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High modulus carbon fiber materials for retrofit of steel structures and bridges

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Abstract

This paper summarizes the research work dealing with the use of high modulus carbon fiber reinforced polymer (HM CFRP) materials for the retrofit of steel structures and bridges. The research work included selection of an appropriate adhesive for bonding HM CFRP materials to steel and the performance of Large-scale steel-concrete composite beams tested to examine the behavior of different strengthening schemes. The research findings demonstrate the effectiveness of the proposed HM CFRP strengthening system.

1. Introduction

Fiber reinforced polymer (FRP) materials are commonly used for the repair and strengthening of concrete structures. Due to the success of this technique, several researchers have investigated the use of externally bonded carbon FRP (CFRP) materials to strengthen steel beams. A number of different approaches have been investigated to assess the effectiveness of various CFRP materials for the strengthening and repair of steel bridges and structures, including repair of overloaded girders (Sen et al., 2001), repair of naturally deteriorated girders (Mertz and Gillespie, 1996), strengthening of undamaged girders (Tavakkolizadeh and Saadatmanesh, 2003) and repair of girders with simulated corrosion damage (Al-Saidy et al., 2004). The previous research has demonstrated that conventional modulus CFRP materials can be effectively used to increase the yield strength and ultimate capacity of steel and steel-concrete composite beams. However, due to the relatively low modulus of elasticity as compared to steel and also possibly due to the presence of a shear-lag effect between the steel surface and the CFRP, a large amount of CFRP material may be required to achieve an adequate increase of the elastic stiffness. Other research has demonstrated that the fatigue durability of CFRP strengthened steel beams is at least comparable to that of typical steel details which are commonly used (Miller et al., 2001).

This paper presents the findings of an experimental program, which was conducted over four years, to develop a high modulus CFRP (HM CFRP) system for strengthening and repair of steel bridges and structures. The first phase of the system development includes the selection of an appropriate saturating resin to be used for wet lay-up of the dry fibers as well as selection of an appropriate adhesive for externally bonded, pre-cured HM CFRP laminates. In the second phase of the research, three large-scale steel-concrete composite beams were strengthened and tested to study the effectiveness of various configurations of HM CFRP materials to increase the strength and stiffness of typical highway bridge girders. The third and fourth phases of the experimental program examined the behavior of the strengthened beams under overloading conditions and fatigue loading conditions. The research also investigated the possible presence of a shear-lag effect between the steel surface and the HM CFRP materials. Currently, the ongoing research is considering the bond behavior of CFRP-to-CFRP bonded joints in order to determine the required overlap lengths and joint details for spliced connections to facilitate strengthening of longer-span girders using finite lengths of the material. Based on the findings of the experimental program, guidelines are proposed for the design of HM CFRP strengthening for typical steel flexural members for a specified increase of the live load carrying capacity.

2. HM CFRP Materials

High modulus carbon fiber materials are commonly produced as dry fiber tow sheets. The sheets can be impregnated with a saturating resin on-site using a wet lay-up technique and are well suited for curved applications or highly irregular surfaces. For applications requiring a higher degree of strengthening, the HM carbon fibers can also be pultruded into a precured laminate which can be subsequently bonded to the surface of the structure using a structural adhesive. The typical material properties of the dry fibers and pultruded laminate HM CFRP materials are given in Table 1.

Table 1: HM carbon fiber material properties

	Fiber Properties (Mitsubishi, 2004)	Laminate Properties
Tensile modulus, E	640 GPa	460 GPa
Tensile strength, f_u	2600 MPa	1540 MPa
Rupture strain, ε_u	0.004	0.0033
Fiber volume fraction	-	70%

3. Phase 1: Resin and Adhesive Selection

The first phase of the system development focused on the selection of appropriate adhesives to bond the HM carbon fiber materials to steel surfaces. This included selection of a saturating resin to bond the dry fiber tow sheets and selection of an appropriate structural adhesive for bonding the pultruded CFRP laminates (Schnerch, 2005).

