Strengthening Shear Deficient Thin-Walled Steel Beams by Bonding Pultruded GFRP Sections

Tuna Ulger
Louisiana State University and Agricultural and Mechanical College, tunaulger@yahoo.com
STRENGTHENING SHEAR DEFICIENT THIN-WALLED STEEL BEAMS 
BY BONDING PULTRUDED GFRP SECTIONS

A Dissertation

Submitted to the Graduate Faculty of the 
Louisiana State University and 
Agricultural and Mechanical College 
in partial fulfillment of the 
requirements for the degree of 
Doctor of Philosophy 

in 
The Department of Civil and Environmental Engineering

by 
Tuna Ulger 
B.S., Ege University, 2007 
M.E., Texas A&M University, 2012 
August 2016
All the praises and thanks be to Allah, the owner of life here and hereafter.

To my father and mother for their guidance and prayer

Muhittin Ulger

and

Aysel Ulger

To my wife and daughter for their encouragement and patience

Esra Ulger

and

Elif Sena Ulger

O Allah give us better future and guide us in the right path.
Acknowledgments

I would like to first express all my deepest respect and gratitude to my academic advisor, Dr. Ayman M. Okeil, for his continuous support to earn my degree. I thank him for guiding me with his immense knowledge and being more than an advisor from beginning to the end. Nothing is enough to express my sincere respect to him.

Beside my advisor, I thank my committee members, Dr. Steve Cai, Dr. Michele Barbato, and Dr. Guoqiang Li, for assisting me during my research and giving their valuable advises. And also, I thank Dr. George Voyiadjis for his support during my study.

Special thanks first go to Kazim Sekeroglu and his family for being a supportive companion, and then to all other friends with whom we shared the good moments during my study.

Finally, in addition to financial grant from the National Science Foundation and scholarship from Republic of Turkey Ministry of National Education, the donation of materials by Fyfe Co., LLC, and Bedford Reinforced Plastics, Inc. and support from Strongwell Corporation are greatly appreciated. Additional support from the Department of Civil and Environmental Engineering at Louisiana State University is also acknowledged.
# Table of Contents

Acknowledgments .................................................................................................................. iii

Abstract .................................................................................................................................................. viii

Chapter 1. Introduction ................................................................................................................... 1
  1.1 Retrofitting Using Composite Materials ................................................................................. 2
    1.1.1 Concrete Structures ........................................................................................................... 2
    1.1.2 Steel Structures ............................................................................................................... 5
  1.2 Strengthening by Stiffening (SBS) – Proposed Retrofitting Method ........................................ 8
    1.2.1 Beam Specimens ............................................................................................................ 8
    1.2.2 Adhesives ....................................................................................................................... 9
    1.2.3 Pultruded Stiffeners ........................................................................................................ 11
    1.2.4 Surface Preparation and Bonding .................................................................................. 12
  1.3 Data Acquisition ...................................................................................................................... 13
    1.3.1 Strain Gauges .................................................................................................................. 13
    1.3.2 Displacement Sensors .................................................................................................... 14
  1.4 Main Structure of the Chapters .............................................................................................. 15
  1.5 References ............................................................................................................................ 17

Chapter 2. Effect of Initial Panel Slenderness on Efficiency of Strengthening-By-Stiffening using FRP for Shear Deficient Steel Beams ........................................................................................................ 20
  2.1 Introduction ............................................................................................................................ 20
  2.2 Literature Review ................................................................................................................... 21
  2.3 Proposed Strengthening Method ............................................................................................ 22
  2.4 Experimental Program .......................................................................................................... 23
    2.4.1 Beam Specimens ............................................................................................................ 23
  2.5 Material Properties ............................................................................................................... 25
    2.5.1 Pultruded GFRP Section ............................................................................................... 25
    2.5.2 Adhesive Type and Properties ...................................................................................... 26
  2.6 Specimen Preparations .......................................................................................................... 27
  2.7 Experimental Setup ............................................................................................................... 28
  2.8 Results and Discussion ......................................................................................................... 30
    2.8.1 Load-deflection Curves ................................................................................................. 30
    2.8.2 Initial Global Stiffness .................................................................................................... 33
    2.8.3 Ductility ....................................................................................................................... 35
    2.8.4 Strain Readings ............................................................................................................. 35
  2.9 Conclusions .......................................................................................................................... 37
  2.10 Nomenclature ....................................................................................................................... 39
  2.11 References .......................................................................................................................... 40
<table>
<thead>
<tr>
<th>Chapter 3.</th>
<th>Strengthening-By-Stiffening: FRP Configuration Effects on Behavior of Shear-Deficient Steel Beams</th>
<th>43</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.1</td>
<td>Introduction</td>
<td>43</td>
</tr>
<tr>
<td>3.2</td>
<td>Literature Review</td>
<td>44</td>
</tr>
<tr>
<td>3.3</td>
<td>Experimental Program</td>
<td>49</td>
</tr>
<tr>
<td>3.3.1</td>
<td>Specimen Details</td>
<td>49</td>
</tr>
<tr>
<td>3.3.2</td>
<td>Adhesives</td>
<td>51</td>
</tr>
<tr>
<td>3.4</td>
<td>Material Properties</td>
<td>51</td>
</tr>
<tr>
<td>3.4.2</td>
<td>GFRP Stiffeners</td>
<td>52</td>
</tr>
<tr>
<td>3.4.3</td>
<td>CFRP Sheets</td>
<td>53</td>
</tr>
<tr>
<td>3.5</td>
<td>Specimen Preparations</td>
<td>54</td>
</tr>
<tr>
<td>3.5.1</td>
<td>Surface Treatments</td>
<td>54</td>
</tr>
<tr>
<td>3.5.2</td>
<td>Bonding the Composites</td>
<td>54</td>
</tr>
<tr>
<td>3.5.3</td>
<td>Tested Specimens</td>
<td>55</td>
</tr>
<tr>
<td>3.6</td>
<td>Experimental Results</td>
<td>56</td>
</tr>
<tr>
<td>3.6.1</td>
<td>Load-Deflection Curves</td>
<td>56</td>
</tr>
<tr>
<td>3.6.2</td>
<td>Post Buckling and Ductility</td>
<td>63</td>
</tr>
<tr>
<td>3.6.3</td>
<td>Strain Readings</td>
<td>66</td>
</tr>
<tr>
<td>3.7</td>
<td>Comments on Strengthening Alternatives</td>
<td>69</td>
</tr>
<tr>
<td>3.8</td>
<td>Conclusions</td>
<td>70</td>
</tr>
<tr>
<td>3.9</td>
<td>Nomenclature</td>
<td>72</td>
</tr>
<tr>
<td>3.10</td>
<td>References</td>
<td>72</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Chapter 4.</th>
<th>Mixed Mode Fracture Properties of Adhesives for FRP Strengthening of Steel Structures</th>
<th>76</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.1</td>
<td>Introduction</td>
<td>76</td>
</tr>
<tr>
<td>4.2</td>
<td>Literature Review</td>
<td>78</td>
</tr>
<tr>
<td>4.3</td>
<td>Mixed Mode Fracture Investigation</td>
<td>80</td>
</tr>
<tr>
<td>4.3.1</td>
<td>Calculation of Phase Angles Using Theoretical Formulations</td>
<td>82</td>
</tr>
<tr>
<td>4.3.2</td>
<td>Validation of Epoxy Submodel</td>
<td>83</td>
</tr>
<tr>
<td>4.3.3</td>
<td>Full Beam Model</td>
<td>83</td>
</tr>
<tr>
<td>4.3.4</td>
<td>Refined Epoxy Submodel</td>
<td>85</td>
</tr>
<tr>
<td>4.4</td>
<td>Mixed Mode Single Leg Bending (SLB) Tests</td>
<td>86</td>
</tr>
<tr>
<td>4.4.1</td>
<td>Extraction of Fracture Energies Using Theoretical Formulation</td>
<td>88</td>
</tr>
<tr>
<td>4.4.2</td>
<td>Material Properties</td>
<td>89</td>
</tr>
<tr>
<td>4.4.3</td>
<td>Experimental SLB Test Procedure</td>
<td>90</td>
</tr>
<tr>
<td>4.5</td>
<td>Phase Angle Results</td>
<td>94</td>
</tr>
<tr>
<td>4.5.1</td>
<td>Effect of Epoxy Thickness</td>
<td>94</td>
</tr>
<tr>
<td>4.5.2</td>
<td>Effect of Element Location</td>
<td>95</td>
</tr>
<tr>
<td>4.5.3</td>
<td>Effect of Web Thickness</td>
<td>96</td>
</tr>
<tr>
<td>4.5.4</td>
<td>Effect of Crack Length</td>
<td>98</td>
</tr>
<tr>
<td>4.5.5</td>
<td>Effect of Epoxy Type</td>
<td>99</td>
</tr>
<tr>
<td>4.6</td>
<td>SLB Test Results</td>
<td>101</td>
</tr>
<tr>
<td>4.6.1</td>
<td>Image Data</td>
<td>101</td>
</tr>
<tr>
<td>4.6.2</td>
<td>Fracture Properties of Epoxies Obtained from SLB Test</td>
<td>101</td>
</tr>
<tr>
<td>4.6.3</td>
<td>FE Simulation of SLB specimens</td>
<td>104</td>
</tr>
</tbody>
</table>
4.7 Conclusion ................................................................. 106
4.8 Nomenclature ............................................................ 108
4.9 References ............................................................... 109

Chapter 5. Numerical Analysis of SBS Retrofitted Beams and Design Considerations ..... 114
5.1 Introduction ............................................................ 114
5.2 Experimental Investigation ........................................... 117
  5.2.1 Test Set-up and Specimens ....................................... 117
5.3 Material Properties .................................................... 119
  5.3.1 Steel .............................................................. 119
  5.3.2 Adhesives ......................................................... 120
  5.3.3 Composites ....................................................... 120
5.4 FE Model ............................................................... 122
  5.4.1 Initial Imperfections ................................................ 123
  5.4.2 Adhesive Model .................................................. 124
  5.4.3 GFRP Stiffeners ................................................ 125
  5.4.4 Mesh Size ......................................................... 126
5.5 Parametric Study ...................................................... 128
  5.5.1 GFRP Stiffeners ................................................ 128
  5.5.2 Panel Aspect Ratio ............................................... 130
  5.5.3 Slenderness ...................................................... 130
5.6 FE Model Validation ................................................... 131
5.7 Results and Discussions ............................................... 136
  5.7.1 GFRP Stiffener Size Study ...................................... 136
  5.7.2 Panel Aspect Ratio and Slenderness ......................... 137
5.8 Investigation of Possible Use of SBS in New Construction .......... 139
  5.8.1 Results .......................................................... 141
5.9 Conclusions ........................................................... 143
5.10 Nomenclature .......................................................... 146
5.11 References ............................................................ 147

Chapter 6. Summary and Conclusions .................................................... 150

Appendix I. Effect of Adhesive Type on Strengthening-By-Stiffening for Shear-deficient
Thin-walled Steel Structures .................................................. 154
  A. I Abstract .................................................................. 154
  A. I 1 Introduction ........................................................ 154
  A. I 2 Experimental Program .......................................... 157
    A. I 2.1 Epoxy Adhesives Considered in this Study ............ 158
    A. I 2.2 Epoxy Preparation and Bonding Procedure .......... 159
    A. I 2.3 Beam Specimens ............................................ 159
  A. I 3 Experimental Results ............................................. 162
    A. I 3.1 Epoxy Tests .................................................. 162
    A. I 3.2 Beam Tests .................................................. 165
A. I 4 Conclusions ........................................................................................................................................ 170
A. I 5 Nomenclature ................................................................................................................................... 171
A. I Acknowledgements ............................................................................................................................... 171
A. I References ............................................................................................................................................. 172

Appendix II. Letters of Permissions ........................................................................................................... 174
A. II 1 Chapter 2. Effect of Initial Panel Slenderness on Efficiency of Strengthening-By-Stiffening Using FRP for Shear Deficient Steel Beams .............................................................................. 174
A. II 2 Appendix I. Effect of Adhesive Type on Strengthening-By-Stiffening for Shear-deficient Thin-walled Steel Structures ........................................................................................................ 175

Vita .................................................................................................................................................................. 178
Abstract

Rehabilitation and retrofitting methods offer economical and feasible alternatives for upgrading aged and deficient structures. Structural strengthening using Fiber Reinforced Polymer (FRP) composites have been widely investigated by researchers and used in field applications. The main advantage of FRP composites is the superior mechanical properties they offer over traditional structural materials. One novel alternative of these retrofitting methods was developed at Louisiana State University and called “Strengthening-by-Stiffening” (SBS). In SBS, the external strengthening of shear deficient thin-walled steel structures is achieved by bonding pultruded FRP sections to buckling prone web panels. Contrary to the commonly used uniaxial tension resistance of fibers, here, the geometric properties of pultruded FRP sections play the most important role in stiffening vulnerable thin plates. The research started by testing a series of full size steel beams before and after introducing SBS. The first web panel between the bearing and transverse steel stiffeners was selected as a control panel, and a point load was chosen in an asymmetric three-point loading setup. The experimental investigation was conducted considering different web panel aspect ratios (1.0:1.0; 1.5:1.0), web thicknesses (1/8; 5/32 inch), epoxy types (brittle; ductile), Glass FRP (GFRP) configuration (geometry and orientation). For comparison purposes, one conventionally strengthened beam (by welding additional steel stiffeners) and one beam strengthened by bonding Carbon FRP (CFRP) sheets to the critical web panel were also tested. The experimental tests showed that the global failure mechanism was mainly controlled by the debonding of adhesive layer. Therefore, failure modes and phase angles were investigated for the GFRP/steel interface. Local traction-separation laws for Mode I and Mode II failure modes were determined by conducting single leg bending (SLB) tests, in which digital image capturing and processing techniques were used to determine crack tip displacement fields. Delamination failure within the pultruded GFRP stiffeners was also simulated following Hashin’s failure criteria. Finally, effective SBS design parameters were investigated using an FE model that takes the adhesive’s mixed mode fracture into account using a cohesive zone model (CZM), which was validated using experimental results. Possible
extension of SBS to new construction was studied to explore creating beams free from transverse steel stiffeners by fully bonding the GFRP stiffeners as a substitute for welding of transverse and bearing steel stiffeners as a means for improving the fatigue.
Chapter 1. Introduction

Structural systems are important for economic growth and social development. Many of these structures are prematurely coming to the end of their useful service lives due to the increasing degradation and demand over the years. These structures can be fully replaced to serve with full capacities; however, the cost of replacement in some circumstances cannot be considered an economical or feasible solution. Therefore, more economical and feasible options such as rehabilitation, retrofitting and partial replacement become a preferable alternative to full replacement.

