Development of a carbon fiber reinforced polymer system for strengthening steel structures

Sami Rizkalla a, Mina Dawood a,*, David Schnerch b

a Constructed Facilities Laboratory, North Carolina State University, 2414 Campus Shore Dr., Campus Box 7533, Raleigh, NC 27695-7533, United States
b Wiss, Janney, Elstner Associates Inc., 245 First Street, Suite 1200, Cambridge, MA 02142, United States

Received 9 May 2007; received in revised form 22 September 2007; accepted 6 October 2007

Abstract

This paper summarizes the development and use of high modulus carbon fiber reinforced polymer (HM CFRP) materials for the retrofit of steel structures and bridges. The development 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 using different strengthening schemes. The experimental program investigated the behavior of the strengthening system under fatigue and overloading conditions. A detailed study of bond behavior, including the possible presence of shear-lag effects and performance of spliced joints is also presented. Based on the findings, flexural design guidelines are proposed. The study indicates that CFRP materials can be effectively used to enhance the serviceability and ultimate strength of steel flexural members.

Keywords: B. Fatigue; C. Analytical modeling; D. Mechanical testing; E. Joints/joining

1. Introduction

Fiber reinforced polymer (FRP) materials are now commonly used for repair and strengthening of concrete structures. Due to the success of this technique, researchers have recently investigated the use of externally bonded carbon FRP (CFRP) materials for strengthening steel flexural members. 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 [1], repair of naturally deteriorated girders [2], strengthening of undamaged girders [3] and repair of girders with simulated corrosion damage [4]. 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 [5].

Recent research efforts have focused on understanding the bond behavior of strengthening plates bonded to metallic beams [6-8]. The existing models typically predict high concentrations of shear and normal (peeling) stresses near the plate ends. These stress concentrations can cause premature debonding of the strengthening plate and thus the plate should be carefully detailed in the design. Finite element studies have demonstrated that the bond stress concentrations can be minimized and the strength of the bonded joint increased by using special configurations of the ends of the strengthening plate [9-11].

This paper presents the findings of a research program, which was conducted over five 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 included the selection of an appropriate saturating resin to be used for wet lay-up of the dry fibers as well as the selection of a suitable adhesive for externally bonded, pre-cured HM CFRP strips. 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 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, respectively. The possible presence of a shear-lag effect between the steel surface and the HM CFRP materials was studied in phase five. The sixth phase of the research focused primarily on understanding the bond behavior of the strengthening system in detail. The study investigated the bond and splice behavior of the HM CFRP plates, the plate end details and joint lengths required to effectively splice the CFRP strips to facilitate implementation of the system for strengthening of longer span structures. Based on the findings of this extensive 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 in situ using a wet lay-up technique that is well suited for curved applications or highly irregular surfaces such as steel monopole towers or curved girders. For applications requiring a higher degree of strengthening, the HM carbon fibers can also be pultruded into a pre-cured strip 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 strip HM CFRP materials are given in Table 1.

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 strips [13].

The relative performance of 10 different saturating resins was compared through a series of double-lap shear coupon tests. The typical coupons consisted of two steel plates bonded together by a single ply of wet lay-up CFRP sheets on each face as shown schematically in Fig. 1. The resins were allowed to cure at room temperature for at least 7 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 due to possible incomplete wetting of the fibers by certain resins. 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 perfor-

![Fig. 1. Typical double-lap shear coupon for selection of the saturating resin.](image-url)
mance 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.

Small scale flexural tests were used to select the most appropriate adhesive for bonding HM CFRP strips to steel surfaces. These tests identified the bond stress distribution 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 × 1.4 mm thick CFRP strips of varying lengths to the bottom of the tension flange. The typical test specimen used in this phase is shown schematically in Fig. 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 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 Fig. 2. Additional tests indicated that two plies of the HM CFRP strips 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.