The relative performance of ten different saturating resins was compared through a series of double lap shear coupon tests. The typical test specimens are shown schematically in Figure 1. The resins were allowed to cure at room temperature for at least seven days after which the coupons were loaded in axial tension to failure. The average shear strength of the resins was used to evaluate their relative performance. For the best performing resins, failure of the coupons was predominantly by rupture of the carbon fibers indicating complete utilization of the HM CFRP materials. An average shear stress of 12 MPa was achieved for these resins prior to rupture of the fibers. Some pull-out failures were also observed which may have been due to incomplete wetting of the fibers by the resin. The use of a wetting agent was investigated to improve the saturation of the fibers and increase the capacity of the bond, however, no significant improvement of the performance was observed. The poor performance of some of the resins was due to debonding of the CFRP from the steel surface. The use of an elevated temperature cure cycle was also investigated to enhance the performance of the resins, however, no significant improvement of the performance was observed.

In order to select an appropriate adhesive to bond HM CFRP laminates to steel surfaces small scale flexural tests were conducted. These tests represent the bond stress distribution which is typically induced in beam applications. The test specimens consisted of an 813 mm long steel wide flange beam with a steel plate welded to the compression flange to simulate the presence of a composite concrete deck slab. The beams were strengthened by bonding 36 mm wide x 1.4 mm thick CFRP laminates of varying lengths to the bottom of the tension flange. A typical test specimen used in this phase is shown schematically in Figure 2. The beams were subsequently loaded to failure in a four point bending configuration. A total of six different structural adhesives were evaluated using this test configuration. Of the six adhesives tested, the Spabond 345 two part epoxy adhesive distributed by SP Systems North America was capable of developing the full rupture strength of the 1.4 mm thick HM CFRP strips within a development length of 102 mm. The development length of the strips was measured from the end of the strip to the nearest load point as shown in Figure 2. Additional tests indicated that two plys of the HM CFRP laminates required twice the development length to achieve rupture of the fibers. Due to the relatively short development length required, this adhesive was selected for further development as a part of the HM CFRP strengthening system for steel bridges and structures.

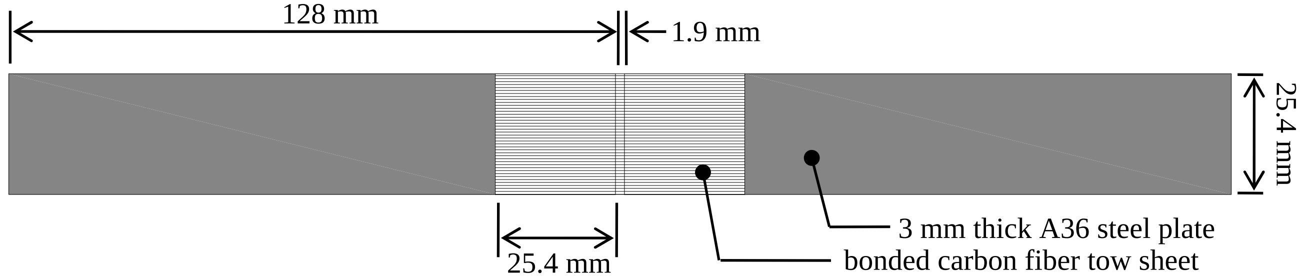


Figure 1: Typical double lap shear coupon for selection of the saturating resin (adapted from Schnerch, 2005)

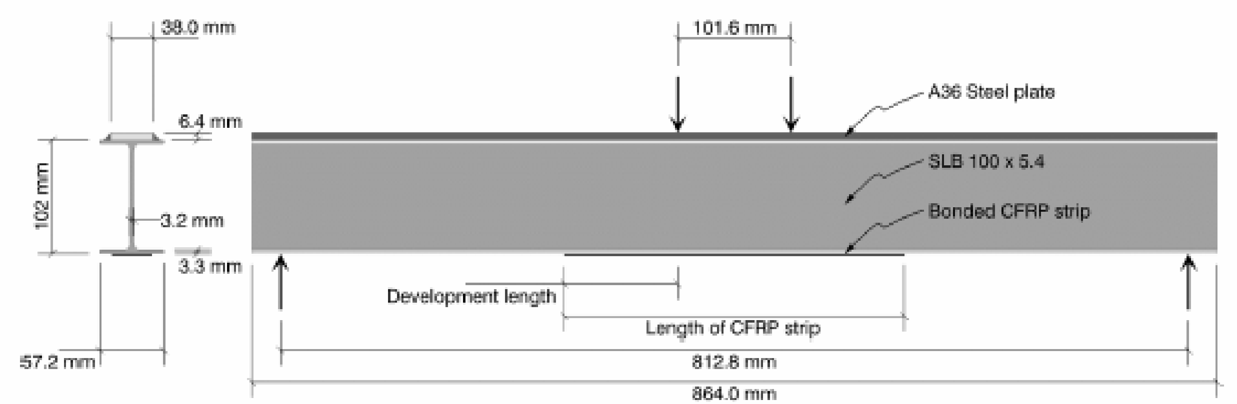


Figure 2: Typical test beam for selection of the structural adhesive (Schnerch, 2005)