Conventional retrofitting techniques have been practiced to increase the load capacities and performance of deficient structures. Common methods such as external prestressing, jacketing, and stiffener welding have been implemented in real structures successfully. For example, concrete columns can be retrofitted by casting concrete jackets or wrapping steel jackets to increase their capacity or prevent buckling, or steel girders can be retrofitted by welding longitudinal or transverse steel stiffeners to prevent buckling of the flange or web plates. However, these conventional strengthening methods bring several disadvantages. For example, concrete jacketing adds large amount of frame work and dead weight in structure, or heavy steel plates reduce the mobilization and extend the time frame for retrofitting, or localized stress concentrations due to on site welding and related fatigue problems. Recent advancements in composite and adhesive technologies revealed the bonded strengthening alternatives for structural engineering. Fiber-reinforced polymers (FRPs), for instance, are widely used in research activities and field implementations; and FRPs have become a part of the customary structural materials for retrofitting deficient structures. Lightweight, corrosion and fatigue resistance, high tensile strength, and flexible placement are some prominent properties of FRP composites.

In this chapter, the use of composites materials and adhesives for retrofitting concrete and steel structures are presented. The proposed, “Strengthening-by-Stiffening” (SBS), retrofitting
method will then be presented. Finally, the main structure of each chapter is summarized at the end of the introduction.

1.1 Retrofitting Using Composite Materials

1.1.1 Concrete Structures

Structural strengthening by bonding FRP composites has been widely investigated and implemented in numerous field applications. The first bonded FRP application was the IBACH Bridge in Switzerland in 1991 (Stratford et al. 2004). It has been reported that the most successful strengthening applications were reported for concrete and wood structures (Sen et al. 2001) because of these materials inherently lower elastic modulus relative to FRP composites. Ritchie et al. (1991) tested sixteen under-reinforced concrete beams bonded with FRP plates in their tension region to investigate the performance of those beams under static loading. Carbon, glass and Kevlar fibers were bonded using two component epoxies. The initial stiffness and ultimate strength increase over control specimens ranged from 17 to 99% and 40 to 97%, respectively. The selected epoxy’s strength, Fusor 320/322, was higher than that of concrete; therefore, only two failure modes were observed at the maximum moment region, while other failure modes occurred at the end of FRP plates (spalling of concrete). Triantafillou et al. (1992) tested eight concrete beams strengthened with unidirectional Carbon FRP (CFRP) sheets. The results of those experimentally tested beams were used to construct a numerical model for externally strengthened concrete beams in flexure. The observed failure modes were steel yield-FRP rupture, steel yield-concrete crushing, compressive failure and debonding of FRP strengthened beams. It was observed that FRP debonding limits the ultimate flexural capacity of the stiffened beams with brittle failure (Triantafillou et al. 1992). A similar flexural and shear retrofitting investigation were conducted by Norris et al. (1997). CFRP composites bonded to bottom and side surfaces of the concrete beams had different orientations. CFRP strips were employed on the side faces, which resulted in a considerable increase in the ultimate capacity of the retrofitted beams. CFRP strips perpendicular to the cracks exhibited high strength and stiffness increase but failed in a brittle manner. However, CFRP strips diagonally placed with
respect to crack orientation resulted in less strength and stiffness increase but failure was more ductile with early warning signs. Lamanna et al. (2004) proposed mechanical fasteners for externally stiffened concrete beams to reduce the brittle failure of concrete/FRP bond at the ends. Pre drilled holes with various depths reduced the initial crack and exhibited higher strength and ductility. The observed failure modes were compression failure and FRP detachments of the anchored FRP composites. Yield moment and ultimate moment values increased up to 21.6 and 20.1% with the proposed FRP anchoring system.

Bae et al. (2013) investigated reinforced concrete T beams that were retrofitted by bonding CFRP strips in shear deficient region, and these specimens were tested to failure with and without CFRP strengthening under static load. Other CFRP strengthened concrete beams were tested to failure after the beams were exposed to 2 million cycles of repetitive shear loading to simulate fatigue conditions. The shear load capacity of the CFRP strengthened beams was enhanced by about 26% more than unstrengthened beam’s capacity. Mofidi et al. (2014) strengthened reinforced concrete T-beams in shear by bonding L shaped CFRP strips on both side walls. Grooves perpendicular to the beam’s longitudinal axis were made on the side-wall of two specimens, and CFRP strips were bonded into these groves partially and fully embedded. Six T-beams including the control T-beam and shear strengthened beams without grooves were tested to failure, and shear capacity of the strengthened beams increased 40% in average over the control T beam. The partially embedded configuration was found to be more effective among the proposed strengthening techniques. The schematic view of fully embedded shear strips are shown in Figure 1.1.

![Figure 1.1 Details of fully embedded CFRP shear strips (a) elevation (b) section view](Mofidi et al. 2014)
Other than flexural and shear strengthening of reinforce concrete beams, confining reinforced concrete columns and shear walls have promising results. Mirmiran et al. (1996) implemented FRP confining to hollow concrete column sections instead of steel confinement. The confined column illustration can be seen in Figure 1.2. This technique offered high strength, ductility and durability to the FRP confined columns in addition to the lightweight and corrosion resistance of FRP composites over steel sections. A similar approach was applied to bridge piers to improve the seismic performance of those reinforced concrete piers. Column specimens were formed in 1/5 scale, and possible plastic regions, which were the predefined length from the footing, were wrapped using FRP straps. Axial and lateral loads were applied to obtain hysteresis curves of the retrofitted columns. Buckling failure of the longitudinal bars were postponed, higher displacement ductility and stable hysteresis loops were obtained as a result of FRP strap confinement (Saadatmanesh et al. 1996). The effect of slenderness on the performance of the FRP confined columns was theoretically modeled by Jiang et al. (2013), and their results showed that slenderness reduces the performance of FRP confinement.

El-Sokkary et al. (2013) experimentally tested three reinforced concrete shear walls with two different CFRP strengthening configurations, and one of three was a control specimen without CFRP strengthening configuration. The first shear wall was fully wrapped, and the

Figure 1.2 FRP-concrete composite columns (Mirmiran et al. 1996)
second wall was braced diagonally on both sides in addition to the horizontal strips at the top and bottom of the shear wall. Constant axial load and synchronized shear and moment loads were applied at the top of the shear walls. Flexural capacities of the first strengthened wall and second stiffened wall were increased 80 and 50%; however, displacement ductilities were reduced 50 and 15%, respectively after retrofitting.

As can be seen from aforementioned studies, the concept of retrofitting concrete structure by bonding composite materials is well established, and plenty of studies about FRP retrofitted concrete structures can be found in literature and field applications. This level of maturity in such retrofitting techniques and the confidence in their performance led to the development of several design guidelines and codes. For example, a design guide for externally bonded FRP strengthening systems for concrete structures (ACI-440.2R 2008) is now available and has undergone two major revisions. In the meantime, FRP retrofitted steel structures have recently gained attraction and most of the studies are still at the research level. Some of these research efforts are presented in next section.

1.1.2 Steel Structures

Similar use of planar composites (i.e. CFRP plate and laminates) have been investigated in previous studies. The conducted studies for retrofitting steel structures using composites are fewer mainly because of the lesser efficiency of composites in retrofitting steel structures due to the superior properties of steel (Ulger et al. 2016). Therefore, high modulus FRPs are the main choice of researchers in literature for steel retrofitting applications, and several examples are presented in this section.

Sen et al. (2001) retrofitted full size concrete-steel composite girders bonding CFRP strips (2 and 5 mm thickness) to the bottom flange of the steel girders. A similar size composite beam were also retrofitted by adding anchorage at the end of CFRP strips to eliminate the premature peeling failure. Pre-yielding stiffnesses and load capacities of the retrofitted composite girders increased by up to 67% and 52% using 2 and 5 mm thick CFRP strips, respectively. Bonding was recommended in the addition to anchoring the CFRP laminates at the end because
of the uniform stresses transfer along CFRP laminates. Another four full scale deteriorated steel bridge girders retrofitted with pultruded CFRP strips were tested by Miller (Miller 2001). Deterioration of the bottom flanges was estimated to be 13 and 32% of undeteriorated conditions. Pultruded CFRP strips were bonded to the top and bottom surfaces of the tension flanges, and the ultimate failure load for the two beams increased 17% and 25% over the estimated capacity of deteriorated girders. The initial stiffnesses of these two beams increased by 10% and 37%. Fatigue tests were conducted on another two beams to investigate the long-term performance of retrofitted beams, and promising results were obtained. Similar retrofitting was applied to a girder in a real bridge, and field loading tests revealed 11.6% increase in stiffness from pre-retrofitted condition. Concrete-steel composite beams were strengthened by bonding CFRP strips to the tension side of the beams’ bottom flange and webs (Al-Saidy et al. 2007). Experimental results showed that the load capacities increased between 21 and 45% depending on the amount and elastic modulus of the CFRP strips. Failure occurred at the interface between steel and CFRP which caused slight reduction in terms of ductility. Galal et al. (2012) strengthened artificially deteriorated steel beams by bonding CFRP sheets and plates to the bottom flange of the beams. Bonded CFRP sheets using two different epoxies and anchored CFRP plates without bonding were installed in five layers in deteriorated regions. Two failure modes were observed; debonding at the interface of steel/CFRP, and rupture at the CFRP plates. The strength increase was 25% compared to the artificially deteriorated beams. An anchorage system was also investigated. It did not contribute to the flexural capacity of the deteriorated steel beams but showed ductile behavior similar to epoxy bonded system.

Narmashiri et al. (2010) used uniaxial CFRP strips to enhance the shear load capacity of transversely stiffened steel beams. CFRP strips were vertically bonded to one or both surfaces of the control web panels with different amounts of CFRP. Okuyama (2012) tested steel beams where uniaxial and biaxial CFRP sheets were bonded to their webs. CFRP sheets overlapped diagonally to create biaxial CFRP sheets in this experiment. Square and rectangular panels between the transverse steel stiffeners were stiffened by bonding uniaxial and biaxial CFRP
sheets. One rectangular panel beam with different CFRP sheet orientations is shown in Figure 1.3. The load capacities of the stiffened beams over unstiffened beam increased by about 12 and 29\% for square and rectangular panel beams, respectively. Xiao et al. (2012) retrofitted the thin walled hollow connections by wrapping CFRP sheets around the fatigue cracked region. Square and rectangular hollow sections (SHS and RHS) cross welded to each other to obtain T joints, and these joints were wrapped with CFRP sheets at crossing joints to increase fatigue performance of hallow joints; however, experimental results did not show any increase by wrapping only CFRP sheets due to peeling effect at the corners. The same method was repeated on similarly cracked joints by adding L-shaped steel plates between the CFRP sheets, which can be seen in Figure 1.4. The updated method showed increase in flexural and fatigue resistance.

