Effective Splices for a Carbon Fiber-Reinforced Polymer
Strengthening System for Steel Bridges and Structures

Mina Dawood, Murthy Guddati, and Sami Rizkalla

Carbon fiber-reinforced polymer (CFRP) materials have been successfully used to strengthen reinforced concrete bridges and structures. Recently, a new high modulus CFRP strengthening system was developed to increase the allowable load carrying capacity and to enhance the serviceability of steel bridges and structures. Because of the relatively high flexural rigidity of the CFRP materials, the length of the CFRP plates that can be transported to the job site is limited. To implement the proposed strengthening system in longer-span steel bridges, adjacent lengths of CFRP must be spliced. To develop an effective splice joint for the proposed strengthening system, an experimental and analytical research program was conducted to study the bond behavior of the CFRP materials. The parameters considered included plate end geometry, splice length, and the possibility of using mechanical anchorage. The analytical study included a finite element analysis to determine the distribution of the stresses within the adhesive layer for different splice configurations. On the basis of the findings, a simplified method was proposed to design lap splice joints with different reversed taper angles and adhesive properties. The research concluded that, with proper detailing, the proposed CFRP system could be effectively used to strengthen steel bridges and structures.

Fiber-reinforced polymer (FRP) materials have been successfully used in civil engineering infrastructure applications to strengthen reinforced concrete structures. On the basis of this success, the use of carbon FRP (CFRP) materials for strengthening steel bridges and structures has also been studied (1). Although conventional CFRP materials can be used to increase the ultimate strength of steel girders, because of their relatively low modulus compared with steel, large amounts of strengthening materials are required to enhance the serviceability of the members. Recently, a high-modulus CFRP system was developed that can be effectively used to increase the load-carrying capacity and to enhance the serviceability of steel bridges and structures (2, 3). On the basis of the success of the proposed strengthening system, guidelines have been published to facilitate the design of the proposed strengthening system to achieve a specific increase of the allowable live load of a typical steel-concrete composite beam (4). Because of the stiffness of the high modulus CFRP materials, there are limitations on the length of the plates that can be delivered to the site. To implement the proposed strengthening system in longer span-structures, a suitable CFRP splice joint should be developed. Because of the short length and high flexural rigidity of the CFRP splice plates, these elements are particularly susceptible to debonding type failure modes.

Research has demonstrated that significant stress concentrations are induced within the adhesive layer near the ends of externally bonded FRP plates, which can lead to premature debonding of the FRP materials. Although a number of expressions exist to predict the magnitude of the stress concentration for externally bonded FRP plates, these expressions are overly complex to be easily implemented for design purposes (2, 5). Other research has shown that the magnitude of the stress concentrations can be significantly reduced by modifying the geometry of the end of the strengthening plate, thereby increasing the debonding strength of the FRP system (6, 7). This study presents the findings of an experimental and analytical research program that was conducted to study the behavior of bonded CFRP lap splice joints that can be used to strengthen civil engineering infrastructure, including long-span steel bridges and flexural members. Different parameters were studied to enhance the strength of the lap spliced joints, including the plate end geometry, splice length, and possible use of mechanical anchorage. On the basis of the research findings, a simplified method is proposed to design splice joints with different reversed taper angles and adhesive properties. Details of the experimental results and the analytical phase are presented elsewhere (8).

HIGH MODULUS CFRP STRENGTHENING SYSTEM

The proposed CFRP strengthening system consists of pultruded high modulus CFRP plates that are bonded to the steel surface by an epoxy adhesive. The material properties of the CFRP and the adhesive were determined experimentally. The average measured tensile modulus and ultimate strength of the CFRP plates were 418,000 MPa and 1,490 MPa, respectively. The average measured tensile modulus and ultimate strength of the adhesive were 2,980 MPa and 38 MPa, respectively. Both materials exhibited essentially linear-elastic behavior up to failure (8). Although glass spacer beads were used to maintain the adhesive thickness for the test specimens, the tensile properties of the adhesive were obtained from pure adhesive samples without the addition of glass spacer beads.