4. Phase 2: Large-scale Validation

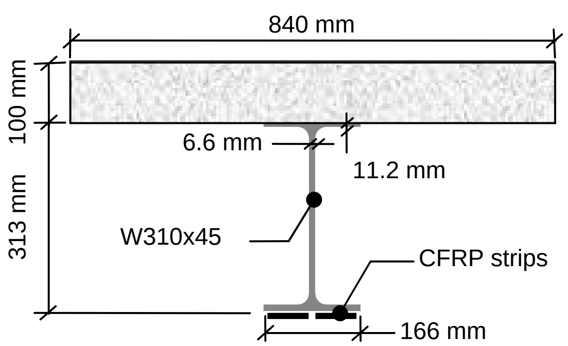
Three large-scale steel-concrete composite beams were tested to investigate the effectiveness of using different configurations of HM CFRP laminates to increase the strength and stiffness of typical steel highway bridge girders (Schnerch, 2005). The details of the testing program are presented in Table 2. Both intermediate and high modulus CFRP materials were considered. The possibility of prestressing the HM CFRP laminates prior to installation on the steel surface was also investigated. All of the beams were loaded monotonically to failure using a four-point bending configuration with a total span of 6400 mm and a 1000 mm constant moment region. A schematic of the cross-section of a typical test beam and the test setup are shown in Figure 3 (a) and (b) respectively.

The load-deflection behavior of the three strengthened beams is presented in Figure 4. The load-deflection behavior of the three beams was essentially linear up to yielding of the steel. After yield, the behavior became increasingly non-linear until rupture of the CFRP occurred. After rupture of the CFRP the behavior of the beams followed a similar trend to that of an unstrengthened beam until failure occurred due to crushing of the concrete. The ultimate capacity of the strengthened beams was governed by rupture of the CFRP while for an unstrengthened beam the ultimate capacity is typically governed by crushing of the concrete.

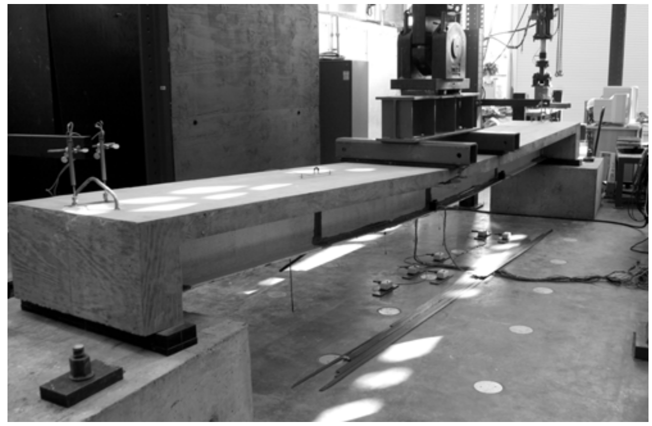
Table 2: Test matrix for the large-scale steel-concrete composite beam tests

Beam Designation	Reinforcement Ratio, ρ^*	CFRP Laminate Properties				Application Method
		Elastic Modulus	Tensile Strength	Thick.	FVF	
IM-4.5-AB	4.5%	229 GPa	1220 MPa	3.2 mm	55%	Bonded
HM-7.6-AB	7.6%	460 GPa	1530 MPa	4.0 mm	70%	Bonded
HM-3.8-PS	3.8%	460 GPa	1530 MPa	2.9 mm	70%	Prestressesd

*Ratio of the CFRP cross-sectional area, accounting for the fiber volume fraction (FVF), to the total cross-sectional area of the steel section



(a) cross-section



(b) test setup

Figure 3: Details of the test-specimen and test setup for the large-scale validation tests