Gao et al. (2013) confined circular steel braces by wrapping them with different number of CFRP layers. Two different initial out of plane imperfections (2.4 and 4.8 mm) were imposed at mid-height of steel braces. Axial deformation and lateral deflection of the retrofitted braces were measured with given experiments varying the number of CFRP layers (2, 4, 6 and 8). Compressive strengths of the confined steel braces increased from 28 to 124\% with 2.4 mm initial out plane imperfection, and 25 to 105\% with 4.8 mm initial out of plane imperfection as the numbers of CFRP layers increased. Initial imperfections did not affect the axial resistance significantly; however, larger lateral deflections were observed for the larger initial imperfections. Lesani et al. (2013) analytically studied tubular T-joints with and without GFRP strengthening. The results showed 22 to 68\% improvements in joint strength depending on the number of GFRP layers for that proposed wrapping scheme.

Figure 1.3 (a) Biaxial and (b) uniaxial bonding form of CFRP sheets (Okuyama 2012)
Contrary to the aforementioned planar utilization of composites in retrofitting concrete and steel structures, pultruded FRP stiffeners in SBS provide additional stiffness with its pultruded geometry and mechanical properties. In other words, main strength contribution relies on the out-of-plane resistance of pultruded stiffener geometry, which originates from the elastic modulus and moment of inertia of the pultruded sections. The concept of the proposed SBS retrofitting method is illustrated in Figure 1.5. The main concept behind the proposed SBS technique is the utilization of pultruded FRP sections to enhance the capacity of shear deficient thin-walled steel structures.

1.2.1 Beam Specimens

In the scope of this dissertation, one specific application of SBS method was investigated to enhance the shear strength of buckling prone thin-walled steel web panels of built-up I-section steel beams by bonding pultruded FRP stiffeners. In the design of the steel beam specimens, web panel thickness was relatively reduced to account for typical structural deficiencies; such as corrosion and artificial degradation. The experimental test configuration was designed to cause the steel beams to fail in shear. Therefore, an asymmetric three point loading test set-up was devised causing a shear load the critical panel that between the bearing and first transverse steel stiffener under the applied load. The critical panel in the later chapters refer the web panel which was subjected to the shear induced failure.
1.2.2 Adhesives

Advancement in retrofitting deficient steel structures followed the developments of retrofitting techniques for concrete structures using composites; therefore, the state of knowledge about retrofitting steel structures is not yet as mature as it is for concrete structures. In light of the existing retrofitting studies, substrate failures such as spalling most likely occur in externally strengthened concrete specimens. However, substrate failure of strengthened steel structures is not an issue for steel structure retrofitting because superior mechanical properties of steel does not allow such a failure to occur. Therefore, one of the major issues in bonded steel strengthening is the cohesion failure at the interface or the substrate failure when steel is bonded with a lower modulus material such as FRP. Flexural and shear strengthening and confining either axial members or joints account for the majority of concrete strengthening. FRP composites have been traditionally utilized mainly for their in plane resistances in both concrete and steel structures. As a result, adhesives are one of the most important links in either concrete or steel bonded applications. Therefore, this section provides experimental studies on adhesives that are commonly used in structural retrofitting.

Epoxy resins which are commonly used in structural applications fail in a brittle manner (Lee et al. 1967). Several researchers tested plain epoxy specimens under tension, shear and
compression loading at different strain rates and temperatures (Fiedler 2001; Gilat et al. 2007; Littell 2008). Fiedler (2001) conducted tests where tension coupons failed at smaller strain levels, and compression and shear coupons failed at higher strain levels. Gilat et al. (2007) conducted tension, compression, and shear tests. Tension specimens failed in a brittle manner; however, compression and shear specimens revealed a more ductile behavior. The effect of low strain rates on the failure behavior of plain epoxy coupons was a more ductile response in tension tests, and medium and high strain rates resulted in a brittle failure (Gilat et al. 2007). In addition to the results reported by Gilat et al. (2007), tension coupons at elevated temperatures failed at smaller stress levels (Deb et al. 2008; Littell 2008).

Some researchers investigated epoxy types with additional materials to improve the mechanical properties (Dean 2004; Imanaka et al. 2009; Zavareh et al. 2012). Dean (2004) added rubber particles in epoxy mix to reduce the inherent brittle failure of coupons and increase the deformation ability. Similar efforts were tried and implemented by Imanaka et al. (2009) where liquid rubber and cross linked rubber particles were added to the epoxy mix. Zavareh et al. (2012) added bitumen to the epoxy mix; therefore, the toughness of the coupons was increased without effecting other mechanical properties of plain epoxy coupons.

The need for ductile epoxy to improve the performance of structural retrofitting is obvious as understood from the previous studies, and new adhesives are addressing the issue with advanced properties. Yu et al. (2012) constructed a CFRP/steel specimen that was bonded with two different adhesives to define the bond slip model of the adhesives. The investigated parameters were the thickness of epoxy layer between the specimens and strength of CFRP laminates in uniaxial direction. The experimental results showed that trapezoidal bond-slip model obtained when ductile adhesives were used, and triangular bond-slip model obtained when brittle adhesive was used. Bond-slip curves confirm the excessive deformation capacity of ductile adhesives. Similarly, interfacial fracture energy between the CFRP and steel substrates that was also bonded with ductile adhesive revealed trapezoidal bond slip behavior (Fernando et al. 2013). Saldanha et al. (2013) investigated a new epoxy, XNR 6852, that already exhibits large
deformation and toughness without additional rubber or liquid additives. The performance of the new epoxy was promising and elongation capacity before fracture was about 100%.

As a result, two different commercially available structural adhesive types were used to bond GFRP stiffeners in the current study. The first one is Tyfo® S epoxy, and is promoted for general structural bonding application. The second one, Tyfo® MB3 epoxy, is a more ductile epoxy, and is promoted specifically for steel bonding applications. Material properties of each adhesive will be provided in related chapters. The published work to present uniaxial tension properties of both epoxy types are provided in Appendix I.

1.2.3 Pultruded Stiffeners

Research on steel strengthening applications using composite materials generally focus on CFRP because of its superior mechanical properties over lower elastic moduli counterparts such as glass fiber reinforced polymer (GFRP) and Kevlar fiber reinforced polymer (KFRP) composites. Resistance contribution of CFRP fibers is higher than other FRPs, and structures strengthened by externally bonding CFRP are typically more brittle (Okeil et al. 2009). The performance of a strengthened structure becomes less brittle when GFRP and KFRP are introduced to the strengthening applications (Triantafillou et al. 1992), but the required amount of GFRP and KFRP composites to strengthen a deficient steel structure will be larger than the amount required if CFRP is used (Sen et al. 2001; Triantafillou et al. 1992). Considering the utilization of pultruded FRP sections in SBS retrofitting, the superior mechanical properties of CFRP composites are not necessary because the strength of FRP section becomes less relevant in SBS applications. Instead, the FRP stiffeners flexural rigidity becomes more prominent. As a result, pultruded GFRP composites were chosen as stiffeners in this study. The pultruded GFRP sections were selected from commercially available wide flanges (WF) beams (EXTERN® 500, and PROForm® WF). T-shaped GFRP stiffeners were obtained by cutting one flange of the WF beams. It should be noted that the alternative FRP composites (i.e. CFRP) can be utilized to extend the work in this dissertation. The material and geometric properties of the chosen GFRP stiffeners will be given in each chapter.
1.2.4 Surface Preparation and Bonding

Surface preparation of the substrates is an extremely significant process in bonding applications. The proposed external strengthening technique uses steel and pultruded GFRP sections as the substrates. Both sections require different surface pretreatments to increase the bond quality between the sections. Several pretreatment methods can be considered for metallic and nonmetallic surfaces such as mechanical, chemical, and plasma method; however, the most effective method is the mechanical pretreatment to increase the surface roughness (Baldan 2004). The bond performance of metallic surfaces can be enhanced with wetting agents or chemical etchants; however, environmental effects and the elevated curing temperatures of the chemicals restrict the chemical use in surface treatments (Wegman et al. 2012). Only mechanical pretreatment was considered to create some degree of mechanical engagement between the steel and GFRP sections. Typically, sand papers, poly abrasive wheels and grid blasting are common tools used for the surface preparations.

Metallic surface preparations require more effort than nonmetallic surfaces (Harries et al. 2012). The beams were prefabricated and transported directly to the lab. Hence, they were not subjected to large scale contaminants such as mud, oil or grease in testing environment; therefore, the first step is to free the bond surface from any rust and paint. The poly abrasive wheels attached to a hand drill were used until the white metallic texture was reached (Schnerch 2007). Chemical cleaning agents were not used until the white metallic surface was revealed. The dry abrasive action by nature produces some fine particles that become a barrier between the surfaces and reduce the bond performance. These unwanted particles were removed using a cleaning agents i.e. acetone. A rag wetted with acetone was used to wipe the white textured surface in one direction out of the bonding area. The epoxy layer was applied after the surface completely dried.

Pultruded FRP sections are nonmetallic and contain fibers inside the resins. The finished surface of the FRP sections is glazed and polished. There is not any FRP products that are specifically manufactured for SBS technique. Therefore, the commercial FRP products were
pretreated before the bonding applications. The glazed/polished surface of the FRP sections were scratched with chisel tool creating an uneven surface in the resin. The indents were not uniform on the FRP surfaces with a depth ranging from 0 to 2 mm. The dust and any other particles resulting from the process were also removed using acetone to obtain clean surface before bonding.

The final step is to bond these two different but completely clean surfaces. Premixed epoxy is applied on cleaned steel and GFRP surfaces as a thin layer which fills the notches and makes even surface. The pultruded GFRP section is then placed on the bonding zone, and slightly pressured towards the steel web to remove air and gaps between epoxy layers. Excess epoxy that overflowed from between the steel web and GFRP sections was scraped with a spatula and wiped with towel to eliminate artificial epoxy thickening of the web plate out of the stiffening region. Even though there was not a specific epoxy thickness assigned in experimental study, the average measured thickness varied between 2 and 3 mm in all experiments. The obtained epoxy thickness can be considered thick in comparison with other bonding applications. However, in real applications, the initial imperfections can cause similar epoxy thicknesses and variations. The effect of the epoxy thicknesses on the proposed strengthening method was investigated with validated finite element models (see Chapter 4).

1.3 Data Acquisition

National Instruments (NI) cDAQ-9178 was used to transfer the stain and displacement readings from the experiments to the MTS Flex Test SE controller using strain gauges and linear variable differential transformer (LVDT). The details of strain gauge installation and LVDT readings is given in the following sections.

1.3.1 Strain Gauges

General purpose Micro-Measurements (M-M) linear pattern strain gauges were installed at the top and bottom locations of the steel beam flanges in longitudinal directions. Similarly, rectangular rosettes recording strains in three directions were placed on the web surface of CFRP sheets in the alternative CFRP sheet retrofitting method. The maximum number of installed
strain gauges on one steel beam can be 8 gauges using one NI9235 module, and the strain range of each linear gauge was ±5%. The strain data was recorded continuously during the experiments.

The gauge installation is also another important step to obtain reliable results. M-M installation kits were used to clean the steel surfaces and bond gauges to the steel surfaces. Similar to metal surface treatment, local zones on the steel surface where the gauges were placed were abraded using poly abrasive wheel until the white texture obtained. The surface was chemically enhanced for bonding using M-M’s water-based cleaners. Minimum curing time was 1 day before testing the experiments. Finally, gauge terminals were securely soldered to connection cables to the NI cDAQ-9178. One of the installed strain gauges with cable connections can be seen in Figure 1.6.

1.3.2 Displacement Sensors

The vertical displacement readings at the loading point and supports were measured using LVDTs. The voltage change in LVDT is converted to the displacement values using NI9205 module of NI cDAQ-9178 and manufacturer provided calibration coefficient. The possible flexibility in the supports of the testing system and load cell connection at the loading tip were monitored using LVDTs. The support displacements from LVDT readings were proportionally subtracted from the LVDT readings placed at the load-point. The deflection values were then

Figure 1.6 (a) Linear strain gauge at top of the flange (b) rectangular rosette on bonded CFRP sheets
used in experimental calculations and graphs. The placement of a support and loading tip LVDT is shown in Figure 1.7 for one of the experimentally tested beams.

1.4 Main Structure of the Chapters

This section summarizes each chapter. The chapters are organized in journal paper format except the introduction and conclusion chapters. Therefore, it is inevitable that some repetitiveness exists to make each paper as a standalone manuscript. For example, some of the experimental results can be found in several chapters. The introduction chapter outlines the retrofitting application in literature for traditional structures, and method. The specimen specifications and composite specifications are introduced for the proposed retrofitting method. Chapter 1 also describes the data acquisition systems used in experimental tests are presented.

The second chapter provides the experimental results of the effect of initial web panel slenderness in proposed SBS retrofitting method. Different GFRP stiffener configurations were also included in this chapter.

In Chapter 3, experimental results from three different retrofitting method for an identical control beam are presented. The experimental results and findings are compared with traditionally welded steel stiffener and alternative bonded CFRP sheet retrofitting methods.