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 strips to increase the strength and stiffness of typical steel bridge girders [13]. The details of the testing program are presented in Table 2. Both intermediate (IM) and high modulus CFRP materials were considered. While the modulus of the HM materials is approximately 45% higher than that of the IM materials, the ultimate strain capacity of the IM CFRP is approximately 50% higher than that of the HM materials. The appropriate material should be selected for the particular strengthening application being considered. The possibility of prestressing the HM CFRP strips prior to installation on the steel surface was also investigated. Prior to installation of the CFRP materials, the unstrengthened beams were loaded to 60% of the yield load to determine their initial stiffness. After strengthening all of the beams were loaded monotonically to failure using a four-point bending configuration with a total span of 6400 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 Fig. 3a and b, respectively.

The load-deflection behavior of the three strengthened beams is presented in Fig. 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

![Fig. 2. Typical test beam for selection of the structural adhesive.](image_url)

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>CFRP ratio, $\rho^a$ (%)</th>
<th>CFRP strip properties</th>
<th>Application method</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Elastic modulus (GPa)</td>
<td>Tensile strength (MPa)</td>
<td>Thickness (mm)</td>
</tr>
<tr>
<td>IM-4.5-AB</td>
<td>4.5</td>
<td>229</td>
<td>1220</td>
</tr>
<tr>
<td>HM-7.6-AB</td>
<td>7.6</td>
<td>460</td>
<td>1530</td>
</tr>
<tr>
<td>HM-3.8-PS</td>
<td>3.8</td>
<td>460</td>
<td>1530</td>
</tr>
</tbody>
</table>

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

The findings of the large-scale tests demonstrate that the different strengthening systems enhanced the elastic stiffness and the ultimate capacity of the strengthened beams, as highlighted in Table 3. The reported stiffness increase was determined as compared to the measured initial stiffness of the unstrengthened beams. The strength increase was determined by comparing the load at rupture of the CFRP to the load at crushing of the concrete. The latter represents the typical failure mode of the unstrengthened beams. 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 the 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 strips helped to improve the efficiency of the strengthening system by reducing the amount of strengthening required to obtain a comparable increase of the elastic stiffness.

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 [14]. All of the beams were tested in a four-point bending configuration with a span of 3050 and a 610 mm long constant moment region. The beams were unloaded and reloaded at various load lev-

Table 3
Stiffness and strength increases for large-scale tests

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>Stiffness increase (%)</th>
<th>Strength increase (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>IM-4.5-AB</td>
<td>10</td>
<td>16</td>
</tr>
<tr>
<td>HM-7.6-AB</td>
<td>36</td>
<td>45</td>
</tr>
<tr>
<td>HM-3.8-PS</td>
<td>31</td>
<td></td>
</tr>
</tbody>
</table>

Table 4
Test matrix for the overloading study

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>CFRP ratio, ( \rho ) (%)</th>
<th>Loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>ST-CONT</td>
<td>0</td>
<td>Unload/reload</td>
</tr>
<tr>
<td>OVL-1</td>
<td>4.3</td>
<td>Unload/reload</td>
</tr>
<tr>
<td>OVL-2</td>
<td>8.6</td>
<td>Unload/reload</td>
</tr>
</tbody>
</table>

Fig. 3. Details of the test-specimen and test setup for the large-scale validation tests.

Fig. 4. Load–deflection behavior of three beams tested in the large-scale validation study.
els to simulate the effect of severe overloading conditions. The typical cross-section consisted of a standard W200x19 steel beam and a 525 mm × 65 mm concrete deck slab. The configuration of the cross-section was similar to that shown in Fig. 3a. The beams tested in this phase of the experimental program were strengthened with the same type of HM CFRP strips which were used to strengthen beam HM-7.6-AB in the previous phase.

The typical load–deflection behavior of an unstrengthened beam (ST-CONT) and a strengthened beam (OVL-2) are presented in Fig. 5a 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 capacity of the strengthened beams as can be seen in Fig. 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.

