthening and rehabilitation of steel structural elements by bonding high modulus carbon fiber reinforced polymer (CFRP) laminates has been investigated for steel-concrete composite beams that are typical of those used for bridge structures. Two series of flexural tests have been conducted. Large-scale beams, spanning 6400 mm have been strengthened using carbon fiber with an elastic modulus of either 640 GPa or 440 GPa. One of these beams was strengthened using CFRP laminates that were prestressed prior to bonding. This system has shown the potential to increase the flexural stiffness of the beam more than an equivalent amount of unstressed laminate, while preserving its full ductility. The second series of five smaller beams, spanning 3050 mm, investigated in detail the strain distribution of the strengthened beams compared to an unstrengthened beam. The effect of overloading and subsequent unloading has also been examined.

1. INTRODUCTION

1.1 Research Objective

Severe deterioration of steel bridge girders in service, in conjunction with demands for increases in allowable truck weights and limited funding for new bridge structures, has created the demand for a reliable and durable system for strengthening these structures. A feasibility study was initially conducted to determine the potential to strengthen and provide stiffness increases at service load levels for steel-concrete composite beams that are typical of bridge structures. This work involved the use of new, high modulus, CFRP materials. Continuing work is now focused on addressing the problems of shear-lag, overloading and fatigue for steel members strengthened with these high modulus CFRP materials.

1.2 Previous Work

Considerable previous work has been focused on the use of standard modulus CFRP materials for strengthening of steel structures. Several studies have shown the possibility of strengthening steel girders with CFRP materials [1], repairing corroded girders removed from service [2] or repair of girders that simulated corrosion by reduction of the cross-section [3,4]. While these studies have shown significant increases in the ultimate strength or restoration to the original strength for corroded sections, no significant increase in stiffness in the service range has been observed. This is due to the similar stiffness between the CFRP laminates used in these studies and the steel, which requires that a great deal of strengthening be applied. However, the efficiency of any bonded system decreases as laminate thickness increases or multiple plys are used. Use of higher modulus CFRP material has the potential to provide stiffness increases in the service range, using CFRP laminates of a reasonable thickness.

1.3 High Modulus Carbon Fiber Material

Two types of unidirectional, pultruded CFRP laminates were used in the experimental program. Diversified Composites custom pultruded the CFRP laminates, designated DC-I, using an intermediate modulus (440 GPa) DIALEAD K63312 fiber produced by Mitsubishi Chemical America, Inc. Epsilon Composite manufactured the other type of laminate using the higher modulus (640 GPa) DIALEAD K63712 fiber, designated THM-450, which was a standard product. Material properties for the laminates produced by Diversified Composites were determined by the producer. An external laboratory, determined the properties for the laminates produced by Epsilon Composites. The values given for the THM-450 laminates are the average of tests results for the same laminates pultruded to two different thicknesses. These values are listed in Table 1.

Table 1 Properties for the two types of CFRP laminates used in the experimental program

Property	DC-I	THM-450
Laminate thickness (mm)	3.2	2.9 or 4.0
Tensile strength (MPa)	1224	1534
Tensile modulus (GPa)	229	457
Ultimate elongation (percent)	0.508	0.332

2. PHASE 1: LARGE-SCALE STEEL-CONCRETE COMPOSITE BEAMS

2.1 Test Specimens

The experimental program included three large-scale beam specimens that were fabricated from grade A992 steel W310 x 45 sections, 6550 mm in length. These beams had an average yield strength of 373 MPa. Shear studs that were 76 mm in length and 19 mm in diameter were welded at a spacing of 152 mm along the length of the compression flange to ensure full composite action between the concrete deck and the steel section. These studs were also staggered to minimize the possibility of deck splitting. Concrete strength, determined at the time of testing, varied between 37 and 44 MPa for all the large-scale specimens. Longitudinal reinforcement of the deck was provided by grade 60 steel reinforcing bars that were 12.7 mm in diameter. The spacing of the longitudinal concrete reinforcement varied as shown in Figure 1 and was continuous over the full length of the beam. Transverse reinforcement, using the same bars, was provided at a constant spacing of 152 mm. All the reinforcing bars were determined to have a yield strength of between 436 and 463 MPa using the 0.2% offset method.