An investigation of the delamination failure of adhesives using linear elastic fracture mechanic is presented in Chapter 4. The expected failure mixed mode phase angles for two
epoxy types were experimentally obtained conducting single leg bending (SLB) tests and image capturing techniques.

Chapter 5 presents a construction of a full FE model accounting for epoxy delamination and GFRP stiffener degradation failures as well as geometric and material nonlinearities. The FE model was validated by comparing the results of maximum loads and simulating the post buckling global failure behaviors. The parametric studies and fully GFRP stiffener bonded beams instead of steel stiffeners were investigated using the validated FE model.

Finally, the general conclusions are drawn in chapter six. The results of the each chapter are highlighted. Published and possible future publication titles are listed below:

1) Chapter 2 (Published)

2) Chapter 3 (2nd Review)

3) Chapter 4 (Submitted)

4) Chapter 5 (Submitted)

5) Appendix I (Published)
1.5 References


Fernando, D., Yu, T., and Teng, J. (2013). "Behavior of CFRP Laminates Bonded to a Steel Substrate Using a Ductile Adhesive." *Journal of Composites for Construction*, 0(0), 04013040.


Chapter 2. Effect of Initial Panel Slenderness on Efficiency of Strengthening-By-Stiffening using FRP for Shear Deficient Steel Beams†

2.1 Introduction

The high cost of replacing aging structures drives owners to look for more feasible and economical solutions. Therefore, retrofitting existing structures has become the most common and practical solution to enhance the minimum strength and serviceability limits for aging structures. Retrofitting also addresses sustainability by extending the service life of existing structures without the need to invest unavailable larger capital resources for a new structure. Thus, existing structures can be utilized for a longer period, and the need for recycling it is delayed resulting in the use of smaller amounts of materials with lesser carbon footprint (Jones et al. 2013).

Composite materials such as fiber reinforced polymers (FRP) are well suited for retrofitting concrete structures because of their superior mechanical properties relative to traditional construction materials (e.g. concrete) in addition to being light weight and corrosion resistant. In the United States, ACI 440.2R-08 (ACI-440.2R 2008) provides guidance for the design of externally bonded FRP systems for strengthening concrete structures. Several other guidelines and codes have also been published around the world (FIB Bulletin No.14 2001; International Concrete Repair Institute 2006; Japan Building Disaster Prevention Association (JBDPA) 1999; The ISIS Canada Research Center 2004). Strengthening steel structures using externally bonded FRP systems is relatively new when compared to traditional strengthening techniques (i.e. steel plate welding and concrete jacketing) and is lagging behind FRP applications for strengthening concrete structures. Advanced manufacturing technologies of FRP systems allow FRP fibers to be woven within a matrix to form various structural shapes such as I- and T-shapes. FRP structural shapes have the advantage of offering out of plane resistance in addition to the typically utilized in-plane resistance of thin FRP products such as sheets and

† “This chapter previously appeared as [Ulger, T., and Okeil, A. M. (2016). "Effect of initial panel slenderness on efficiency of Strengthening-By-Stiffening using FRP for shear deficient steel beams." Thin-Walled Structures, 105, 147-155]. It is reprinted by permission of Elsevier”
laminates. The out-of-plane resistance of pultruded FRP sections was first used to stiffen thin walled steel beams by Okeil et al. (2009b) as a pilot study, and the ultimate shear resistance increased by 56% when the pultruded GFRP stiffeners were bonded to the web. The strengthening technique whereby pultruded FRP shapes are used as stiffeners to steel plates in thin-walled beams will be referred to as Strengthening-By-Stiffening, or SBS.

In this paper, an experimental program to investigate the effect of web slenderness and shear panel’s aspect ratio on the efficiency of the SBS technique is first described. Three thin-walled steel beams with two different panel aspect ratios and web thicknesses employing the SBS technique were tested to failure. Results from the conducted tests are then presented, and finally, conclusions are drawn based on the findings from the presented results and discussions.

2.2 Literature Review

External bonding of FRP composites is an accepted strengthening technique for concrete structures As is evident by the many successful applications reported in the literature (Bakis 2002; Nanni 1995). In comparison, traditional strengthening techniques (e.g. post tensioning bolting of additional steel plates) still account for the vast majority of the strengthening jobs of steel structures. The same can be said about research in both strengthening arenas as well. A quick search shows that the published work on FRP strengthening of concrete structures is about three times that of steel structures. Therefore, there is a need to fill the knowledge gaps on the use of FRP for strengthening steel structures before any design guidelines can be established, which is the first step towards acceptance and use in field applications.

Increasing the ultimate load carrying capacity is mainly the primary objective for strengthening applications, which is often accompanied by loss of ductility (Okeil et al. 2009b). A more ductile performance was observed when GFRP (Glass FRP) or KFRP (Kevlar FRP) were used for strengthening reinforced concrete structures (Triantafillou et al. 1992), however, the required amount of FRP is typically larger than the amount of CFRP (Carbon FRP) (Sen et al. 2001; Triantafillou et al. 1992). Published work shows that the most widely used composite material for strengthening steel structures is the CFRP sheet/strips with some efforts.
recommending high or ultra-high modulus CFRP for strengthening steel structures (Harries et al. 2011; Schnerch 2007). This is due to the higher elastic modulus of CFRP as opposed to other types of composites (e.g. GFRP) makes it more compatible with the mechanical properties of steel. For example, the flexural strengthening of the steel sections were studied experimentally and numerically utilizing different forms and layers of CFRP composites on the tension side of the steel girders (Al-Saidy et al. 2007; Galal et al. 2012; Kim et al. 2012; Miller 2001; Sen et al. 2001). In addition to the flexural strengthening efforts, researchers also investigated the feasibility of using composite materials to strengthen steel structures subjected to axial and shear forces. Different steel joint types (e.g. K and V) subjected to the axial forces were also strengthened by wrapping different number of CFRP composite layers (Fam et al. 2006; Gao et al. 2013; Xiao et al. 2012). A limited number of experiments investigating shear strengthening of steel structures using composites were conducted bonding different form CFRP composites in different configurations (Narmashiri et al. 2010; Okuyama 2012).

The concept in these conventional techniques is the utilization of in-plane resistance of an external reinforcing material, which quickly revealed that the efficiency is less than that observed in strengthening concrete structures due to the large amount of FRP needed for strengthening steel structures (Fam et al. 2006; Sen et al. 2001). In the proposed SBS technique, a different form of composite materials; pultruded FRP sections, is utilized in an innovative way resulting in a practical strengthening technique while reducing the amount of FRP usage.

2.3 Proposed Strengthening Method

The main concept behind the proposed SBS technique is the utilization of pultruded FRP sections to enhance the capacity of shear deficient thin-walled steel structures. Figure 2.1 shows an illustration of how a wide-flanged pultruded FRP section can be bonded to a thin, buckling-prone steel plate. The enhancement in shear resistance is caused mainly by delaying buckling of the steel plate as a result of the additional out-of-plane stiffness provided by the pultruded FRP section. Therefore, this stiffening method allows using cheaper, low-modulus fibers within the matrix resins of the composite section to strengthen steel structures whose elastic modulus is
inherently higher. SBS success stems from the fact that the flexural rigidity, $EI$, of the additional stiffener is an order of magnitude higher than that of the deficient steel plate. The first preliminary study on using the SBS technique was conducted by Okeil et al. (2009a), and more technical details can be found elsewhere (Okeil et al. 2009b).

2.4 Experimental Program

An experimental program was designed to study the effect of initial web slenderness on the efficiency of SBS in enhancing the shear strength of thin-walled steel beams. The program consisted of eight beam specimens with different web thicknesses, shear panel dimensions, and FRP stiffener configuration. Varying the web thickness directly affects the web slenderness, and the shear panel’s dimensions directly affect the compression field and hence the beam’s shear strength. Finally, the FRP stiffener configuration determines the additional stiffness provided by the pultruded FRP sections.

The following sections describe the specimens and the experimental setup in detail.

2.4.1 Beam Specimens

The tested specimens were first designed to be shear critical by overdesigning for other modes of failure (e.g. flexure, lateral torsional buckling). Finite element models were built to assist in the design process and in predicting modes of failure and loads. The final design was
then especially fabricated for the project as a built-up I–shaped section using A36 steel. The mechanical properties of the steel sections were obtained from uniaxial coupons and are given in Table 2.1. The two nominal web thicknesses considered in these tests were 3.2 mm [1/8 in.] and 4.0 mm [5/32 in.]. While the choice for such thin webs was mainly due to maximum applied load limitations in the available structural testing facilities, it was also justifiable for the goals of the project for two reasons. First, shear deficiency may be due to uniform corrosion of the web plate, which often leads to a reduction in the thickness of structurally sound plates. The other reason is the potential of using SBS as a mean for optimizing thin-walled steel sections by reducing steel plate thickness and complementing its stiffness by bonding pultruded FRP sections during the fabrication process in lieu of welding steel stiffeners and its associated disadvantages. Square and rectangular shear panels were considered. The dimensions of the square panel were 518 x 521 mm [20\(\frac{3}{8}\) x 20\(\frac{3}{8}\) in.] resulting in a nominal aspect ratio equal to 1.0. The rectangular panel specimens were of the same depth, but wider (772 mm [30\(\frac{3}{8}\) in.]) leading to a nominal aspect ratio of 1.5. The T-shaped FRP stiffeners were obtained from commercially available wide-flanged I-shaped sections [6x6x3/8 in.] by cutting one of the two flanges. The remaining flange would serve as the bonding surface with the steel plate. Length-wise, the FRP stiffeners were cut to 482.6 mm [19 in.] long pieces, which is short of covering the entire depth of the web to avoid the weld seam between the flange and the web. The width of the bonding flange was reduced to 76 mm [3 in.] for square panel specimens. Control beams, i.e. without any strengthening FRP stiffeners, were also tested.

<table>
<thead>
<tr>
<th>Table 2.1 Mechanical properties of steel material</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
</tr>
<tr>
<td>Square Panel Beams</td>
</tr>
<tr>
<td>Rectangular Panel Beams</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Square Panel Beams</td>
</tr>
<tr>
<td>Rectangular Panel Beams</td>
</tr>
<tr>
<td></td>
</tr>
</tbody>
</table>
The following designation system was used for easier reference to the specimens. The first two characters indicated whether the specimen had square shear panels (SB) or rectangular ones (RB) corresponding to the 1.0 and 1.5 panel aspect ratios, respectively. This was followed by a fraction representing the nominal web thickness in inches in square parenthesis; i.e. [1/8] and [5/32]. The last part of each specimen designation described the FRP stiffener configuration. A ‘0’ indicated that no stiffeners were used for this specimen; i.e. control, ‘1’ indicates that only one stiffener was bonded to one side of the web, and finally ‘2’ indicates that two stiffeners were bonded to the shear panel; one on each side. For example, SB[1/8]-2 is a square panel specimen with a 3.2 mm-thick [1/8 in.-thick] web strengthened with two FRP stiffeners, whereas RB[5/32]-0 is the control for rectangular panel configurations with 4.0 mm-thick [5/32 in.-thick] web specimens.

The test matrix of the tested beam can be seen in Table 2.2. The table also lists the slenderness of the beams’ webs, which is defined as a ratio of the web height, $h$, to the web thickness, $t_w$.

2.5 Material Properties

2.5.1 Pultruded GFRP Section

The proposed stiffening technique enhances the strength of the thin walled steel structure significantly by using low modulus pultruded GFRP composites (Okeil et al. 2009a). The same stiffening technique can be employed using high modulus pultruded FRP sections (e.g. CFRP);

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Nominal Web Thickness (mm [in.])</th>
<th>Panel aspect ratio</th>
<th>Width of FRP flange (mm[in])</th>
<th>No. of FRP stiffeners</th>
<th>Slenderness ($h/t_w$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SB[1/8]</td>
<td>0 1/2</td>
<td>3.2 [1/8]</td>
<td>1.0</td>
<td>76.2 [3.0]</td>
<td>152.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RB[1/8]</td>
<td>0 1/2</td>
<td>3.2 [1/8]</td>
<td>1.5</td>
<td>152.4 [6.0]</td>
<td>152.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RB[5/32]</td>
<td>0 2</td>
<td>4.0 [5/32]</td>
<td>1.5</td>
<td>152.4 [6.0]</td>
<td>121.6</td>
</tr>
</tbody>
</table>
however, the use of high modulus FRPs is not necessary for this strengthening technique because all failure modes were observed to occur at the bonding interface in the preliminary investigations Okeil et al. (2009b). The pultruded FRP sections are mainly utilized to enhance the out-of-plane resistance of deficient plates in buckling prone regions. Even though the elastic modulus of GFRP is lower than that of steel, the flexural rigidity, EI, is higher due to the additional stiffness provided by the pultruded GFRP sections.

The pultruded GFRP sections used in this study are commercially available in wide flange beam forms. (EXTERN® 500, and PROForm® WF used for square and rectangular panel beams, respectively). As stated earlier, the chosen 6x6x3/8 wide flange pultruded FRP section was modified to obtain the T-shaped stiffener by cutting one of the flanges. The mechanical properties of pultruded GFRP are listed in Table 2.3.