EXPERIMENTAL PROGRAM

The experimental program was conducted in two phases. The first phase consisted of eight CFRP double-lap shear specimen tests to study the bond behavior of spliced CFRP plates. The second phase
included 10 steel beams strengthened with CFRP plates with different configurations of CFRP splices installed at midspan to evaluate the behavior of the splice connections under flexural loading conditions.

Double-Lap Shear Tests

The double-lap shear specimens consisted of two 8-mm-thick x 38-mm-wide main CFRP plates that were bonded together using two 4-mm-thick x 38-mm-wide CFRP splice plates. Glass spacer beads were mixed into the adhesive with a ratio of 1 gram of beads per 100 grams of adhesive to maintain the adhesive thickness at approximately 1 mm.

The testing program for the double-lap shear specimens is given in Table 1. Each specimen configuration was assigned a three-part identifier. The first part of the identifier indicates the overall length of the splice plate, in millimeters. The second part indicates the plate end detail. The third part, denoted by a C, indicates the presence of the steel clamp. The number in parentheses after the identifier indicates multiple repetitions of the same specimen configuration. The measured failure loads for all of the tested specimens are also presented in Table 1.

The four different plate end details investigated in this program are shown schematically in Figure 1. Different geometrical configurations of the plate end were considered to reduce the bond stress concentrations as outlined in Figure 1a. Taper angles and fillet angles of approximately 20° were considered in the experimental study. Typical square, tapered, and rounded plate ends are shown in Figure 1b, 1c, and 1d, respectively. The use of a steel clamp at the ends of the splice plate was also considered, as shown in Figure 1e. The specimens were instrumented with 6-mm-long electrical resistance strain gauges that were bonded at different locations along the surface of the CFRP plates.

The specimens were loaded monotonically to failure at a displacement rate of 0.5 mm/min. All of the tested double-lap shear specimens failed suddenly because of debonding of one or both of the CFRP splice plates, as shown in Figure 2a. Typically, a mixed mode failure was observed with some of the adhesive remaining bonded to the main CFRP plate and some of the adhesive bonded to the CFRP splice plate. Inspection of the failure surfaces suggested that the failures typically occurred within a thin layer of resin at the surface of the CFRP. However, several cracks and diagonal failure planes were observed in the adhesive layer, which suggests that the failure may also have been partially cohesive within the adhesive layer.

The measured load–strain behavior of the tested coupons is presented in Figure 2b. The predicted load–strain behavior of a continuous CFRP coupon without a splice joint is shown by the dashed line in the figure. The predicted behavior is based on the measured

### Table 1: Double-Lap Shear Specimen Test Matrix

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Splice Length (mm)</th>
<th>Plate End Detail</th>
<th>Mechanical Anchorage</th>
<th>Mechanical Failure Load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>400-S(1)</td>
<td>400</td>
<td>S</td>
<td>None</td>
<td>89</td>
</tr>
<tr>
<td>400-S(2)</td>
<td>S</td>
<td>None</td>
<td>97</td>
<td></td>
</tr>
<tr>
<td>400-S(3)</td>
<td>S</td>
<td>None</td>
<td>97</td>
<td></td>
</tr>
<tr>
<td>400-T1</td>
<td>T1</td>
<td>None</td>
<td>160</td>
<td></td>
</tr>
<tr>
<td>400-T2(1)</td>
<td>T2</td>
<td>None</td>
<td>191</td>
<td></td>
</tr>
<tr>
<td>400-T2(2)</td>
<td>T2</td>
<td>None</td>
<td>228</td>
<td></td>
</tr>
<tr>
<td>400-U</td>
<td>U</td>
<td>None</td>
<td>157</td>
<td></td>
</tr>
<tr>
<td>400-U-C</td>
<td>U</td>
<td>Steel clamp</td>
<td>284</td>
<td></td>
</tr>
</tbody>
</table>

*See Figure 1.