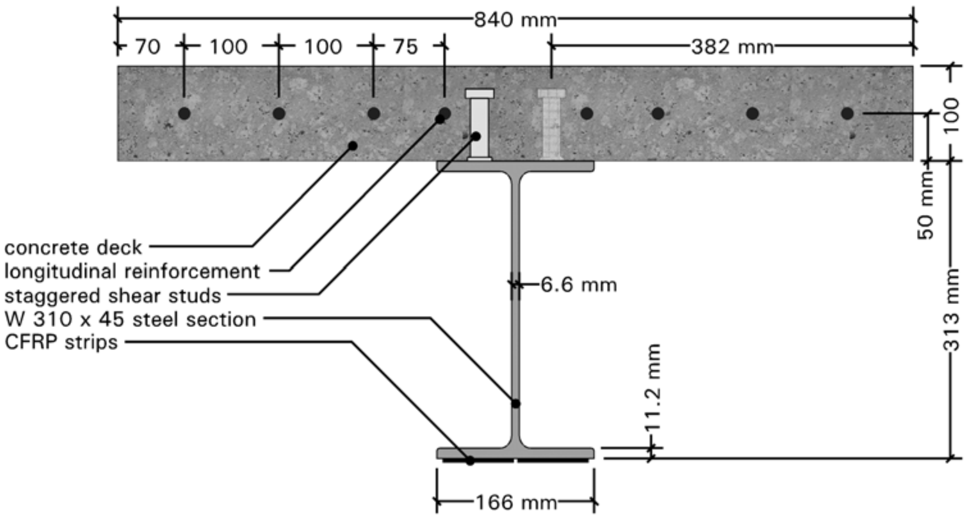

Figure 1 Cross-section dimensions for large-scale steel-concrete composite beams

2.2 Test Matrix

Different strengthening configurations were used for each of the three large-scale beams. The designation of each specimen was given based on three different parameters. The first parameter indicated the modulus of the fiber used for the strengthening, whether intermediate modulus or high modulus. The second parameter gives the reinforcement ratio of the applied strengthening, defined as the cross-sectional area of longitudinal fiber, taking into account the fiber volume fraction of the laminates used, divided by the cross-sectional area of the steel section. The final parameter lists whether the strengthening was adhesively bonded without stressing the CFRP laminates or if the laminate was first prestressed. The test matrix is listed in Table 2.

Table 2 Test matrix for large-scale steel-concrete composite beam specimens

Designation	CFRP type	Reinforcement ratio	Application method
IM-4.5-AB	DC-I	4.5%	adhesive bonded CFRP laminates
HM-7.6-AB	THM-450	7.6%	adhesive bonded CFRP laminates
HM-3.8-PS	THM-450	3.8%	prestressed CFRP laminates

2.3 Test Procedure

The large-scale steel-concrete composite beam specimens, spanning 6400 mm, were tested in four-point bending under displacement control, with loads applied symmetrically about the center of the span. The constant moment region was 1000 mm in length. Neoprene pads were used at the supports and under the load points to allow for rotation. Loading was applied across the full width of the concrete slab by the use of 150 mm deep steel HSS sections that were placed between the spreader beam and the neoprene pads. Each of the beams was subjected to three loading cycles. The first cycle of loading, was to induce a strain of 0.12 percent (or about 60 percent of the yield stress) at the level of the tension flange before the beam was strengthened. After strengthening and curing of the adhesive, the next loading was to the same stress. Finally, the beam was reloaded to its ultimate strength.

2.4 Results and Observations

The behavior in the service range was examined by loading each of the beams to a tension strain of 0.12 percent at the bottom of the steel tension flange before and after strengthening. Table 3 shows the results in the elastic range for the three large-scale beams. As was expected, use of higher modulus fiber in conjunction with the highest reinforcement ratio resulted in the highest stiffness increase. For beam HM-3.8-PS strengthened with the prestressed CFRP laminate, even though only half the amount of CFRP material was used, a similar stiffness increase was recorded in the elastic range. This takes into account the 2.0 mm of camber that was induced during prestressing before any load was applied to the beam.