2.5.2 Adhesive Type and Properties

Only one adhesive type was used for this study, which is a general purpose bonding agent that is commonly used in strengthening applications of concrete structures for bonding external composite materials. Even though the nature of SBS demands on the adhesive interface between the steel plate and the FRP stiffener are different than the demands in typical strengthening applications, the same adhesive was still chosen because it was shown that it is still effective for SBS applications (Okeil et al. 2009a). The adhesive (Tyfo® S – Fyfe Co) is a two-component chemical that is mixed at a specific ratio as per the manufacturer’s instructions. Fumed silica was added to the resulting mix and stirred for at least 5 minutes to achieve uniform consistency and a workable viscosity. The mechanical properties of the selected adhesives are given in Table 2.4.

<table>
<thead>
<tr>
<th>FRP</th>
<th>$E_f$, GPa [ksi]</th>
<th>$\sigma_u$, MPa [ksi]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Extern</td>
<td>$\mu$ (mean)</td>
<td>13.20 [1915]</td>
</tr>
<tr>
<td></td>
<td>$\sigma$ (SD)</td>
<td>1.85 [268]</td>
</tr>
<tr>
<td></td>
<td>$C_V$ (%)</td>
<td>14.0</td>
</tr>
<tr>
<td>PROForm</td>
<td>$\mu$ (mean)</td>
<td>24.4 [3536]</td>
</tr>
<tr>
<td></td>
<td>$\sigma$ (SD)</td>
<td>1.5 [216]</td>
</tr>
<tr>
<td></td>
<td>$C_V$ (%)</td>
<td>6.1</td>
</tr>
</tbody>
</table>

Table 2.3 Flexural properties of FRP specimens
It should be noted that other adhesives that are more suitable for bonding composites to metals can be used. Okeil A. (2014) compared the performance of two types of adhesives in a recent study, where it was revealed that adhesive with ductile properties perform better in steel strengthening applications.

2.6 Specimen Preparations

Several surface preparation techniques may be required such as prevention of the galvanic corrosion between the steel/FRP surfaces if different types of FRP are in consideration. Since debonding has been found to be the critical mode of failure in previous investigations (Okeil et al. 2009a), the bonding procedure requires utmost care. Furthermore, it is reported that the short and long term durability of the bond is increased with the proper pretreatment of the substrates (Baldan 2004). In general, FRP surface treatment requires less effort than the steel surface treatment (Harries et al. 2012). The FRP’s glazed finish was removed by scratching the outer matrix layer to create a rougher surface that is more suitable for bonding. All residual particles resulting from this step were cleaned using a solvent (i.e. acetone) before the adhesive was applied. The other bond side; i.e. steel plate surface, could be treated with a chemical etchants to reach the higher durability level; however, the high curing temperature and the environmental hazard of many etchants limits the application of this treatment (Wegman et al. 2012). Instead, another feasible and effective method for steel surfaces was employed. Mechanical surface treatment (i.e. grit blasting or sanding) creates a roughened steel surface, into which the adhesive penetrate and creates a mechanical bond (Baldan 2004) in addition to the chemical adhesion. In this study, mechanical surface treatment was used to prepare the steel surfaces in three steps: (1) removing the contaminants from the steel surface, (2) sanding the

<table>
<thead>
<tr>
<th>Tyfo® S</th>
<th>Rate of Loading 1.27 mm/min [0.05 in/min]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$E_a$, MPa [ksi]</td>
</tr>
<tr>
<td>$\mu$ (mean)</td>
<td>2575 [373.5]</td>
</tr>
<tr>
<td>$\sigma$ (SD)</td>
<td>202 [29.32]</td>
</tr>
<tr>
<td>$CV$ (%)</td>
<td>7.8</td>
</tr>
</tbody>
</table>
steel surface by poly abrasive wheel and sanding papers until a white texture appears (Schnerch 2007), (3) wiping the revealed white metal surface with a cleaning solvent right before the bonding process.

After the steel and FRP surfaces were completely freed from any contaminants, rust or particles with a proper surface treatment, the adhesive was applied to the clean and roughened FRP and steel surfaces. A small amount of pressure was applied on the FRP section until the adhesive could be seen filling the entire interfacial bond area between the two materials. The goal was to have a thin uniform adhesive layer (about 2 mm-thick [5/64 in.-thick) to avoid the brittle failure (Harries et al. 2012). The leftover adhesive was scraped from the steel web to avoid increasing the strength of the steel plate inadvertently by increasing the plate’s thickness when the leftover adhesive hardens.

2.7 Experimental Setup

All beam specimens were tested in three-point bending. The load was applied over the first internal stiffener to create high shear demand on the critical test panel. In Figure 2.2 (a) and (b), typical beam specimen showing the main dimensions and location of the applied load are provided. As stated earlier, this configuration ensures that the expected failure mode will be shear buckling of the first panel. It should be noted that due to laboratory difficulties, Specimen SB[1/8]-1 was tested with a shorter span length equal to the length of 3 square panels ($L=1654$ mm [65.13 in]) versus the typical one for SB specimens of 4 panels ($L=2172$ mm [85.50 in]).

Early analytical and experimental results verified that buckling occurs within the first stiffened panel (Okeil et al. 2010). Diagonal and vertical FRP stiffener orientations were considered and compared in these pilot studies, and the vertical FRP orientation was found to perform in a more ductile manner than diagonal FRP orientation (Okeil et al. 2009a). Therefore, the pultruded FRP stiffeners were bonded vertically in the middle of the critical panel. Control specimens were tested without any external FRP stiffener. The strains in the stiffened beam specimens were recorded at the applied load location where the maximum moment occurs. A
total of four strain gages were attached to the beam specimen at the section where the load was applied. Two strain gauges were placed at each of the top and bottom flanges (top and bottom surface of each flange). The locations of the strain gages can be seen in Figure 2.2 (b) and Figure 2.3, which shows two cross sections for the control specimen and for a beam strengthened with stiffeners on both sides; i.e., with and without SBS. The choice of the strain gage location is based on previous experience to capture sway-frame action after shear panel buckling (Okeil et al. 2009a). In addition to strains, the data acquisition system recorded readings from a load cell that measures the applied load and an LVDT displacement at the loading location.

Figure 2.2 Typical (a) square and (b) rectangular beam specimen and test set up configurations
2.8 Results and Discussion

The experimental results are presented in this section for the tested specimens. Load-deflection curves from recorded LVDT and load cell readings will be first presented. Strain readings at the section under the load will then be discussed with emphasis on shear panel buckling identification and the distinct behavioral shift pre- and post-buckling.

2.8.1 Load-deflection Curves

The load-deflection curves for all square panel specimens SB[1/8] are plotted in Figure 2.4. The three shown curves are for the control SB [1/8]-0 and strengthened specimens, with one stiffener SB[1/8]-1 and two stiffeners SB[1/8]-2. The flange width of the FRP T-shaped stiffener was 76.2 mm-wide [3 in.-wide.] for the strengthened specimens in this group. The ultimate load capacities of SB[1/8] beams are given in Table 2.5 for the stiffened and unstiffened cases, and it can be seen that load capacities increased 30% and 34% for one and two stiffener cases, respectively. The yield load (identified as the end of the proportional limit) also increased with the number stiffeners which can be seen in Figure 2.4. Furthermore, it can be seen that the maximum load was reached in a gradual increase after yielding for the specimen with one stiffener, while a clear plateau was observed for the beam with two stiffeners. This behavior may be attributed to the following two reasons. First, the adhesive layer is subjected to different stress
Figure 2.4 Load vs deflection plot of SB[1/8]-0, SB[1/8]-1, SB[1/8]-2 beams

states on opposite sides of the web. If the buckling wave causes the web to bulge such that the adhesive layer on one side is subjected to tension as the FRP stiffener is forced to separate from the steel web, the adhesive on the opposite side will be under compression between the FRP and steel surfaces. This behavior is illustrated in Figure 2.5. It is known that the compression resistance of the adhesives is higher than its tension and shear resistance (Fiedler 2001; Littell 2008), and the tensile stresses cause the failure of the adhesives while the shear stresses cause excessive deformation without failure (Fiedler 2001). Hence, even if the adhesive interface gets damaged on the tension side of a beam with two opposite stiffeners, the opposite side would still be intact and keep the panel stiffness. Laboratory observations confirmed that the adhesive failed locally on tension side, while the adhesive on the compression side remained intact. Figure 2.4 also shows a load drop at higher load levels for the one stiffener case, which confirms that localized adhesive failure takes place at higher load levels. It can be seen that the load drop for the one stiffener configuration is not as pronounced for the two opposite stiffener specimen. The
second reason for having a clear plateau for the specimen with two opposite stiffeners is that the additional stiffness provided by the FRP stiffener and adhesive layer on both sides of the web panel, which ensures that the panel’s behavior will be closer to a split panel than in the case of one-sided stiffener. Thus allowing for more plasticization of the less slender web plate.

The load deflection behavior for the rectangular panel beam specimens with 3.2 mm [1/8 in] web thickness, RB[1/8] is similar to that for SB[1/8] beam specimens as can be seen in Figure 2.6. The FRP stiffener’s flange width for this group was 152 mm [6 in.] and the panel aspect ratio was 1:1.5. The ultimate failure load for the stiffened specimens increased by 30% and 36% for the RB[1/8]-1 and of RB[1/8]-2 cases, respectively. The third group of specimens with

<table>
<thead>
<tr>
<th>Beam Labels</th>
<th>Failure Load kN[kips]</th>
<th>Capacity Increase</th>
<th>Initial Stiffness (k/in) $K_i=P/\Delta$</th>
<th>Stiffness Increase</th>
</tr>
</thead>
<tbody>
<tr>
<td>SB[1/8]</td>
<td>0 240[54.0]</td>
<td>--</td>
<td>377</td>
<td>--</td>
</tr>
<tr>
<td></td>
<td>1 311[7.0]</td>
<td>30%</td>
<td>428</td>
<td>14%</td>
</tr>
<tr>
<td></td>
<td>2 322[7.5]</td>
<td>34%</td>
<td>413</td>
<td>10%</td>
</tr>
<tr>
<td>RB[1/8]</td>
<td>0 222[50.0]</td>
<td>--</td>
<td>301</td>
<td>--</td>
</tr>
<tr>
<td></td>
<td>1 289[65.0]</td>
<td>30%</td>
<td>384</td>
<td>28%</td>
</tr>
<tr>
<td></td>
<td>2 302[68.0]</td>
<td>36%</td>
<td>424</td>
<td>41%</td>
</tr>
<tr>
<td>RB[5/32]</td>
<td>0 294[66.0]</td>
<td>--</td>
<td>431</td>
<td>--</td>
</tr>
<tr>
<td></td>
<td>2 411[9.25]</td>
<td>40%</td>
<td>452</td>
<td>5%</td>
</tr>
</tbody>
</table>

Figure 2.5 FRP stiffener failure mechanism at adhesive level (not scaled sketch)
thicker web (4.0 mm [5/32 in.]) exhibited a 40% load capacity increase as a result of SBS. As can be seen in Figure 2.7, the RB[5/32]-2 specimen resistance to the applied load did not drop suddenly indicating that a major loss of bond between the FRP stiffeners and the steel web never took place. The test procedure was stopped when the stroke of the test machine reached the designated deflection limit. A similar limit was also set for Specimens SB[1/8]-2 and RB[1/8]-2. Figure 2.8 shows the critical panel for Specimen RB[1/8]-1 after failure. One can see that after complete debonding of FRP section, the web buckled in a single wave along the tension field of the critical panel.

2.8.2 Initial Global Stiffness

In addition to enhancing the load capacity, SBS can also enhance the global stiffness of strengthened beams. The initial stiffness value, $K_i$, is defined as an initial slope of load-deflection curve. The $K_i$ values of the tested beams increased after the introduction of SBS and stiffnesses for specimens with two stiffeners were higher than those with only one stiffener. As expected, the increase in stiffness for the SB[1/8]-1 specimen was more than the stiffness increase of SB[1/8]-2 as a result of the aforementioned different span lengths at which this group was tested.
Therefore, the 4% difference between the initial stiffnesses of SB\[1/8\]-1 and SB\[1/8\]-2 does not reflect the proportional increase as the number of stiffener increases. The effect of using two opposite stiffeners is much clearer for the RB\[1/8\] specimens. Specimens RB\[1/8\]-1 and RB\[1/8\]-2 show 28% and 41% increase in initial stiffness, respectively, compared to the control specimen RB\[1/8\]-0. Table 2.5 lists initial stiffness values for all tested cases. In general, it is clear that the effectiveness of the proposed strengthening technique in terms of initial stiffness is more pronounced for beams with higher initial slenderness ratios as they benefit more from the introduction of SBS.