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

### 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 [14]. 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 reported in phase 3. Two of the beams tested in the fatigue study were strengthened with the same reinforcement ratio of CFRP, however, using two different adhesive thicknesses and different bond preparation techniques. For beam FAT-1, no measures were taken to control the adhesive thickness. The average measured adhesive thickness for this beam was approximately 0.1 mm. For beam FAT-1b, however, small glass spacer beads were mixed with the epoxy to achieve an average measured adhesive thickness of about 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 of this adhesion promoter which has been demonstrated in the literature to enhance the environmental durability of the steel-to-epoxy interface. Beam FAT-CONT remained unstrengthened to serve as a control beam for the fatigue study.

All three of the test beams were subjected to 3 million fatigue loading cycles at a frequency of 3 Hz. The minimum applied load used for the cyclic loading for all three beams, shown in Fig. 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.

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>Percent increase</th>
</tr>
</thead>
<tbody>
<tr>
<td>ST-CONT</td>
<td>N/A</td>
</tr>
<tr>
<td>OVL-1</td>
<td>27</td>
</tr>
<tr>
<td>OVL-2</td>
<td>46</td>
</tr>
</tbody>
</table>

Table 6

<table>
<thead>
<tr>
<th>Test matrix for the fatigue study</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam ID</td>
</tr>
<tr>
<td>---------</td>
</tr>
<tr>
<td>FAT-CONT</td>
</tr>
<tr>
<td>FAT-1</td>
</tr>
<tr>
<td>FAT-1b</td>
</tr>
</tbody>
</table>

Fig. 5. Typical load–deflection behavior of beams tested under overloading conditions.
was selected to be equivalent to 30% 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, \( P_{\text{max}} \), was selected to be equivalent to 60% of the calculated yield load to simulate the combined effect of dead-load and live-load. The maximum load for the two strengthened beams, \( P_{\text{max}} \), was selected to be equivalent to 60% of the calculated increased yield load of the strengthened beams. This maximum load also simulated an increase of 20% of the allowable live-load level, \( \Delta P \), in comparison to the unstrengthened beam.

All three of the tested beams survived the 3 million-cycle fatigue loading course without exhibiting signs of deterioration or failure. Fig. 6a and b shows the degradation of the stiffness and mean deflection, respectively, of the three beams throughout the 3 million-cycle loading course. The values presented in Fig. 6 are normalized with respect to the initial values at the beginning of the fatigue loading program.

All three beams exhibited a minimal degradation of the elastic stiffness of less than 5% throughout the 3 million fatigue loading cycles as shown in Fig. 6a. However, the control beam, FAT-CONT, exhibited a nearly 30% increase of the mean deflection due to the applied fatigue cycles as shown in Fig. 6b. This was likely due to the fatigue-creep behavior of the concrete deck slab and possibly also due to an increase of the residual stresses in the tension flange of the steel beam which may have caused drift of the beam. Since the presence of the CFRP strengthening helped to reduce the stresses in the steel tension flange, both of the strengthened beams exhibited superior performance with an increase of only 10% in the measured mean deflection. The observed degradation of the two strengthened beams throughout the 3 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 [14]. The beam was identical to those tested in the previous two phases of the experimental program and was strengthened with a CFRP reinforcement ratio of 8.6%. 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 [14]. The observed small 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 and splice behavior

The main objective of the sixth phase of the research program was to examine the bond behavior of CFRP bonded lap splice joints [15]. This type of joint simulates splice connections between adjacent CFRP strips which are necessary for implementing the strengthening system to longer span beams. The experimental program included 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 installation of these joints. A total of six double-lap shear coupons and ten large-scale beams were tested.

The test matrix for the double-lap shear coupon tests and the beam tests are presented in Tables 7 and 8, respectively. The primary parameters which were studied include the overall length of the CFRP splice plate, the detailing of the plate end and the application of additional mechanical anchorage near the plate ends. Four different plate end details were investigated as shown schematically in Fig. 7.