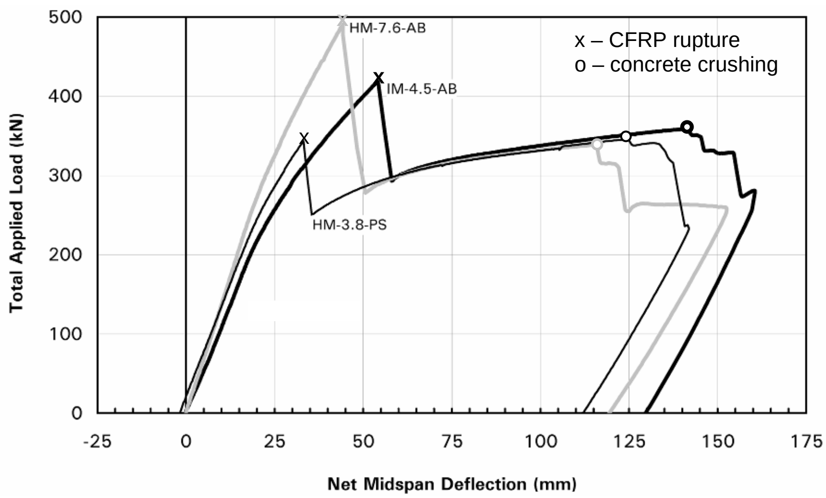


Figure 4: Load deflection behavior of three beams tested in the large-scale validation study

The findings of the large-scale tests demonstrate that the different strengthening systems investigated helped to increase the elastic stiffness and the ultimate capacity of the strengthened beams, as highlighted in Table 3. Both the intermediate-modulus and the high-modulus strengthening systems resulted in a significant increase of the elastic stiffness and the ultimate capacity of the strengthened beams. Alternatively, the prestressed beam was designed primarily to increase the stiffness, without increasing the ultimate strength of the section. This may be advantageous in cases where it is desired to improve the serviceability of a member while maintaining the full ductility of the original section. By comparing the results of beams HM-7.6-AB and HM-3.8-PS in Table 3, it is clear that the use of the prestressed laminates helped to improve the utilization of the strengthening system by reducing the amount of strengthening required to obtain a comparable increase of the elastic stiffness.

Table 3: Stiffness and strength increases

Beam Designation	Stiffness increase	Strength increase
IM-4.5-AB	10%	16%
HM-7.6-AB	36%	45%
HM-3.8-PS	31%	-

5. Phase 3: Overloading Behavior

Once the effectiveness of the HM CFRP strengthening system was established, three additional steel-concrete composite beams were tested, as outlined in Table 4, to investigate the detailed behavior of the strengthened members under overloading conditions (Dawood, 2005).

Table 4: Test matrix for the overloading study

Beam ID	Reinf. Ratio, ρ	Loading
ST-CONT	0%	unload/reload
OVL-1	4.3%	unload/reload
OVL-2	8.6%	unload/reload

All of the beams were tested in a four-point bending configuration with a span of 3050 mm and a 610 mm long constant moment region. The beams were unloaded and reloaded at various load levels to simulate the effect of severe overloading conditions. The typical cross-section consisted of a standard W200x19 steel beam and a 525 mm x 65 mm concrete deck slab. The configuration of the cross-section was similar to that shown in Figure 3 (a). The beams tested in this phase of the experimental program were strengthened with the same type of HM CFRP laminates which were used to strengthen beam HM-7.6-AB in the previous study.

The typical load-deflection behavior of an unstrengthened beam (ST-CONT) and a strengthened beam (OVL-2) are presented in Figure 5 (a) and (b) respectively. From the figures, it can be seen that after yielding, the unstrengthened beam exhibited substantial residual deflection upon unloading while the strengthened beams exhibited minimal residual deflections after yielding up to rupture of the CFRP. In the event of overloading conditions, an unstrengthened beam would likely exhibit a significant residual deflection which may necessitate replacement of the member while a similar strengthened beam would remain in excellent serviceable condition.

In addition to increasing the elastic stiffness and ultimate capacity of the beams as discussed previously, the presence of the CFRP also helped in reducing the level of stresses in the tension flange of the steel beam, thereby increasing the yield load of the strengthened beams as can be seen in Figure 5. Table 5 presents the increase of the stiffness, the yield load and the ultimate capacity for the three beams tested in the overloading study.