Figure 2.7 Load vs deflection plot of RB[5/32]-0, RB[5/32]-2 beams

Figure 2.8 Failure mode for specimen RB[1/8]-1 (a) front - (b) back
2.8.3 Ductility

With the exception of few applications (Idris et al. 2014), the use of composite materials to strengthen concrete and steel structures in flexure is known to reduce the ductility of strengthened beams (ACI-440.2R 2008). Even though the proposed failure mode for shear deficient steel structures strengthened using SBS is typically sudden when the debonding occurs at the interface between the GFRP and steel surfaces, a ductile behavior was observed before failure in previous studies (Okeil et al. 2009b). The tested beams showed substantial ductility beyond the yield point up to the debonding of the FRP stiffener, which was usually accompanied by a major load drop. Cracking of the epoxy layers, which sounds like glass shattering, was clearly heard starting around the initiation of yielding. The intensity of the cracking sounds increased as the test progressed further until the entire FRP stiffener debonded. After the tests were completed, the internal epoxy cracks under the debonded GFRP stiffener could be clearly seen. Another possible type of failure was the adhesion failure either at steel or GFRP surfaces. Therefore, both adhesion and cohesion failures should be investigated for SBS applications.

2.8.4 Strain Readings

As stated earlier, the tested beams were designed with slender webs to create an elastic buckling mode of failure as per the AASHTO LRFD bridge design specification (AASHTO 2012). Two beams, SB[1/8]-1 and RB[1/8]-2, were instrumented with strain gauges to measure longitudinal strains in the top and bottom flanges of the steel beams at the loaded section. Table 2.6 lists the strain readings at the top and bottom surfaces of the top and bottom flanges. The maximum strain readings at the time of web buckling were 0.0563% and 0.0604% on the bottom surface of the tension flanges for SB[1/8]-1 and RB[1/8]-2, respectively. It can be seen from the table:

<table>
<thead>
<tr>
<th>Beam</th>
<th>Top Flange</th>
<th>Bottom Flange</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top Strain</td>
<td>Bottom Strain</td>
</tr>
<tr>
<td>SB[1/8]-1</td>
<td>-0.0296%</td>
<td>-0.0307%</td>
</tr>
<tr>
<td>RB[1/8]-2</td>
<td>-0.0377%</td>
<td>-0.0115%</td>
</tr>
</tbody>
</table>
these values that the steel flanges were under elastic stress levels when web buckling initiated. Up to this point, the applied load is transferred through the web panel mainly through a tension field tie.

Strain readings of the rectangular panel beams are plotted in Figure 2.9 and Figure 2.10 for one and two stiffeners cases, respectively, and the readings followed a similar trend for both beams. As expected, the top flange gages initially recorded compressive strains and the bottom flange gages recorded tensile strains. The readings start in an almost linear trend with small differences between the top and bottom readings for each flange. This behavior continues until the applied load reached a level that caused buckling initiation. Once the beams were loaded beyond that point, buckling ensues and the strain readings for each flange (top and bottom) start changing in different directions. This is an indication of local bending of each flange. This post buckling behavior is analogous to a sway frame mechanism where plastic joints form on the local members. Figure 2.11 shows a schematic of the sway mechanism and local plastic moments at the top and bottom flanges for the tested beams. This is caused by the fact that the beams’ resistance to the applied force starts shifting from the tension field tie mechanism to the sway

![Figure 2.9 Strain readings of SB[1/8]-1 at the top and bottom flanges](image-url)
frame mechanism as the web buckling progresses. Quantifying the portion of the load resisted by each mechanism experimentally is challenging. Therefore, the authors are currently developing finite element models that can be used for this purpose.

![Figure 2.10 Strain readings of RB[1/8]-2 at the top and bottom flanges](image)

2.9 Conclusions

One square and two rectangular panel beams with two different web slenderness values and web thicknesses were experimentally tested with and without FRP stiffeners. The steel webs of the beams were stiffened by externally bonding vertical pultruded FRP sections in two configurations; one and two stiffeners (one on each side). The proposed strengthening technique, Strengthening-By-Stiffening or SBS, utilizes the out-of-plane resistance of cheaper pultruded FRP sections in contrast to conventional FRP strengthening techniques that rely only in-plane resistance of relatively expensive composite fibers. In the current study, one adhesive type was used for external bonding, and all failure mechanisms were developed at the bond joint. Built-up I-shaped steel beams were tested monotonically under three-point loading configuration, where the load was applied on the first internal transverse stiffener.

Based on the experimental results, the following can be concluded:
1. The proposed SBS method enhances the ultimate failure capacity of the controlled steel beams by up to 40%. The maximum percentage increase in shear resistance happened when two FRP stiffeners were bonded to the web; one on each side.

2. The square and rectangular panels that have 3.2mm [1/8in] steel web thicknesses did not experience sudden load drops during the experiments when stiffened with two stiffeners. One stiffener scheme of these beams did not reach the two stiffeners’ strength level or almost reached that level but did not maintain this load level for long after buckling. Therefore, two-stiffener strengthening sustain the post yielding behavior of all the stiffened beams. The main reason for this behavior can be attributed to the fact that the resistance of epoxy layers facing each other is different under the tension and compression forces, hence, they complement each other.

3. SBS caused the initial global stiffness of the tested steel beams to increase for all tested specimens. Other than the fact that introducing additional stiffness increases the initial global stiffness, a clear correlation between the amount of increase in initial stiffness and the FRP stiffener configuration could not be established.

4. Strain readings showed that the load path from the point of application to the supports started in a classical tension field tie manner. Once the web panel buckling
initiated, the load transfer shifted to a sway frame mechanism causing local plasticization of the flanges.

In summary, the proposed SBS technique shows promise as an alternative strengthening technique for steel structures that does not require on site welding or bolting. It is capable of achieving substantial capacity gains as shown in this paper. It can be said that SBS is a feasible alternative for strengthening steel beams that may not be initially deficient, but whose slenderness increases over time due to environmental effects, which is especially true for built-up sections typically used in bridge construction whose webs are relatively more slender than hot-rolled sections. SBS also has the potential of being introduced in the design of new steel beams to achieve lighter sections with thinner webs that are stiffened with FRP stiffeners without the need or welding or bolting.

The presented results warrant further investigations of SBS with special focus on better, more ductile adhesives that are more suitable for steel applications. This can be considered an optimization problem where effective parameters, such as bonding area, out of plane resistance of stiffeners, environmental effects and fatigue life, could be studied to establish a full understanding of the behavior of this new strengthening technique. Another area of future research would be to establish practical design methodologies for SBS.

### 2.10 Nomenclature

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_v$</td>
<td>Coefficient of variation</td>
</tr>
<tr>
<td>$E_a$</td>
<td>Axial Modulus of Elasticity</td>
</tr>
<tr>
<td>$E_f$</td>
<td>Flexural Modulus of Elasticity</td>
</tr>
<tr>
<td>$h$</td>
<td>Height of steel web</td>
</tr>
<tr>
<td>$K_i$</td>
<td>Initial stiffness</td>
</tr>
<tr>
<td>$L$</td>
<td>Length of the beam</td>
</tr>
<tr>
<td>$P$</td>
<td>Applied load on the control panel</td>
</tr>
<tr>
<td>$t_w$</td>
<td>Thickness of steel web</td>
</tr>
<tr>
<td>$\Delta$</td>
<td>Vertical displacement at the load line</td>
</tr>
</tbody>
</table>
\( \varepsilon_u \)  Ultimate strain

\( \mu \)  Sample mean

\( \sigma \)  Standard deviation of a sample

\( \sigma_u \)  Ultimate strength

\( \sigma_y \)  Yield strength

2.11  References


The ISIS Canada Research Center (2004). "FRP Rehabiitaion of Reinforced Concrete Structures." *Design Manual No. 4Quebec, Canada*.


Chapter 3. Strengthening-By-Stiffening: FRP Configuration Effects on Behavior of Shear-Deficient Steel Beams

3.1 Introduction

Restoring the original capacity of structures or upgrading it using externally bonded fiber reinforced polymer (FRP) composites is a feasible alternative that has become appealing because it satisfies engineering, operational and economical demands. Concrete and masonry structures have a major share in composite retrofitting. Retrofitting steel structures using composite materials has also gained attention but it is still relatively lagging compared to the other materials. FRP composites have been used in flexural, shear and axial strengthening of concrete structures. Most of strengthening application utilize FRP composites in form of laminate, strip or sheet plates with relatively small thickness and negligible out of plane stiffness. Therefore, the uniaxial strength of composite fibers is the major contribution to enhance the capacity of existing structures. FRP bonding to the tension flange of a girder, or confining a concrete column are the typical examples of the conventional use of composites. Implementing the same technique to restore a steel section’s capacity requires large amounts of FRP composites due to inherent mechanical properties of steel material (Sen et al. 2001). Alternatively, some researchers proposed the use of ultra-high modulus FRP composites (Schnerch 2007). The origin of composite fibers used in restorations are mainly glass (GFRP) and carbon (CFRP) composites, and CFRP are the most preferable composite material due to higher tensile resistance in retrofitting applications (ACI-440.2R 2008).

The researchers developed a new strengthening technique to overcome the aforementioned difficulties faced when composite materials are used in conjunction with steel by proposing the Strengthening-by-Stiffening or SBS method. In SBS, the steel section is strengthened by bonding pultruded FRP sections (Okeil et al. 2009a) to the vulnerable region. This method provides additional bending stiffness due to the geometric properties of the pultruded FRP section, which enhances the buckling resistance of thin plates, and hence the higher strength of the structure. The contribution of the pultruded FRP stiffener to the plates
overall stiffness is shown in Figure 3.1. Another benefit is that since high tensile strength of FRP composites is not the primary contribution of the pultruded FRP section, CFRP composite can be replaced with relatively cheaper composite materials such as GFRP composites.

An experimental program was devised to investigate the efficiency of the proposed SBS technique under different FRP configurations. Results from one and two FRP stiffener configurations; i.e., one FRP on one side and one FRP on each side, in a rectangular panel beam are presented. In addition to the proposed SBS method, two different retrofitting alternatives for shear regions were investigated. One alternative of the shear strengthening methods of the web plates was the conventionally welded transverse steel stiffeners which is a typical strengthening technique of steel structures that involves on site welding. The same size steel stiffener used in the original unstrengthened beam was welded at the same location where the FRP pultruded stiffeners were bonded on each side of the web panel. The steel stiffeners were fully welded to the flanges and web plates. Even though welded steel stiffeners have substantial share in field applications, the stress concentrations due to welding and the related fatigue issues, difficulties in handling heavy steel plates and welding equipment, are some disadvantages of using welded steel sections as a strengthening technique, which may lead to service interruptions during strengthening.

The second alternative method considered for strengthening shear deficient steel beams relies on bonding FRP sheets or laminates in buckling prone regions. A biaxial (0°/90°) CFRP sheet product was used for one specimen. The sheet was bonded to the entire web panel area on both sides of the web plate. The choice of a biaxial sheet is dictated by the fact that shear regions are subjected to a multidirectional stress state.

3.2 Literature Review

Structural retrofitting using adhesively bonded composite materials are well established for masonry and concrete structures applications. Material properties of composite materials yielded successful capacity increase and durability in experimental studies and field applications. Therefore, standardized design specifications and guidelines have been published and are
currently used in practice all over the world (ACI Committee 440 2007; FIB Bulletin No.14 2001; International Concrete Repair Institute 2006; Japan Building Disaster Prevention Association (JBDPA) 1999; The ISIS Canada Research Center 2004). Retrofitting steel structures using composite materials, however, is relatively lagging. The main hurdle to massive utilization of composite materials in steel structure retrofitting applications is the inherent mechanical properties of steel, which are closer to composite material properties than concrete or masonry. As such, large amounts of composite materials are needed to achieve similar capacity enhancements as those reported for concrete structures (Sen et al. 2001; Triantafillou et al. 1992). Alternatively, some studies showed that the use of high and ultra-high modulus (HM and UHM) composite materials can improve the efficiency of this technique by reducing the required amounts of composite materials for retrofitting steel structures (Harries et al. 2011; Schn erch 2007). Consequently, the most common composite materials used in strengthening applications of steel structures are CFRP, high modulus CFRP (HM-CFRP) and ultra-high modulus CFRP (UHM-CFRP) because they are the most compatible with steel due to their higher elastic
modulus despite the susceptibility of the system (steel-CFRP or aluminum-CFRP) to galvanic corrosion which is often addressed by applying putty or non-corrosive composite materials at interface layer (i.e. GFRP) (Fam et al. 2006; Gao et al. 2013).

The most common forms of FRP composites used in retrofitting are the uniaxial or biaxial sheets, laminates, and strips. The thin FRP layers, or fibers within a layer, can be arranged in different orientations within the planar section of the FRP product; such as, 0°, 45°, or 90°. The majority of strengthening applications utilizes the uniaxial strength of fibers in the composite in the plane of the composite elements. Therefore, in literature, the out of plane resistance of the planar composites are irrelevant for retrofitting masonry, concrete, and even steel structures, even though the latter is more susceptible to local buckling, which is not a major issue for concrete or masonry.