Table 3 Stiffness improvement at induced strain of 0.12 percent for steel-concrete composite beams

		Unstrengthened		Strengthened	
Specimen	Stiffness increase	Load (kN)	Net midspan deflection (mm)	Load (kN)	Net midspan deflection (mm)
IM-4.5-AB	10 %	150	15.8	173	16.3
HM-7.6-AB	36 %	152	15.0	226	16.4
HM-3.8-PS	31 %*	151	15.2	184	14.1*

The load-deflection behavior of each of the beams to ultimate is shown in Figure 2. The behavior of each of the three beams was essentially bilinear until rupture of the CFRP laminates, with only a slight decrease in stiffness occurring once the steel reached yield. Similar to previous findings [5,6], most of the strength increase occurred in the region between yielding of the steel and rupture of the CFRP. No debonding of the CFRP laminates was evident, indicating that the bonding and surface preparation technique used was satisfactory.

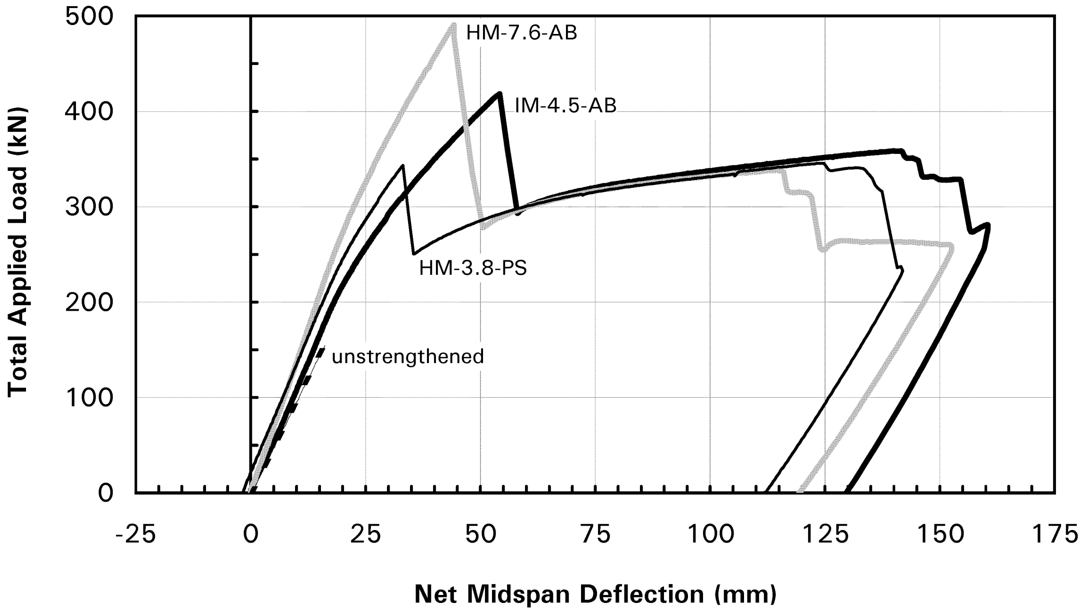

Figure 2 Load-deflection behavior of the three large-scale steel-concrete composite beams

Once the laminates ruptured, they were no longer effective in strengthening the beams and the beams reverted to their unstrengthened state as evident by the overlap in the load-deflection behavior for all the beams after rupture. Continued loading resulted in concrete crushing. This allows the comparison to be made between the ultimate strength of the strengthened beam at rupture of the CFRP and the strength of the (now unstrengthened) beam at concrete crushing, shown in Figure 3. For the beams strengthened using the CFRP plates that were not prestressed, increases in ultimate strength were possible, as indicated in Table 4. The prestressed beam was designed to provide the maximum stiffness increase, without increasing the ultimate strength of the section, which may be advantageous in cases where it is desired to maintain the full ductility of the member. This was accomplished by designing the strengthening such that the CFRP laminates would rupture near flexural yield of the cross-section.