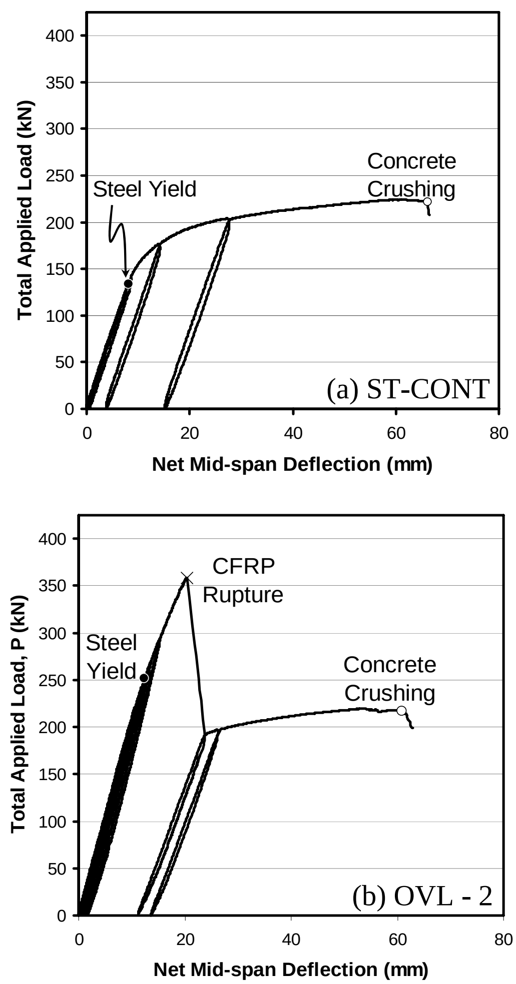


Figure 5: Typical load-deflection behavior of beams tested under overloading conditions

Table 5: Strength and stiffness increase achieved for the overloading beams using HM CFRP

Beam	Percent Increase		
	Stiffness	Yield Load	Capacity
ST-CONT	N/A	N/A	N/A
OVL-1	27%	32%	17%
OVL-2	46%	85%	61%

The values presented in Table 5 indicate that doubling the reinforcement ratio of HM CFRP, from 4.3% for beam OVL-1 to 8.6% for beam OVL-2, more than doubled the increase of the strength and stiffness of the beams. This demonstrates that increasing the amount of HM CFRP materials applied to the beams also increased the efficiency of the HM CFRP materials.

6. Phase 4: Fatigue Behavior

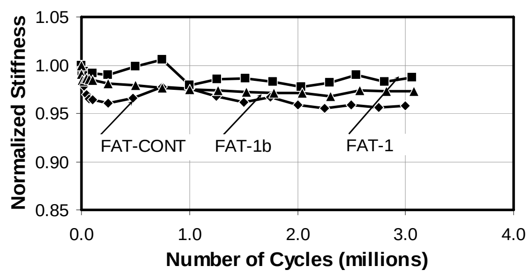
Another three steel-concrete composite beams were tested, as outlined in Table 6, to investigate the behavior of the strengthened members under fatigue loading conditions (Dawood, 2005). The cross-section dimensions and test setup of the beams tested in the fatigue study were identical to those of the beams tested in the overloading study. Two of the beams tested in the fatigue study were strengthened with the same reinforcement ratio of HM CFRP, however, using two different adhesive thicknesses and different bond preparation techniques. Beam FAT-1 was strengthened using an adhesive thickness of 0.1 mm while beam FAT-1b was strengthened using an adhesive thickness of 1.0 mm. A primer coating of silane was also applied to the steel surface of beam FAT-1b prior to application of the adhesive to investigate the effect on the durability of the bond. Beam FAT-CONT remained unstrengthened to serve as a control beam for the fatigue study.

Table 6: Test matrix for the fatigue study

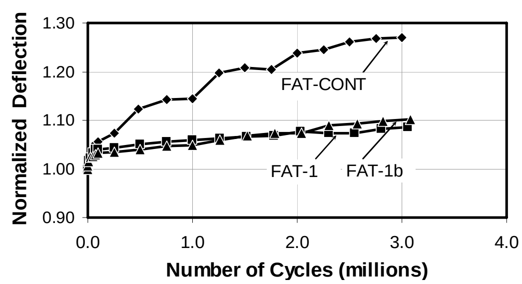
Beam	Reinf. Ratio, ρ	Loading
FAT-CONT	0%	P_{min}=50 kN, ΔP=50 kN
FAT-1	4.3%	P_{min}=50 kN, ΔP=60 kN
FAT-1b	4.3%	P_{min}=50 kN, ΔP=60 kN