The literature shows that a large portion of steel retrofitting studies emulated strengthening techniques for concrete structures where thin planar CFRP composites were bonded to deficient members to enhance its flexural or axial strength. Sen et al. (2001) investigated the behavior of steel-concrete composite beams that were retrofitted with CFRP strips in tension flange of the steel girders after being loaded up to levels that caused the tension flange to reach its yield point; thus simulating harsh service conditions. Different numbers of CFRP layers were used to increase flexural capacity of the beams, which resulted in an average of 9 to 52% capacity increase. Al-Saidy et al. (2007) also tested flexural strengthening of steel beams with different numbers and elastic modulus values of CFRP plates. CFRP plates were bonded to the tension flange and both sides of lower part of the web in their experimental program. Four sets of steel girders were obtained from an existing bridge after being exposed to severe environmental conditions. Flexural strengthening by bonding pultruded CFRP plates to top and bottom surfaces of tension flange was investigated, and then the concept was extended to a girder from a bridge in service. The results showed that the global stiffness of that girder increased 11% from preretrofitted condition (Miller 2001). The same approach was applied to a group of artificially deteriorated RHS steel beams where hybrid composite systems were bonded
to the tension flange. Ultra-high modulus, or high modulus of CFRP composites, were
sandwiched between either flat or U shaped GFRP composites to obtain the hybrid composite
system (Photiou et al. 2006). In another study, flexural and shear strengthening of steel beams by
bonding CFRP laminates was studied. A strength increase of 15 and 26% was reported for
retrofitted tension flange and web sections, respectively (Patnaik et al. 2008). Different types of
bonded materials with different number of CFRP sheet/plate layers were investigated (Galal et
al. 2012) for flexural strengthening of the steel beams. In addition to the bonded CFRP
composites, the tension flange of the same type beams was strengthened by anchoring the CFRP
composites at both flange ends as an alternative to adhesive bonding. However, there was not
obvious flexural strength gain reported with only anchored CFRP system due to premature
rupture of CFRP (Galal et al. 2012). Steel beams that were artificially notched in tension flange
were retrofitted by bonding CFRP strips, and were experimentally tested (Kim et al. 2012). The
result showed that full load capacities were restored to pre-notched condition. Similarly, HM-
CFRP sheets were applied by fully wrapping damaged and undamaged rectangular and square
hollow steel sections to increase the flexural and bearing capacities (Elchalakani 2014). The
sectional properties and the number of CFRP layers were varied in the experiments. The test
results showed that the most flexural and bearing strength increase was reported for the most
slender section. The use of HM-CFRP materials for steel retrofitting applications was suggested
by some researchers to reach higher load and energy absorption capacities (Elchalakani 2014; Kim et al. 2012).

Another common use of composites is the retrofitting of axially loaded members or joints
connecting a group of axial members. In an example of a retrofitted axial member, circular
tubular steel braces including initial imperfections were wrapped with different number of CFRP
layers [12]. Axial strength of the retrofitted specimens was increased from 28% to 124% with 2.4
mm initial imperfection, and from 25% to 105% with 4.8 mm initial imperfection as the numbers
of CFRP layers were increased gradually. In an example of a retrofitted joint, K-shaped
aluminum joints were retrofitted by wrapping CFRP and GFRP sheets (Fam et al. 2006). The full
capacity of the K joints was artificially reduced by 10%. Full recovery was achieved with use of CFRP sheets; however, the same number of GFRP sheets was not adequate for full recovery. Similarly, retrofitting cracked T-joints was investigated using two different wrapping schemes (Xiao et al. 2012). In the first scheme, CFRP layers were wrapped around rectangular and square hallow sections. In second scheme, T-steel plates were placed between CFRP layers. Results showed that the fatigue performance and flexural stiffness of the CFRP wrap with T-plates were considerably higher than that of CFRP wrap only.

The literature shows that one of the less studied FRP retrofitting applications for steel structures is for enhancing the resistance of shear deficient steel members. The few typical shear strengthening applications reported in the literature involve bonding planar CFRP composites to the web of the sections. The ultimate load capacity of the built up I sections were increased by bonding CFRP sheets on the web plate. Uniaxial and biaxial fiber sheets were bonded on both sides of the rectangular and square web panels. An increase in load capacity of 29 and 12% was reported by Okuyama et al. (Okuyama 2012) for the rectangular and square panel beams, respectively. In another study by Narmashiri et al. (Narmashiri et al. 2010), CFRP strips in different numbers were bonded on one or both sides of steel web. The ratio between bonded composite and steel web areas in shear zones considered in the study was of 0.48 and 0.72. The maximum load capacity was increased about 51% of the un-retrofitted steel sections’ load capacity. However, the difference between these two ratios did not translate into a difference in the maximum load carrying capacities when CFRP strips were bonded on both sides. Zahurul Islam & Young (Zahurul Islam et al. 2014) investigated the behavior of artificially degraded stainless steel rectangular and square hallow sections. The crippling strength of the hollow sections were enhanced by bonding CFRP laminates to the web. Slenderness of the hollow sections, different adhesive types, surface preparations, and loading conditions were the investigated parameters in their experiments, from which strength gains between 4 and 76% were observed.
It is clear from these studies that retrofitting shear deficient sections has relied exclusively on planar CFRP composites. The work presented in this paper focuses on strengthening of steel beams using pultruded GFRP sections by bonding them to webs of shear-deficient beams. Two different SBS strengthening schemes were considered, and two different adhesive types were used to bond the pultruded GFRP sections. Load carrying capacities and ductility performances were investigated. For comparison purposes, one specimen was retrofitted using a conventional method whereby steel stiffeners are welded on both sides of the web panel. The design of welded steel stiffeners has been extensively studied, and design provisions already exist in design codes such as AASHTO (2012) and AISC (2010).

3.3 Experimental Program

I-shaped steel beams were fabricated for this study by a local professional supplier to mimic typical quality and workmanship prevalent in the industry. This includes welding materials and quality and initial distortions from design plans. This section describes the tested specimens and the material properties of steel, pultruded GFRP sections, CFRP fabric sheets, and adhesives.

3.3.1 Specimen Details

The SBS method for retrofitting shear deficient steel members was investigated using welded A36 grade steel built up I-shaped beams. The slenderness of the web panel was chosen such that the flange and stiffeners were overdesigned to ensure that no other local or global failures modes take place before the web buckles (Okeil et al. 2009a). The chosen web plate thickness was 3.2 mm [1/8 in.] for all steel beams reported in this study. This thickness resulted in a slender web as opposed to the overdesigned top and bottom flanges 12.70 mm [0.5 in.] and steel stiffeners 9.53 mm [3/8 in.]. Even though beams are typically designed with less slender web plates, exposure to aggressive environments often reduce the designed thickness significantly, and cause more slender webs than the initial design. The overall dimensions of the beams were designed such that the failure load was within the load capacity of the test setup. Transversely, welded steel stiffeners divided the web into equal panels whose dimensions were...
518 x 521 mm [20\(\frac{3}{8}\) x 20\(\frac{3}{8}\)in.], and 521 x 772 mm [20\(\frac{1}{2}\) x 30\(\frac{3}{8}\)in] for square and rectangular panel beams, respectively. Therefore, the resulting nominal panel aspect ratios (width to height) of square and rectangular web plates were 1:1 and 1.5:1. The total number of the panels in a steel beam was set to four and three for the square and rectangular configurations. Three point loading was applied for all the beams to determine the increase in ultimate load capacities and post-buckling behaviors. The load was applied on the first internal steel stiffener from one of the supports where the maximum shear stresses was carried. A schematic view of the test set-up can be seen in Figure 3.2 (a) and (b) for square and rectangular panel beams. The GFRP stiffeners were bonded on each side can be seen in Figure 3.3 (see Section B-B).

The dimensions of the reference specimen with conventional strengthening using steel stiffeners were the same as those used for SBS specimens. The steel stiffeners had the same clear height between the top and bottom flanges and were positioned at middle of the critical (exterior)
panel; i.e., splitting the length of the panel. The dimensions of the stiffeners were identical to the panel stiffeners; i.e., 114 x 508 x 9.53 mm [4 1/2 x 20 x 3/8 in.], and they were fully welded to web and flanges. The cross section of panel stiffeners and welded stiffeners in a failure region can be seen in Figure 3.3 (see Section A-A).

3.3.1 Adhesives

One of the adhesives, Tyfo® MB3, that was used in SBS method and bonding CFRP sheets method is recommended for metal bonding. The primary advantage of the Tyfo® MB3, epoxy is the excessive elongation ability before rupture (Okeil et al. 2015), which is not common for most of the structural epoxies that exhibit brittle behavior (ACI Committee 440 2007; Lee et al. 1967; Mays et al. 1992). The two-part adhesive was obtained mixing its components for at least 5 min. in room temperature until a uniform epoxy mix was obtained.

3.4 Material Properties

The other adhesive type used in this study, Tyfo® S, is mainly recommended for most generic structural bonding applications such as concrete. The final mix of Tyfo® S was similarly obtained mixing two components with a pre-defined mixing ratio. The workable consistency was adjusted to avoid run offs by adding fumed silica powder into the mix. Earlier work showed that Tyfo® S epoxy fails in brittle manner and did not have a post yielding behavior as obtained with
Tyfo® MB3 (Okeil et al. 2015). It should be noted that one of the major differences between the two adhesives is their viscosity. The viscosity of the Tyfo® MB3 epoxy in manufacturer’s data sheet is about 55000 cps (centipoise) which reduced the workability of Tyfo® MB3 epoxy for alternative applications; for example, impregnation or saturation of the CFRP sheets cannot be fully achieved with such high viscosity. Therefore, the other relatively low viscos epoxy (600 cps), Tyfo® S, was used to saturate CFRP sheets. The mechanical properties of the both epoxies are given in Table 3.1.

3.4.2 GFRP Stiffeners

The main advantage of the proposed SBS method is the utilization of low modulus fibers (i.e. glass fibers) instead of using high or ultra- high modulus fibers (e.g. carbon fibers) (Okeil et al. 2009a). The out-of-plane strength of the pultruded sections does not only depend on the fiber type in the matrix but also the geometric properties of the stiffener’s cross section. Since the out-of-plane properties of the cross section provide ample resistance, it was observed that the failure takes place at the interface (Okeil et al. 2009b). This bond failure at the adhesive/steel interface for the FRP/steel bonding is a dominant mode of failure (Harries et al. 2012; Okeil et al. 2009b). The pultruded GFRP T-shaped stiffeners used in this study were cut from commercially available wide flange I-shaped sections [6 x 6 x 3/8 in.]. The desired length of the pultruded GFRP stiffeners was chosen to fit diagonally from the loading tip to the support location in square panel beam. The vertical stiffeners were bonded between the steel beam flanges

<table>
<thead>
<tr>
<th>Table 3.1 Material properties of epoxies</th>
</tr>
</thead>
<tbody>
<tr>
<td>Adhesives</td>
</tr>
<tr>
<td>Tyfo® S</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Tyfo® MB3</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
</tbody>
</table>
482.6 mm [19 in.] in rectangular panel beams. In both configurations, the GFRP stiffeners did not extend all the way to the shear panel surrounding elements (steel flanges or stiffeners). A gap of 12 mm (0.5 in.) was maintained to avoid loads other than those imposed on the GFRP stiffeners to resist web buckling. Two different retrofitting schemes of SBS method were considered in rectangular panel beams. In the first retrofitting scheme, one GFRP stiffener was bonded to one side of the web, whereas in the second, one GFRP stiffener was bonded on each side of the web. These two SBS methods were repeated using two different adhesive types, Tyfo® S and Tyfo® MB3 epoxies. The pultruded section used in SBS method was T-shaped beam obtained by cutting one of the flanges of Extern and PROform® WF beams. The flexural properties for the pultruded sections were obtained experimentally and are given in Table 3.2.