FIGURE 1  Tested splice configurations: (a) schematic and typical (b) square, (c) tapered, (d) rounded and tapered, and (e) clamped plate ends.
stress-strain relationship obtained from tests of continuous CFRP coupons with smaller cross-sectional dimensions that were conducted by the authors. Inspection of the figure indicates that, initially, the measured strains at the surface of the CFRP splice plate were lower than the predicted strains for the continuous CFRP. This behavior is caused by the presence of the adhesive between the ends of the main CFRP plates, which likely helped to transfer shear stresses across the joint, thereby decreasing the measured strain at the surface of the splice plate. During testing of the three 400-S square plate end specimens, a cracking sound was heard, which was accompanied by a sudden increase of the measured strain, shown in Figure 2b, at the 42 kN, 46 kN, and 50 kN load levels for the three specimens, respectively. This adhesive cracking occurred within the center of the joint, as was confirmed later using a nonlinear finite element analysis (FEA). The FEA further indicated that although cracking of the adhesive initiated at the center of the joint, the failure was ultimately caused by the propagation of a crack, which initiated near the end of the splice plate at a higher load level.

Inspection of Figure 2b indicates that similar cracking occurred for the 400-T1 configuration at the 41 kN and 144 kN load levels. The cracking of the adhesive was likely caused by a stress concentration within the center of the joint because of the square end configuration of the ends of the main CFRP plates within the center of the joint for the 400-S and 400-T1 configurations. Both the 400-T2 and 400-U specimen configurations included modified plate end geometries within the center of the joint, which helped to reduce the stress concentration at this location. Therefore, no cracks were observed for these specimens. Inspection of Figure 2b and Table 1 further indicates that the presence of the reverse taper at the end of the CFRP splice plate, such as for joint configurations 400-T1 and 400-T2, approximately doubled the measured tension strength of the joints as compared with the square configuration, 400-S.

The measured failure load of specimen 400-U-C with steel clamps installed at the ends of the splice plates, given in Table 1, was 284 kN. This is approximately 80% higher than the measured failure load of the similar unclamped specimen, 400-U, also given in the table. This suggests that using clamps could help to confine the end region of the splice and resist the normal (peeling) stress component, which typically forms near the plate end.

**Beam Specimens**

The typical test beams, shown in Figure 3, consisted of a W12 x 30 wide flange 350 MPa steel beam. A steel channel was welded to the compression flange to simulate the presence of a reinforced concrete deck. The beams were strengthened using two 2.210-mm-long x 100-mm-wide x 4-mm-thick main CFRP plates, which were butted together at midspan to simulate a discontinuity of the main CFRP plate. A CFRP splice plate was subsequently bonded to the main CFRP plates. Before installation of the CFRP, the steel was wiped with acetone, grit-blasted to a white metal finish, and rewiped with acetone. A Peel Ply layer was removed from the surface of the CFRP, and the adhesive was thoroughly mixed and applied to the surface of the CFRP. Glass beads were mixed into the adhesive with a weight fraction of 1 gram of beads per 100 grams of adhesive to maintain the thickness of the adhesive layer at approximately 1 mm.

The steel beam test program is given in Table 2. Three parameters were considered in the study: the length of the splice plate, the geometry of the plate end, and the use of two types of mechanical anchorage. The plate end details that were considered were similar to those tested in the double-lap shear tests. The two types of mechanical anchorage that were considered include a steel clamping system and a transverse CFRP wrap. The transverse CFRP wrap consisted of a single ply of unidirectional high modulus carbon fiber fabric that was similar to the fibers used in the primary strengthening system. The fiber sheets were bonded using a commercially available two-part epoxy. Each splice configuration was assigned a three-part
identifier using a similar convention to that used for the double-lap shear specimens. The specimens were instrumented with 6-mm-long electrical resistance strain gauges that were bonded at different locations along the surface of the CFRP plates.