All three of the test beams were subjected to three million fatigue loading cycles. The minimum applied load used for the cyclic loading for all three beams, P_{min}, was selected to be equivalent to 30 percent of the calculated yield load of the unstrengthened beams to simulate the effect of the sustained dead-load for a typical bridge structure. For the unstrengthened beam, the maximum load in the loading cycle, P_{max}, was selected to be equivalent to 60 percent of the calculated yield load to simulate the combined effect of dead-load and live-load. The maximum load for the two strengthened beams, P_{max}, was selected to be equivalent to 60 percent of the calculated increased yield load of the strengthened beams. This maximum load also simulated an increase of 20 percent of the allowable live-load level, ΔP, in comparison to the unstrengthened beam.

All three of the tested beams survived the three million-cycle fatigue loading course without exhibiting signs of deterioration before failure. Figure 6 (a) and (b) show the degradation of the stiffness and mean deflection respectively of the three beams throughout the three million-cycle loading course. The values presented in Figure 6 are normalized with respect to the initial values at the beginning of the fatigue loading program.



(a) stiffness



(b) mean deflection

Figure 6: Degradation of the fatigue beams

All three beams exhibited a minimal degradation of the elastic stiffness of less than 5 percent throughout the three million fatigue loading cycles as shown in Figure 6 (a). However, beam FAT-CONT exhibited a nearly 30 percent increase of the mean deflection due to the applied fatigue cycles as shown in Figure 6 (b). This was likely due to the fatigue-creep behavior of the concrete deck slab. Both of the strengthened beams exhibited superior performance with an increase of only 10 percent in the measured mean deflection. The observed degradation of the two strengthened beams throughout the three million fatigue cycles was similar which indicates that the bonding technique did not affect the fatigue behavior of the strengthening system. At the completion of the fatigue program, the three beams were loaded monotonically to failure. The observed behavior of the three beams was similar to that of the beams tested in the overloading study which indicates that the applied fatigue cycles did not have a significant effect on the behavior of the beams.

7. Phase 5: Shear-lag Study

One additional beam was tested under monotonic loading conditions in the fifth phase of the study to investigate the possible presence of a shear-lag effect between the steel surface and the HM CFRP materials (Dawood, 2005). The beam was identical to those tested in the previous two phases of the experimental program and was strengthened with a HM CFRP reinforcement ratio of 8.6 percent. Strains were measured at several depths on the cross-section of the beam at midspan and the measured strain profiles were considered to investigate the shear-lag effect. The four other strengthened beams which were tested in the overloading and fatigue studies were also instrumented at midspan and considered in the shear-lag study of which the two beams that were tested in the fatigue study were also instrumented to measure strains at the quarter-span cross-section.

The strain profiles of all of the beams investigated in the shear-lag study were essentially linear and exhibited a minimal discontinuity of the measured strain between the steel surface and the HM CFRP materials. The observed discontinuities of the strain profiles were likely due to the effect of residual stresses which formed in the steel beams due to the manufacturing process or possibly due to the effect of localized instability and/or possible lateral movement of the tension flange of the steel beams. The presence of these effects was verified using independent strain measurements on the unstrengthened test beams.

8. Phase 6: Bond Behavior of CFRP-to-CFRP

The main objective of the sixth phase of the research program, which is currently in progress, is to examine the bond behavior of HM CFRP-to-CFRP bonded joints. This type of joint simulates the behavior of HM CFRP splice connections which are necessary for implementing the strengthening system to longer span bridge girders. The experimental program includes both CFRP-to-CFRP double-lap shear coupon tests as well as large-scale beam tests to investigate the feasibility of using spliced joints and to establish recommendations for the design of these joints. A preliminary series of double-lap shear coupon tests have been conducted to investigate the effect of various joint configurations on the stress transfer and ultimate capacity of the spliced joints. Previous research has demonstrated that the use of a reverse taper and a spew fillet near the end of a bonded joint can substantially reduce the concentration of shear and normal bond stresses at these locations and possibly increase the ultimate capacity of the bonded joint (Hildebrand, 1994). As such, three different splice configurations were considered in the preliminary study by incorporating a 20^{o} reverse taper and an adhesive fillet at different locations along the splice joint, as shown schematically in Figure 7 (a), (b) and (c) respectively.