Table 4 Properties at ultimate for steel-concrete composite beams

		At rupture of the CFRP laminates		At concrete crushing	
Specimen	Ultimate strength increase	Load (kN)	Net midspan deflection (mm)	Load (kN)	Net midspan deflection (mm)
IM-4.5-AB	16 %	418	54.2	359	141.5
HM-7.6-AB	45 %	491	44.3	338	114.6
HM-3.8-PS	none	343	33.1*	346	124.4*

* accounting for the beneficial effect of the camber

Figure 3 Steel-concrete composite beam HM-7.6-AB after failure

3. PHASE 2: SHEAR LAG, OVERLOADING AND FATIGUE

3.1 Test Specimens

Seven beams were fabricated to investigate the effect of shear-lag, overloading and fatigue on steel-concrete composite beams strengthened with CFRP. Two different reinforcement ratios were considered. One beam was tested monotonically to investigate the possible presence of shear-lag effects. Three beams were loaded and unloaded at different displacement levels to simulate overloading conditions. Three beams are being tested under fatigue loading conditions at the same stress range in the steel tension flange, to study the fatigue behavior of the retrofit. To examine the effect of adhesive thickness on the fatigue life of the system two of the fatigue beams were strengthened using the same reinforcement ratio but two different adhesive thicknesses were applied to the two beams. Additionally, a silane primer coating was applied to the beam with the thicker adhesive to investigate the effect of silane on the fatigue durability of the strengthening system. The third fatigue beam was unstrengthened and tested as a control beam. This paper presents the shear-lag and overloading beams and the preliminary fatigue results. The test matrix is presented in Table 5.

Table 5 Shear-lag, overloading and fatigue test matrix

Designation	CFRP Type	Reinforcement ratio*	Adhesive Thick.	Loading
SHL-2	THM 450	8.8%	0.1 mm	Monotonic to Failure
ST-CONT	None	0.0%	N/A	Load/Unload
OVL-1	THM 450	4.5%	0.1 mm	Load/Unload
OVL-2	THM 450	8.8%	0.1 mm	Load/Unload
FAT-CONT	None	0.0%	N/A	3 mill. cycles (0.3 ε_y - 0.6 ε_y)
FAT-1	THM 450	4.5%	0.1 mm	3 mill. cycles (0.3 ε_y - 0.6 ε_y)
FAT-1b	THM 450	8.8%	1.0 mm	3 mill. cycles (0.3 ε_y - 0.6 ε_y)

The test specimens, shown schematically in Figure 4, were fabricated from grade A992 steel W200 x 19 sections. The overall beam length was 3.35 mm. Pairs of transverse stiffeners were welded at the supports and at the load points to prevent local web failure. A 65 mm x 525 mm concrete slab was cast on the top of each specimen. The measured concrete strength at 28 days was between 30 and 43 MPa for the different specimens. Shear interaction between the steel beams and concrete slab was provided by two parallel rows of 12 mm x 50 mm shear studs welded 50 mm apart along the entire length of the beams. The studs were spaced longitudinally at 100 mm on center. The slab was reinforced with 100 mm on center mm plain welded wire fabric to prevent shrinkage cracking and longitudinal splitting of the deck.

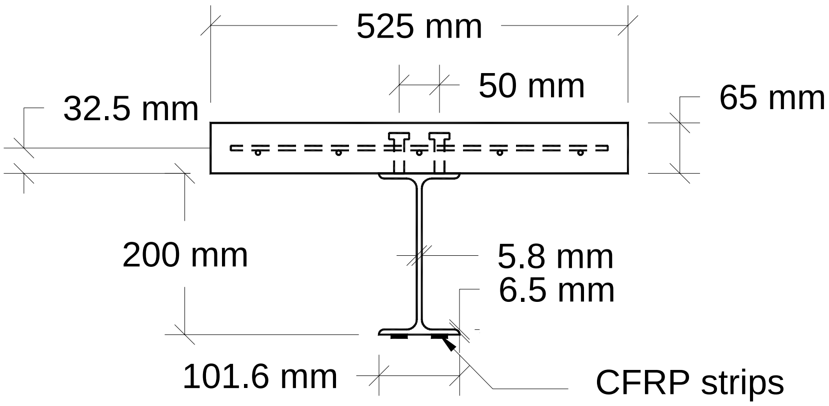

Figure 4 Cross-section of shear-lag and overloading specimens

3.2 Test Procedure

The small-scale beams were tested in four-point bending with a span of 3.05 m. Load points were each 305 mm on either side of mid-span. For the shear-lag and overloading specimens a pin and a roller support were provided on either end of the beam while 75 mm thick neoprene pads were used for the fatigue specimens. Load was applied from a spreader beam to two HSS tubes across the entire deck through neoprene pads in a similar manner to that used for loading of the large-scale beams. To simulate actual service conditions, the fatigue specimens will be strengthened after application of dead load. Dead load will be applied to the beams through two HSS tubes across the entire deck by tightening nuts on threaded rods that will be attached to the laboratory strong floor. After curing of the adhesive, load will be transferred to the actuator, which will apply the fatigue loading. The effectiveness of the system has been verified on beam FAT-CONT.