The beams were loaded monotonically to failure at a displacement rate of 0.5 mm/min. All of the tested steel beams failed because of debonding of the splice plates before rupture of the CFRP in a similar manner to that observed for the double-lap shear specimens. None of the tested beams exhibited any observable cracking before failure. The measured failure loads of the tested beams are presented in Table 2. The table also presents the ratio between the measured failure load, $P_{\text{max}}$, and the calculated yield load of the unstrengthened beam, $P_{\text{y}}$, and the increased yield load of the strengthened beam, $P_{\text{y,s}}$.

Comparison of the results for beams 800-S(1), 800-S(2), and 400-S indicates that increasing the splice length did not increase the measured failure load of the beams. However, similarly to the double-lap shear specimen results, comparison of the results for beams 800-S(1), 800-S(2), and 800-T2 indicates that the presence of the reverse taper near the plate ends approximately doubled the joint strength. A similar trend can be observed by comparing the results of beams 400-S and 400-T2. Test results also indicate that the failure loads of the beams strengthened with square plate end splice joints were less than the yield load of the unstrengthened beam. By comparison, the failure loads of the beams strengthened with reverse tapered splice plates exceeded the yield load of the unstrengthened beam and approached the calculated increased yield load of the strengthened beams. This finding suggests that, even under severe loading conditions, the spliced CFRP strengthening system with tapered plate ends can be used to enhance the serviceability of the structure.

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>Splice Configuration</th>
<th>Failure Load (kN)</th>
<th>$P_{\text{max}}/P_{\text{y}}$</th>
<th>$P_{\text{max}}/P_{\text{y,s}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>800-S(1)</td>
<td>800 mm long, square plate ends</td>
<td>178</td>
<td>0.55</td>
<td>0.41</td>
</tr>
<tr>
<td>800-S(2)</td>
<td>800 mm, square plate ends</td>
<td>205</td>
<td>0.63</td>
<td>0.47</td>
</tr>
<tr>
<td>800-T2</td>
<td>800 mm, reverse tapered plate ends</td>
<td>408</td>
<td>1.25</td>
<td>0.93</td>
</tr>
<tr>
<td>800-U</td>
<td>800 mm, rounded and reverse tapered plate ends</td>
<td>343</td>
<td>1.05</td>
<td>0.78</td>
</tr>
<tr>
<td>800-U-C</td>
<td>800 mm, rounded and reverse tapered plate ends, w/ steel clamp</td>
<td>308</td>
<td>0.94</td>
<td>0.70</td>
</tr>
<tr>
<td>400-S</td>
<td>400 mm, square plate ends</td>
<td>180</td>
<td>0.55</td>
<td>0.41</td>
</tr>
<tr>
<td>400-S-W</td>
<td>400 mm, square plate ends, w/ CFRP wrap</td>
<td>200</td>
<td>0.62</td>
<td>0.61</td>
</tr>
<tr>
<td>400-T2</td>
<td>400 mm, reverse tapered plate ends</td>
<td>340</td>
<td>1.04</td>
<td>0.78</td>
</tr>
<tr>
<td>400-T2-W</td>
<td>400 mm, reverse tapered plate ends, w/ CFRP wrap</td>
<td>218</td>
<td>0.67</td>
<td>0.50</td>
</tr>
<tr>
<td>200-T2-W</td>
<td>200 mm, reverse tapered plate ends, w/ CFRP wrap</td>
<td>236</td>
<td>0.72</td>
<td>0.54</td>
</tr>
</tbody>
</table>
Comparison of the failure loads for beams 400-S and 400-S-W indicated that the presence of the CFRP wrap did not significantly increase the joint strength. The measured strains on the surface of the CFRP wrap were dominated by the Poisson's effect, which indicated that the wrap did not have sufficient stiffness to resist the peeling stresses that developed near the end of the splice plate. The measured failure load of the 400-T2-W splice configuration was lower than that of its unwrapped counterpart, 400-T2. This may have been induced by premature cracking of the CFRP wrap in the weak direction or possibly because of a defect in the adhesive layer that was not detected before testing. For all of the wrapped specimens, 400-S-W, 400-T2-W, and 200-T2-W, the debonding failure of the splice plate was concealed by the presence of the CFRP wrap. This is not desirable in practical applications because failure of the splice joints may go undetected during routine inspections.