The measured load-strain behavior at the surface of the splice plate at the center of the joint is shown for the three completed tests in Figure 8. The initial stiffness of all three joint configurations was the same up to a load level of 40 kN. At the 40 kN load level Joints A and B exhibited a sudden increase of the measured strain which was likely due to cracking of the adhesive due to stress concentrations near the square plate end within the center of the joint. Cracking of the adhesive resulted in a corresponding loss of stiffness of the joint as can be seen in the figure. Joint C did not exhibit a similar increase of strain indicating that cracking did not occur and that the reverse taper was effective in reducing the stress concentration at this location. Joint A failed suddenly due to debonding of the CFRP splice plates at a load level of 90 kN while Joint B failed by debonding of the plates at a load level of 160 kN with additional cracking occurring within the joint at a load level of 144 kN. Debonding of the splice plates of Joint C occurred at a load level of 190 kN. By comparison of the results it is evident that the presence of the reverse taper and the adhesive fillet can increase the capacity of a bonded joint by approximately two times.

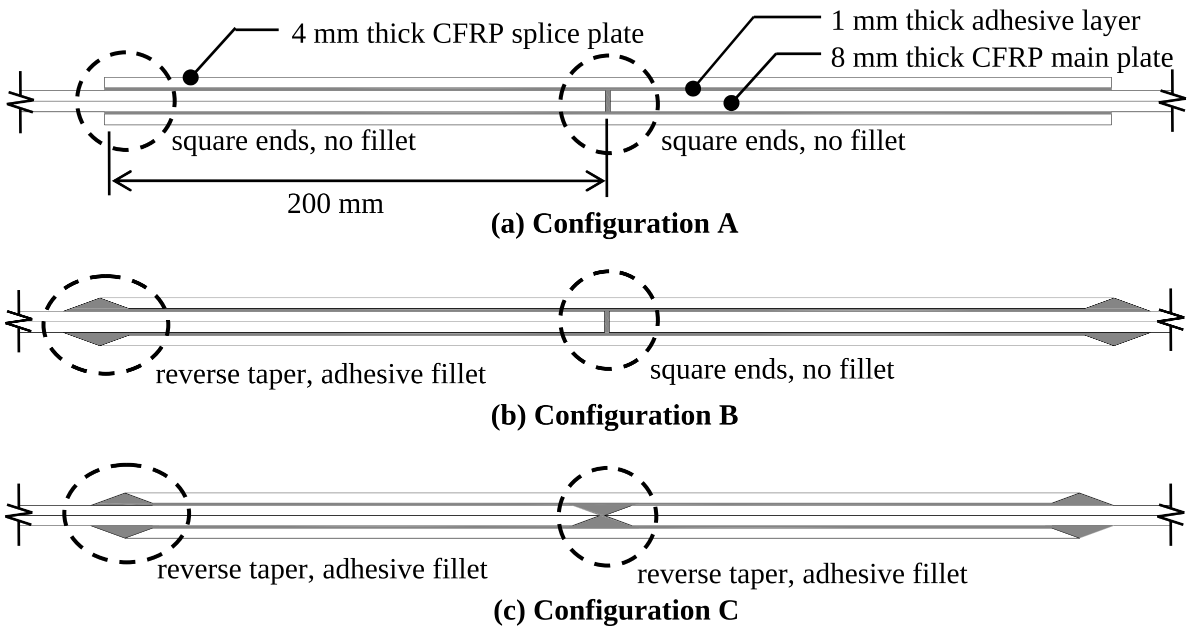


Figure 7: Splice joint configurations for preliminary double-lap shear coupon tests (side view)