![Fig. 6. Degradation of the fatigue beams: (a) stiffness and (b) mean deflection.](image-url)
The first configuration includes square plate end details which represent the simplest and most commonly used joint configuration. To reduce the stress concentrations in the adhesive two different combinations of reverse tapered plate end details were considered. The ‘reverse tapered 1 (T1)’ configuration includes a reverse tapered joint detail only at the end of the splice plate, whereas the ‘reverse tapered 2 (T2)’ configuration also includes reverse tapers within the center of the splice joint. The ‘rounded & tapered (U)’ configuration is similar to the T2 configuration; however, the plate ends are also rounded in plan. Typical square, tapered and rounded plate ends are shown in Fig. 8a–c, respectively. To resist normal (peeling) stresses near the ends of the CFRP plates, different types of mechanical anchorage were implemented for several of the tested coupons and beams. A specially fabricated steel clamp, shown in Fig. 9a, was installed at each end of the splice plate on double-lap shear coupon 400-U-C. A similar clamping system, shown in Fig. 9b, was installed on steel beam 800-U-C. Several of the tested beams also included a transverse HM CFRP wrap, shown in Fig. 9c, to help resist peeling stresses. The average measured adhesive thickness was approximately 1.0 mm for all of the tested splice beams and coupons.

All of the tested coupons failed by sudden debonding of the CFRP splice plates. The average measured load–strain response at the center of the CFRP splice joint, which was measured by a pair of 6 mm long electrical resistance strain gauges located on either side of the coupon, is shown in Fig. 10 for several of the tested coupons. The calculated response of a continuous CFRP plate up to rupture is also shown by the dashed line in the figure. Inspection of the figure indicates that implementation of the reverse tapered plate end detail can significantly increase the ultimate capacity of the bonded splice joints. Comparing the results for coupons 400-T2 and 400-S indicates that the presence of the reverse tapered plate end detail approximately doubled the joint capacity. This was due to a reduction of the bond stress concentration near the ends of the CFRP plates which was confirmed by strain measurements at various locations along the length of the splice joints.

Table 7 presents the maximum measured load and strain for each of the tested coupons. The strains were measured on the main CFRP plate approximately 25 mm from the toe of the splice plate. The table also presents the ratio of the maximum measured strain to the rupture strain of the CFRP material for each of the coupons, \( e_{frp,u} \). Comparison of the results for coupons 400-T2 and 400-S indicates that the presence of the reverse tapered plate end detail approximately doubled the joint capacity. This was due to a reduction of the bond stress concentration near the ends of the CFRP plates which was confirmed by strain measurements at various locations along the length of the splice joints.

Table 9 presents the maximum measured load and strain for each of the tested beams. The maximum measured load–strain response at the center of the CFRP beam splice joint was measured by a pair of 6 mm long electrical resistance strain gauges located on either side of the beam. The table also presents the ratio of the maximum measured strain to the rupture strain of the CFRP material for each of the beams, \( e_{frp,u} \). Comparison of the results for beams 800-U and 800-U-C indicates that the presence of the steel clamps near the splice plate ends helped to increase the joint capacity by approximately 80%. This suggests that the steel clamps provided adequate stiffness to resist the peeling stresses and strains which were induced at these locations. Further examination of the table indicates that the maximum measured strain achieved for all of the tested beams corresponds to only 60% of the maximum rupture strain of the CFRP material. This suggests that the appropriate location for a CFRP splice joint for a typical strengthening application should be selected based on the calculated longitudinal strain in the CFRP obtained from a suitable analysis of the member.