Comparison of the results for beams 800-U and 800-U-C suggest that the presence of the steel clamp assembly may have reduced the strength of the splice joint. The effect of the steel clamps can be better understood by carefully considering the installation process. The steel plates that made up the clamp assembly were first bolted to the beam before tightening the bolts. This was done to simulate an actual field application in which direct electrical contact between the steel clamp and the CFRP splice plate could possibly induce galvanic corrosion of the clamp or the steel beam. The adhesive was allowed to harden before tightening the steel bolts. Tightening the steel bolts to apply the clamping pressure subsequently induced a slight curvature in the plates. Because of the presence of the cured adhesive between the steel clamp and the splice plate, the curvature of the plates may have induced direct tension forces near the end of the splice. The presence of these tension forces near the plate end can explain the observed reduction of the measured failure load because of the installation of the steel clamp.

FINITE ELEMENT ANALYSIS

To understand the bond characteristics of the CFRP splice joints, the FEA was conducted in two phases. The first phase consisted of a series of two-dimensional linear plane strain analyses to evaluate the distribution of bond stresses for double-lap shear joints with different reverse-taper angles near the splice plate end. The effect of the adhesive properties on the adhesive stresses was also considered. In the second phase, a linear, three-dimensional analysis was conducted to evaluate the distribution of bond stresses in the splice joints for the steel beam tests.

Analysis of Double-Lap Shear Specimens

The effect of the reverse taper on the distribution of bond stresses near the end of the splice plate for double-lap shear joints was studied using a two-dimensional, linear, plane strain finite element model. The CFRP was modeled as an orthotropic material, whereas the adhesive was modeled as an isotropic material using the material properties presented previously. Quarter model symmetry was used, and a uniform tension stress was applied at the end of the coupon. Comparison of the tension strains at the surface of the CFRP splice plate indicated good correlation between the FEA and the experimental results (8).

The analysis results indicated that the maximum adhesive stresses typically occurred at the interface between the adhesive and the CFRP splice plate. The distribution of the shear, normal (peeling), longitudinal, and principal adhesive stresses at the adhesive–splice plate interface are presented in Figure 4 for two different taper angles, B. Inspection of the Figure 4a–d indicates the presence of a significant stress concentration within the taper region near the end of the splice plates with the maximum stresses typically occurring at the corner of the splice plate near the beginning of the taper.

Comparison of the shear stress components in Figure 4b indicates that the magnitude of the maximum shear stress is unaffected by the taper angle. Alternatively, Figure 4c indicates that decreasing the taper angle from 800 to 200 essentially eliminates the normal (peeling) stress concentration near the plate end. The reduction of the normal stresses is likely caused by the localized reduction of the flexural stiffness of the splice plate near the plate end because the thinner plate end can more easily conform to the curvature of the main plate, which is induced by the eccentricity of the tension forces in the double-lap shear specimen. In addition, locally increasing the thickness of the adhesive helps to reduce the stiffness of the adhesive layer and further reduces the stress-concentration effect. Figure 4d indicates the presence of a longitudinal stress concentration near the plate end caused by transfer of stresses from the adhesive to the end of the splice plate by direct axial tension. The figure indicates that increasing the taper angle from 200 to 800 also increases the magnitude of the peak longitudinal stress.

Consideration of the individual stress components indicates that for smaller taper angles, the principal stress distribution, shown in Figure 4a, is dominated by the shear and longitudinal stress components, whereas for larger taper angles, all three stress components contribute to the principal stress. The figure also indicates that, away from the taper region, the stress distributions are essentially unaffected by the presence of the reverse taper. Because the adhesive is brittle and essentially linear-elastic, a maximum principal stress failure criterion can be used to predict the debonding strength of the splices.