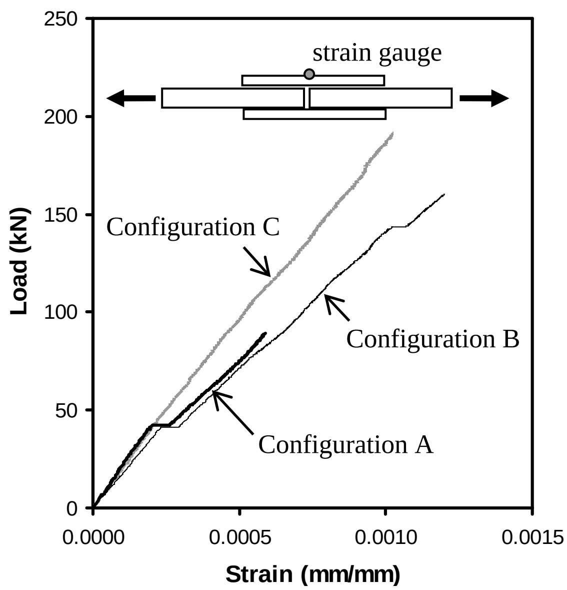


Figure 8: Load-strain behavior at center of splice

9. Proposed Design Guidelines

The design of the HM CFRP strengthening for a steel girder to achieve a given increase of the allowable live load level is based on a moment-curvature analysis which satisfies equilibrium and compatibility and accounts for the non-linear characteristics of the concrete deck slab and the steel girder. A detailed description of the analysis procedure and a worked example of the proposed design guidelines are presented by Schnerch et al. (2006). The increased live load level of the strengthened member should satisfy three conditions, as shown in Figure 9, relative to the moment-curvature relationship of a typical strengthened steel-concrete composite beam. To maintain the fatigue life of the strengthened member, the combined effect of the dead load, M_D, and the increased live load, M_L, should not exceed 60 percent of the increased yield load of the strengthened member, $M_{y,S}$. Additionally, to satisfy the strength limit state, the total factored load, after applying the appropriate dead and live load factors, α_D and α_L respectively, should not exceed the ultimate capacity of the strengthened member, $M_{U,S}$, after applying an appropriate strength reduction factor, ϕ. A strength reduction factor of 0.75 is proposed which is consistent with the American Institute of Steel Construction (AISC) requirements for rupture type limit states (2001). To ensure the safety of the structure in the event of a sudden loss of the strengthening system, the total effect of the applied dead load, M_D, and the increased live load, M_L, should not exceed the residual nominal capacity of the unstrengthened beam, $M_{n,US}$.

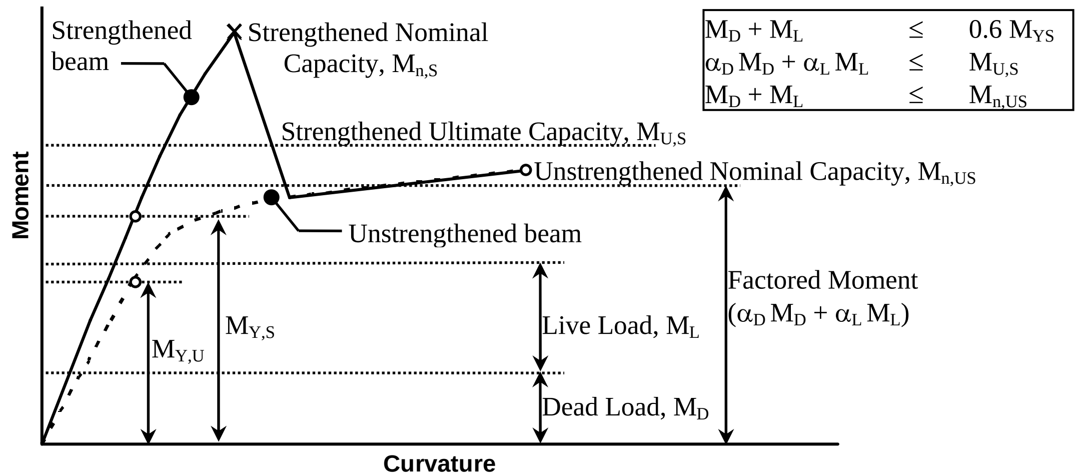


Figure 9: Proposed design criteria for strengthening of steel beams with HM CFRP materials

10. Conclusions

This paper presents the findings of an extensive experimental program to develop a HM CFRP system for flexural strengthening of steel flexural members. The effectiveness of the strengthening system was demonstrated and the detailed behavior of the strengthened beams was examined. Based on the research findings design guidelines are proposed to facilitate proper design of the HM CFRP strengthening for steel bridges and structures. The research findings demonstrate that externally bonded HM CFRP is an effective method for strengthening and repair of steel structures.

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Mina Dawood received his MS degree in Civil Engineering from NC State University and completed his thesis on the repair and strengthening of steel structures with HM CFRP materials. He is currently continuing his research in the same field in pursuit of his Doctoral degree.

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