3.3 Results and Observations

The load deflection relationships of the shear-lag and overloading beams are presented in Figure 5. Rupture of the CFRP occurred at a load of 259 kN for beam OVL-1, and a load of 357 kN for beams OVL-2 and SHL-2. Ultimate failure of the beams was due to crushing of the concrete at loads of 216 kN for beams OVL-1 and OVL-2 and 220 kN for beams ST-CONT and SHL-2. Unloading and reloading of beams ST-CONT, and OVL-1 and OVL-2 prior to rupture of the CFRP, was linear with stiffness losses between five percent and ten percent as compared to the initial stiffness of the beams. After rupture of the CFRP unloading and reloading of beams OVL-1 and OVL-2 remained linear with a post-rupture stiffness equal to that of the unstrengthened beams.

Residual deflections were measured after unloading for all three of the overloading beams. Figure 6 presents the residual deflection induced after loading to a certain applied load level and unloading to the dead load level. The dead load was selected as the load inducing a stress of 0.3 F_y in the tension flange of the unstrengthened beam. The service loads for the three specimens, based on an acceptable tension flange strain of 0.6 F_y, are also presented. Figure 6 demonstrates that residual deflections after loading to service load levels for all three beams are less than span/4000. When the beams were loaded beyond the yield strain of the steel the strengthened beams exhibited much lower residual deflections than the control beam.