The FEA was extended to evaluate the effect of the adhesive properties and the splice length on the distribution of the adhesive stresses. To evaluate the effect of the adhesive properties, the shear modulus of the adhesive layer was varied between 216 MPa and 2160 MPa, whereas the adhesive thickness was varied between 1 mm and 5 mm. Two different taper angles of 200 and 800 were considered. The analysis results indicated that increasing the stiffness of the adhesive layer by increasing the shear modulus or decreasing the thickness of the adhesive generally increased the magnitude of the adhesive principal stresses. The analysis further indicated that, regardless of the taper angle, increasing the overall length of the splice plate beyond 400 mm did not affect the magnitude of the peak adhesive principal stresses. However, decreasing the splice length to less than 400 mm increased the magnitude of the principal stresses. This was likely because of an interaction between the stress concentrations at the end of the splice plate and near the center of the joint. Therefore, for the proposed strengthening system, a critical splice length, l_c, beyond which increasing the splice length will not increase the tension strength of the splice, is defined as 400 mm. For practical applications, it is not recommended to use splices shorter than the critical length.

Analysis of Steel Beam Specimens

To evaluate the distribution of bond stresses for CFRP splice joints used for flexural members, a three-dimensional, linear global–local FEA was conducted. The global model represented the constant moment region of the typical CFRP-strengthened steel beams that
were tested in the experimental program. A maximum element edge length of 5 mm was considered in the analysis. A local model was developed to accurately represent the geometry and to determine the local stresses near the end of the CFRP splice plate. In the region of the plate end, a maximum element edge length of 0.25 mm was specified in the through-thickness direction. For the rest of the local model, the maximum element edge length ranged between 1 and 5 mm, with the smaller elements being used in the CFRP and adhesive materials closer to the plate end. The local model was used to model the different taper angles of the plate end, and the boundary conditions were imported directly from the solution of the global model.

Both models were meshed using 20-node brick elements. The material properties of the CFRP and adhesive were the same as those previously used for the double-lap shear models. The steel was
modeled as a linear, isotropic material with elastic modulus and Poisson's ratio of 205,000 MPa and 0.3, respectively. To validate the results of the FEA, the calculated strains at the surface of the CFRP splice plate were compared with the measured strains from the experimental program for several different splice configurations. The experimental and analytical results showed good correlation (8).

Figure 5 shows the distribution of the individual stress components at the adhesive–splice plate interface for steel beam specimen with a 400-mm-long CFRP splice plate and 200 tapered splice plate ends. The figure also presents the corresponding stresses for a double-lap shear specimen with the same configuration. The applied loads for the two analyses were selected to ensure that the tension stress in the main CFRP plate, away from the splice location, was the same for both specimen types.

Inspection of Figure 5 indicates that the trends of the stresses are similar for the spliced steel beam and the double-lap shear specimen. The magnitudes of the normal and longitudinal stress components, shown in Figures 5b and c, are equal in both cases. However, the adhesive shear and principal stresses, shown in Figure 5a and d, respectively, are slightly higher for the steel beam than for the double-lap shear specimen. This is because of transfer of tension stresses from the tension flange of the steel beam to the CFRP splice plate through the adhesive layer.

The FEA was also used to determine the effect of the taper angle on the distribution of stresses near the end of the CFRP splice plate for the steel beam type specimens. The trend of the results was similar to that obtained for the double-lap shear specimens. Similarly, the analysis results indicate that increasing the length of the splice plate beyond 400 mm does not significantly affect the stress distribution, which suggests that the critical splice length, \( \ell_c \), of 400 mm, is also appropriate for beams with spliced CFRP plates. The results indicate that the mechanisms of the stress transfer from the main CFRP plate to the splice plate are equivalent for both double-lap shear specimens and CFRP splice joints used to strengthen steel beams.