Fig. 11 presents the measured load–deflection behavior for each of the tested splice beams. The dashed line in the figure represents the calculated load–deflection behavior of the strengthened member beyond rupture of the CFRP throughout the full range of loading. All of the tested beams failed due to debonding of the splice plate prior to rupture of the CFRP. The maximum measured
Comparison of the results indicates that, for the square end plate details, increasing the splice length from 400 mm to 800 mm did not significantly affect the capacity of the joint. However, the plate end detail had a significant effect on the joint capacity. Implementation of the reverse tapered joint configuration for the 800 mm long splice joint, 800-T2, approximately doubled the joint capacity as compared to the square plate-ended splice joints, 800-S. This is consistent with the double-lap shear coupon test results. Similarly to the coupon test results, the maximum measured strains from the beam tests can be used to identify a suitable location for a CFRP splice joint based on the results of an appropriate analysis of the structure.

Table 10 also presents the ratio of the maximum measured load to the calculated yield load of the unstrengthened beam, $P_{Y,U}$. Inspection of the table indicates that...
this ratio is greater than 1.0 for several of the splice joint configurations. The beams strengthened with these joint configurations were capable of sustaining loads higher than the yield load of the unstrengthened beam. For example, beam specimen 800-T2 was capable of sustaining a load which was 26% greater than the yield load of the unstrengthened steel beam. Thus, even under severe loading conditions, the spliced CFRP plate significantly enhanced the serviceability of the steel beam.

The test results indicate that the presence of the transverse CFRP wrap did not increase the capacity of the spliced joints. Comparison of the results for beams 400-T2 and 400-T2-W indicate a considerable reduction of the joint capacity for the wrapped splice joint. The observed reduction of the joint capacity may have been due to a possible damaged region in the adhesive which may have formed during fabrication. Alternatively, due to the low tensile strength of the transverse fiber wrap in the longitudinal direction of the beam, a crack may have formed in the transverse fiber wrap which could have possibly induced the observed premature debonding failure of the splice joint.

Comparison of the results for beams 800-U and 800-U-C indicate that the presence of the steel clamps slightly reduced the capacity of the splice joint. Inspection of the joint after failure indicated that this was likely due to the installation sequence of the clamping system. To prevent direct electrical contact between the steel clamp and the CFRP, a thin layer of adhesive was placed between the two components. This was done to help prevent galvanic corrosion under typical outdoor exposure conditions. The adhesive was allowed to cure prior to installation of the high-strength bolts which were used to tighten the clamping system. Tightening of the steel bolts resulted in a slight, unanticipated curvature of the steel plate which induced peeling stresses near the plate end rather than clamping stresses. Consequently, the presence of the clamping system was detrimental to the joint performance rather than beneficial and thus reduced the ultimate capacity of the joint.

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. [16]. The increased live-load level of the strengthened member should satisfy three conditions, as shown in Fig. 12, relative to the moment–curvature relationship of a typical strengthened steel–concrete composite beam. To ensure that the strengthened member remains elastic under the effect of the design service load, the combined effect of the dead-load, \( M_{D} \), and the increased live-load, \( M_{L} \), should not exceed 60% 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, \( z_{D} \) and \( z_{L} \), respectively, should not exceed the ultimate capacity of the strengthened member, \( M_{U,S} \), after applying an appropriate strength reduction factor, \( \phi \). 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 [17]. 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, \( M_{n,US} \).
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. Large-scale testing indicated that the use of HM CFRP materials can significantly increase the elastic stiffness, yield load and ultimate capacity of typical steel–concrete composite beams. The research findings demonstrate that beams strengthened using the proposed system exhibit superior performance under overloading and fatigue loading conditions as compared to an unstrengthened beam. The research further demonstrates that CFRP strips can be effectively spliced using bonded CFRP cover plates. The use of the reverse tapered plate end detail at all plate termination points is recommended to reduce bond stress concentrations at these locations and to increase the ultimate capacity of bonded joints. 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 the use of externally bonded HM CFRP is an effective method to retrofit steel structures.

Acknowledgements

The authors would like to acknowledge the support provided by the National Science Foundation (NSF) Industry/University Cooperative Research Center (I/UCRC) for Repair of Buildings and Bridges with Composites (RB2C) and the support of Mitsubishi Chemical FP America Inc.

References