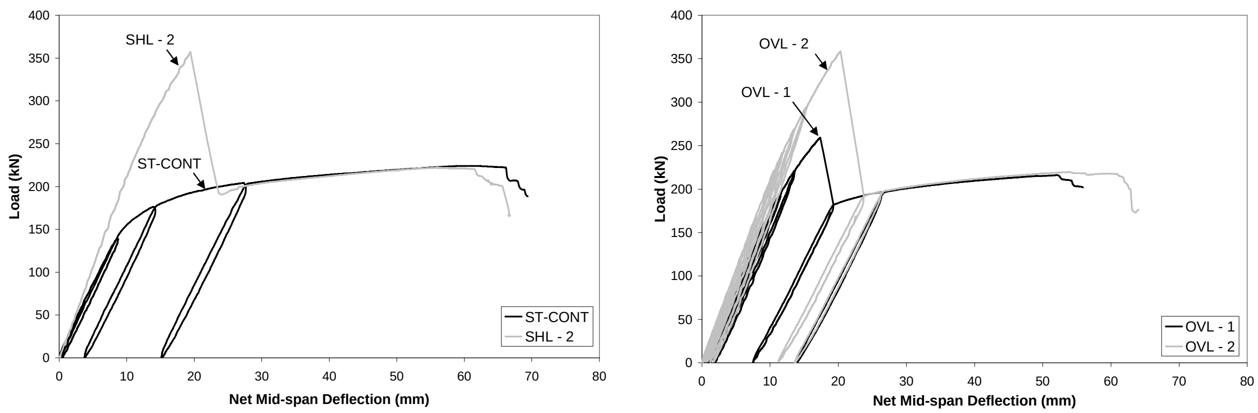

Figure 5 Load deflection relationships for shear-lag and overloading specimens

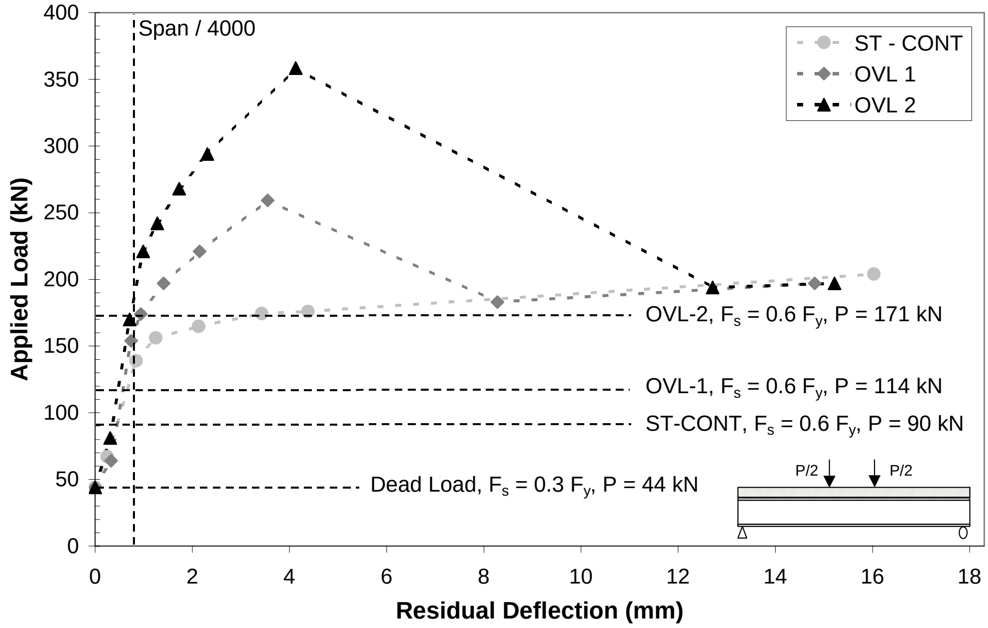

Figure 6 Residual deflections for overloading specimens

The effect of shear-lag was determined by considering the strain profiles through the cross-section of the beams. Beams ST-CONT and OVL-1 exhibited linear strain profiles at mid-span indicating that shear-lag effects were minimal. Minor strain discontinuities were measured in beams SHL-2 and OVL-2 at the steel-CFRP interface. The measured discontinuity of strain is likely due to residual stresses and local variations of the steel strain due to lateral distortion in the flanges and web of the steel beam, which was confirmed by other instrumentation used in the tests.

3.4 Fatigue Behavior

Three beams similar to those used in the shear-lag and overloading study will be used to examine the fatigue behavior of beams strengthened with CFRP. Two beams will be strengthened while the third will remain unstrengthened to serve as a control beam. To simulate actual bridge conditions, a dead load will be applied to induce 0.3 ε_y in the tension flanges of the beams prior to strengthening of beams FAT-1 and FAT-1b. The dead load will be maintained throughout the fatigue testing. Fatigue cycling will range from the dead load level to the service load level including live load and impact calculated as the load inducing a stress in the tension flanges of the beams of 0.6 ε_y. Preliminary experimental results demonstrate that using a reinforcement ratio of 4.5 percent can increase the live load by 17 percent without reducing the fatigue life of the beam.

Beam FAT-CONT exhibited minimal stiffness degradation after being subjected to 3 million loading cycles between 50 kN and 100 kN. Prior to fatigue loading, these loads induced strains in the steel tension flange of 0.062 percent and 0.11 percent corresponding to 0.33 ε_y and 0.6 ε_y respectively. The beam, did, however, exhibit fatigue-creep behavior in which the mean deflection of the beam increased with continued fatiguing without affecting the global stiffness of the member. Minor cracking of the concrete slab and yielding of the steel near stiffener details was observed during the fatigue cycling of the beam, however, this does not appear to have substantially affected the global stiffness of the beam. Testing of this beam demonstrated the effectiveness of applying dead load prior to fatigue loading.

Beams FAT-1 and FAT-2 will be strengthened using the dead load setup described previously and tested to the same strain ranges as beam FAT-CONT to evaluate the allowable service load increases based on fatigue criteria and to determine the fatigue life of the retrofit system.

4. CONCLUSIONS

High modulus CFRP material can be used to increase the stiffness and flexural strength of steel-concrete composite flexural members typically used for bridges and structures. The material can be externally bonded to the tension zone or prestressed before being bonded to the tension zone. Use of prestressed CFRP can significantly increase the stiffness under service loading conditions while maintaining the ductility of the original member. The experimental results indicate that the use of high modulus CFRP materials can greatly reduce the overloading damage due to overloading conditions, in comparison to unstrengthened beams loaded to similar stress conditions. Test results indicate that the use of the selected adhesive for the system can achieve excellent bond, with insignificant shear lag at the interface between the steel surface and the high modulus CFRP material.

5. ACKNOWLEDGEMENTS

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