**Proposed Design Guidelines**

On the basis of the results of the research program, the following guidelines were proposed to facilitate the design of CFRP splice joints for steel beams strengthened with CFRP materials. For design purposes, it is convenient to express the strength of a splice joint in terms of its basic tension strength, \( T_v \):

\[
T_v = f_a A_v
\]

where

\[
f_a = \text{adhesive tension strength;}
\]

\[
A_v = \text{bonded surface area along which tension force is transmitted;}
\]

\[
\ell_{b_d} = \text{for double-lap shear joints, \( \ell_a b_a/2 \) for lap spliced CFRP joints for steel beams;}
\]

\[
\ell_a = \text{overall splice length; and}
\]

\[
b_o = \text{width of splice joint.}
\]

For splices longer than the critical length, \( \ell_c \), the critical length should be used to calculate the bonded surface area, \( A_v \), rather than the overall splice length \( \ell_a \).

Because of the effect of the stress concentrations near the end of the splice plate and because of the combined effect of shear, normal, and longitudinal stresses, the basic strength of the joint cannot be achieved for typical splice joints. Further, the shear stiffness of the adhesive also affects the magnitude of the stress concentration in the adhesive. Therefore, the nominal strength of the joint, \( T_n \), can be calculated as follows:

\[
T_n = \frac{T_v}{\psi_s \psi_v}
\]

where \( \psi_s \) is the stress concentration factor to account for the effect of the taper angle and \( \psi_v \) is the stress concentration factor to account for the effect of the adhesive properties.

**FIGURE 5** Distribution of adhesive (a) shear, (b) normal, (c) longitudinal, and (d) principal stresses for steel beam and double-lap shear splices.
The first stress concentration factor, $\psi_a$, accounts for the effect of the taper angle on the distribution of stresses near the plate end. To establish this factor, the results of several analyses with different taper angles ranging from 200 to 800 were considered. The adhesive thickness and shear modulus and the splice length were taken as 1 mm, 1,080 MPa, and 400 mm, respectively. For each joint configuration, the stress concentration factor, $\psi_a$, was defined as the ratio of the critical principal stress in the adhesive obtained from the FEA, $\sigma_{c,\text{ead}}$, to the average adhesive shear stress caused by the effect of the applied load, $\bar{\tau}$. The relationship between the taper angle, $\theta$, and the stress concentration factor, $\psi_a$, obtained from the FEA of the joint and the double-lap shear specimens is shown in Figure 6a. Inspection of the figure indicates that the stress concentration factor for the steel beams is comparable to but typically slightly higher than the stress concentration factor for the double-lap shear coupons. This is consistent with the trend of the measured shear and principal stresses for the two specimen configurations discussed previously. To simplify the design process, the following linear expression, also shown in Figure 6a, is proposed to represent the relationship between the taper angle, $\theta$, and the stress concentration factor $\psi_a$:

$$\psi_a = 0.04(\theta) + 3.1 \quad (3)$$

Several other analyses were conducted for double-lap shear joints with different adhesive moduli, $G_a$, and thicknesses, $t_a$. A second stress concentration factor, $\psi_a$, was defined by Dawood (8) as:

$$\psi_a = \frac{\sigma_{c,\text{ead}}}{\bar{\tau}} \quad (4)$$

The effect of the adhesive shear stiffness, $G_a/t_a$, on the stress concentration factor, $\psi_a$, was obtained from several different FEAs, as shown in Figure 6b. The different lines plotted in the figure represent the effect of different parameters on the stress concentration factor. Two different taper angles, $\theta$, were considered. Further, the shear stiffness of the adhesive, $G_a/t_a$, was varied by varying the adhesive modulus, $G_a$, and the adhesive thickness, $t_a$, independently. The results shown in the figure illustrate the trend that increasing the adhesive shear stiffness also increases the magnitude of the maximum principal stresses in the adhesive. The relationship between the stress concentration factor, $\psi_a$, and the adhesive shear stiffness, $G_a/t_a$, is nonlinear for the typical range of adhesive shear stiffnesses that were considered as shown in Figure 6b. The figure also indicates, however, that for design purposes, a proposed linear relationship can be used that represents an upper bound to the stress concentration factor, $\psi_a$, within the range of adhesive properties considered:

$$\psi_a = 0.0003(G_a/t_a) + 0.69 \quad (5)$$

Once the nominal tension strength of the splice is determined, a suitable reduction factor should be applied for design purposes. Because of the brittle nature of the de-bonding failure, a reduction factor of 0.75 is proposed. For flexural members, splices should be installed at a location along the length of the girder where the maximum expected tension force in the CFRP, because of the effect of all applied loads, does not exceed the ultimate tension strength of the splice. The tension force in the CFRP can be determined by structural analysis and moment-curvature analysis of the strengthened section (4).

The proposed design guidelines were used to predict the de-bonding load of the tested double-lap shear specimens and steel beams. The predicted strength is compared with the measured failure loads in Table 3 and Table 4 for the double-lap shear specimens and tested steel beams, respectively. Inspection of the tables indicates that the proposed method is conservative in all cases. In general, the member and the strengthening system should be proportioned to prevent other failure modes, including, but not limited to, yielding of the steel tension flange, buckling of the compression flange, fatigue cracking, or shear failure of the web.

CONCLUSIONS

The findings of the current research indicate that the reverse tapered plate end detail significantly reduces the magnitude of the adhesive stress concentrations near the end of a CFRP splice plate, thereby increasing the tension strength of the joint which is similar to the trend observed by researchers in other industries (6, 7). The experimental results further indicated that increasing the splice length...
TABLE 3 Comparison of Proposed Design Guidelines with Experimental Results for Tested Double-Lap Shear Specimens

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Taper Angle</th>
<th>( G_{f} ) ((N/mm^2))</th>
<th>( \psi_0 )</th>
<th>( \psi_e )</th>
<th>Tension Strength, ( T_e ) (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>400-S(1)</td>
<td>90°</td>
<td>6.70</td>
<td>1.080</td>
<td>1.01</td>
<td>89</td>
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<tr>
<td>400-S(3)</td>
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<td></td>
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TABLE 4 Comparison of Proposed Design Guidelines with Experimental Results for Tested Steel Beams

<table>
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<tr>
<th>Beam ID</th>
<th>Splice Length (mm)</th>
<th>Taper Angle</th>
<th>( G_{f} ) ((N/mm^2))</th>
<th>( \psi_0 )</th>
<th>( \psi_e )</th>
<th>Splice Tensile Strength, ( T_e ) (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>800-S(1)</td>
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<td>6.70</td>
<td>1.080</td>
<td>1.01</td>
<td>112</td>
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<tr>
<td>800-S(2)</td>
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<td>90°</td>
<td>6.70</td>
<td>1.080</td>
<td>1.01</td>
<td>112</td>
</tr>
<tr>
<td>800-T2</td>
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<td>3.90</td>
<td></td>
<td></td>
<td></td>
<td>193</td>
</tr>
<tr>
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<td>1.080</td>
<td>1.01</td>
<td>112</td>
</tr>
<tr>
<td>400-T2</td>
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<td>3.90</td>
<td></td>
<td></td>
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<td>193</td>
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</tbody>
</table>

or providing additional mechanical anchorage near the ends of the splice plate did not significantly increase the tension strength of the joint. On the basis of the experimental and analytical findings, design guidelines are proposed to predict the strength of CFRP double-lap shear joints and splice joints used to strengthen steel flexural members. Comparison of the predicted strength of the splice joints using the proposed model to the measured failure loads for several of the tested specimens indicated that the proposed guidelines are conservative in all cases within the limitations of the parameters that were considered in the current study. Because of the brittle nature of the debonding failure mode, the proposed strengthening system should not be implemented in applications that require considerable energy dissipation or repeated loading to levels close to the failure load such as may occur in seismic events. The findings of the current study indicate that, with proper detailing, the proposed strengthening system may be an attractive option to strengthen long-span steel bridges and structures that are subjected primarily to gravity loading.

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REFERENCES