<table>
<thead>
<tr>
<th>Composites</th>
<th>$E_k$, GPa [ksi]</th>
<th>$\sigma_u$, MPa [ksi]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Extern</td>
<td>$\mu$ (mean)</td>
<td>13.20 [1915]</td>
</tr>
<tr>
<td></td>
<td>$\sigma$ (SD)</td>
<td>1.85 [268]</td>
</tr>
<tr>
<td></td>
<td>$C_V$ (%)</td>
<td>14.0</td>
</tr>
<tr>
<td>PROform</td>
<td>$\mu$ (mean)</td>
<td>24.38 [3536]</td>
</tr>
<tr>
<td></td>
<td>$\sigma$ (SD)</td>
<td>1.50 [216]</td>
</tr>
<tr>
<td></td>
<td>$C_V$ (%)</td>
<td>6.1</td>
</tr>
</tbody>
</table>

Table 3.2 Material properties of composites

<table>
<thead>
<tr>
<th>Composite</th>
<th>$E_k$, kN/mm [kip/in]</th>
<th>$\sigma_u$, N/mm [kip/in]</th>
</tr>
</thead>
<tbody>
<tr>
<td>CFRP</td>
<td>$\mu$ (mean)</td>
<td>36.35 [150.5]</td>
</tr>
<tr>
<td></td>
<td>SD</td>
<td>2.51 [14.32]</td>
</tr>
<tr>
<td></td>
<td>$C_V$ (%)</td>
<td>9.5</td>
</tr>
</tbody>
</table>

3.4.3 CFRP Sheets

The fibers of bidirectional CFRP sheet were orientated at 0° and 90°, and bonded over the entire web panel area. The CFRP sheets were first cut to the desired size of the first web panel 508 x 762 mm [20 x 30 in.]. The CFRP sheets were then saturated with Tyfo® S epoxy whose viscosity allows easier saturation of the fibers. Tyfo® MB3 epoxy was then applied to the web panel before applying the saturated CFRP sheets before the Tyfo® S cured. The schematic illustration of the bonded biaxial CFRP sheets and its cross section can be seen in Figure 3.2 (b)
and Figure 3.3 (see Section C-C). The tensile properties of the cured CFRP sheets were determined experimentally and are given in Table 3.2.

3.5 Specimen Preparations

3.5.1 Surface Treatments

Special surface preparations are necessary for steel sections and fabricated pultruded GFRP sections to remove painted, polished, milled or corroded surface substances. Good bond quality can be achieved with properly cleaned surfaces and well prepared adhesives. The premature failure of the bonded joints can be eliminated with an appropriate substrate treatments (Baldan 2004; Okeil et al. 2009a). Steel surface treatments using chemical etchants or curing in high temperature can yield higher bond performances; however, the pollutant effects of chemicals, and the difficulty of creating a proper curing environment in the field are the major obstacles for the steel substrate preparations (Wegman et al. 2012).

Another steel surface preparation is the grit blasting or dry abrading techniques to create cleaned and roughened steel surface. In this research, poly abrasive wheels and sand papers were used to reach a clean white metallic surface at and slightly beyond the bonding region. On the other side, the pultruded GFRP sections were manufactured with a polished glazed finish. Therefore, it was necessary to remove the glazed finish to improve bonding by creating some roughness. This was done by first using abrasive sand papers on the glazed bonding surface. Then, the bonding surface was scratched with a chisel that created minute notches (measured in fraction of millimeter) providing the additional bond strength when filled with epoxy. This step could be avoided for the GFRP stiffeners in future commercialization of the proposed SBS method if the composite sections were to be produced with fabricated notches/rough surfaces to reduce the efforts of composite surface preparations.

3.5.2 Bonding the Composites

In SBS method, the roughened steel and GFRP sections were cleaned with acetone to eliminate any minor residues that prohibit chemical interaction between epoxy and substrates. The GFRP sections were placed diagonally in square panel, and vertically in rectangular panel
beams. The diagonal stiffener was placed between the top and bottom flanges from support to the load line direction with 45° angle. The vertical stiffeners were placed at mid length of the rectangular web panel. After applying adhesive on both surfaces, the pultruded stiffeners were pressed towards the roughened and cleaned steel surface to ensure that the contact between the GFRP stiffener and the steel web is complete and eliminate any gaps. The average epoxy thickness was recorded 2.5 mm in SBS applications. Even though less epoxy thicknesses were reported for similar application in literature, the size of real bridge girders with possible imperfections render the measured average epoxy thickness acceptable for real field conditions given that both adherents are relatively stiff and cannot accommodate such imperfections. Any excessive epoxy that oozed around the bonded GFRP section was scraped to eliminate adding an artificial web thickness, which can also lead to earlier adhesive cracking.

The CFRP sheets were saturated by laying them down into an epoxy filled container until they were fully soaked with Tyfo® S epoxy before bonding. As stated earlier, the viscosity of the Tyfo® S is much lower than the Tyfo® MB3, therefore, saturation was completed properly with lower viscos epoxy. The fully impregnated CFRP sheets were then placed on the fully cleaned web panel, which was already coated with the highly viscous, but more appropriate for steel bonding, Tyfo® MB3 epoxy. The CFRP sheet was firmly pressed with a cylindrical roller to generate the full contact between epoxy and substrates. The same procedure was repeated for the other side of the web panel.

3.5.3 Tested Specimens

Alternative retrofitting options were experimentally investigated by manufacturing square and rectangular panel beams. Diagonally oriented pultruded FRP sections were bonded in square panel beam only (SP-2GFRP-S), and compared with its control beam that was tested without FRP sections (SP-0). The diagonal FRPs were bonded using Tyfo® S epoxy along the compression field of the web panel. The rest of the retrofitting options were applied on the rectangular panel beams which was manufactured with 1:5 panel aspect ratio. The load was applied on the first panel and monolithically increased to failure or until certain displacement
level deemed indicative of a failure level. One beam was tested as a control beam without any retrofitting (RP-0). Another beam was tested with vertically welded steel stiffeners on both sides of the web panel (RP-2STS). The other four beams were retrofitted considering two different GFRP section orientations, that is bonding one or two (one on each side) GFRP sections and using two epoxy types; namely Tyfo®, MB3 and Tyfo®, S, (RP-1GFRP-MB3, RP-2GFRP-MB3, RP-1GFRP-S, and RP-2GFRP-S). Finally, one beam was retrofitted bonding biaxial CFRP sheets fully covering the both sides of the web plate (RP-2CFRP-MB3). The short designations given in parenthesis indicate the web panel aspect ratio (square or rectangular – SP or RP); number of stiffeners on the web panel (1 or 2); type of stiffener (steel, pultruded GFRP section, or biaxial CFRP sheet – STS, GFRP or CFRP); type of epoxy (MB3 or S). These notations will be used in the following sections.

The test loads were applied using an MTS hydraulic machine with a 550 kip capacity. The applied load and deflection values were recorded using an MTS controller for the first; i.e., critical, panel of the beams at the first internal steel stiffener. The same reading were transferred to a Natural Instruments data acquisition (NI-DAQ) system in addition to other strain and LVDT readings that were directly logged by the NI-DAQ. Linear strain gauges were placed on top and bottom surface of the tension and compression flanges at the load line. Linear voltage differential transformers (LVDTs) were placed at the loading point of the top flange and the supports of the beams to allow capturing the real deflection values of the top flange excluding the flexibility of crosshead connections. The system flexibility was removed from the real crosshead displacements by subtracting the support movements recorded at each support using LVDTs from the LVDT reading at the loading point.

3.6 Experimental Results

3.6.1 Load-Deflection Curves

Load-deflection, $P-\Delta$, relationships for the tested beams will be presented here. The change in ultimate load capacities for different alternative retrofitting schemes will also be discussed. As stated earlier, all the beams were tested with a nominal web thickness of 3.2 mm
[1/8 in], and the top and bottom flanges had identical dimensions. Therefore, the differences in observed behaviors are solely due to the different configurations described earlier.

The $P$-Δ relationship for the diagonal retrofitting scheme, SP-2GFRP-S, is plotted with a control unretrofitted square panel beam in Figure 3.4. The $P$-Δ plot for the unstiffened beam, SP-0, in Figure 3.4 followed the expected behavior for steel beams with a well-defined yield plateau. The test procedure was stopped after 6.35 mm [0.25 in] deflection as no increase in capacity was taking place. The diagonally stiffened beam reached a maximum load level of 434kN [97.5 kips] at a 4.57 mm [0.18 in] load tip deflection. At this load level, a sudden load drop exceeding 10 kips took place, which triggered a failure detection command in the test procedure. The load drop was accompanied by audible epoxy cracking noise indicating initial stage of losing bond between the GFRP stiffener and the steel web. Nevertheless, the web of the diagonally stiffened beam did not show any visible buckling like the control beam when the test was stopped. However, it can be stated that had the test procedure continued, complete debonding would have ensued and a post buckling behavior similar to that of the control beam would have been

![Figure 3.4 Load deflection curves of SP-0 and SP-2GFRP-S beams](image)
obtained. It is important to note that Specimen SP-2GFRP-S exhibited far less ductility in comparison with the control beam. While this is true of many FRP strengthening schemes, it is more pronounced here because of the orientation of the GFRP stiffener, which makes it a load bearing member along the compression strut as opposed to being just a stiffening element as will be seen later. The available data showed that the ultimate load capacity of the diagonally retrofitted beam increased 56% of the unstiffened beam at failure detection, and lost its 12% of that load when failure ensued.

Two sets of specimens were tested to investigate the behavior of SBS using vertical GFRP stiffener orientation. The difference between the two sets was in the type of adhesive used to bond the GFRP stiffeners. Each set consisted of two specimens where one and two GFRP stiffeners were bonded to the critical web panel; namely RP-1GFRP-S and RP-2GFRP-S for Tyfo® S adhesive, and RP-1GFRP-MB3 and RP-2GFRP-MB3 for Tyfo® MB3 adhesive. A control beam, RP-0, was also tested for comparison purposes. The $P-\Delta$ relationships for these specimens are plotted in Figure 3.5 and Figure 3.6. In both figures, the specimens with one GFRP stiffener (RP-1GFRP-S and RP-1GFRP-MB3) show distinct failure signs at different load stages. Specimen RP-1GFRP-S reached 289kN [65 kips] load level before an initial load drop suddenly took place, which was still 30% than the resistance of unstiffened control beam, RP-0. A subsequent load drop brought the beam resistance to a level close to that of the control beam indicating a complete loss of the added stiffening effect. The other beam, RP-2GFRP-S, showed 36% increase in load carrying capacity over the control beam, which is slightly better than that of the one stiffener configuration (RP-1GFRP-S). However, after the beam reached its maximum resistance of 302 kN [68 kips], a more stable flat post buckling plateau in comparison with RP-1GFRP-S was observed. Eventually, debonding occurred and the load resistance dropped at a deflection equal to 11.30 mm [0.44 in.], which was much larger than observed for the specimen with diagonal stiffeners, SP-2GFRP-S. Similar SBS retrofitting schemes were repeated with using Tyfo® MB3 epoxy which was primarily introduced for steel bonding applications, and the load-deflection plots for this set can be seen in Figure 3.6. The beam with one stiffener, RP-
1GFRP-MB3, reached a capacity of 302kN [68 kips] before failure which was the same maximum load of the beam stiffened with two stiffeners using Tyfo\textsuperscript{®} S epoxy, RP-2GFRP-S. However, the beam with two stiffeners, RP-2GFRP-MB3, resisted a higher load of 336kN [75.5 kips] without any signs of load drops indicating debonding up to a deflection equal to 16.5 mm [0.65 in] when the test procedure was stopped. The load capacity kept increasing for this specimen all the way to when the procedure was stopped, at which a 51% increase in load capacity had been achieved. The plots presented in Figure 3.5 and Figure 3.6 show that failure ensued by a clear drop in the beams load capacity for specimens with one stiffener (RP-1GFRP-S and RP-1GFRP-MB3). This behavior was not observed for specimens with two stiffeners (RP-2GFRP-S and RP-2GFRP-MB3). This behavior can be explained by two reasons. The first is that bonding two GFRP stiffeners opposite each other with the web plate in between subjects the extremities of the bonding adhesive to tensile stresses on one side and compressive stresses on
the other. It is known that the tensile strength of epoxy materials are weaker than their compressive strength, which also leads to more elongation (Fiedler 2001; Littell 2008).

As a result, the epoxy under compression will still be intact and in contact with the web panel and stiffener even after the tension side epoxy failure. Therefore, the two sided stiffeners is less likely to experience sudden load drops than single stiffeners (Ulger et al. 2016). The experimental observations clearly confirms debonding starts in adhesive regions under tension, which eventually leads to stiffener separation from the web. The second reason for the difference in behavior between one and two stiffener specimens is that the two-sided stiffener configuration provide higher bracing for the web plate, which practically means that the panel is split into two smaller panels (Ulger et al. 2016). Therefore, the efficiency of one stiffener in achieving such a behavior is lesser than that for two-sided stiffener configurations. The separation between the web plate and composite section can be seen in Figure 3.7 (a) for RP-1GFRP-S specimen which shows complete debonding of the single GFRP stiffener; however, there was not major
debonding observed for RP-2GFRP-MB3 specimen in Figure 3.7 (b) during the post-buckling stage.

The load deflection curves for the alternative retrofitting schemes; i.e. welding steel stiffener and CFRP sheet, are plotted in Figure 3.8 with one of the proposed SBS method, RP-2GFRP-MB3. The conventional steel stiffener specimen, RP-2STS, shows a gradual increase in load capacity after yielding at a smaller constant post yielding stiffness. The load level reached in testing this specimen was 334kN [75kips] at 17.8mm [0.70in] displacement which was 50% than the capacity of the unstiffened beam, RP-0. In comparison to the behavior of Specimen RP-2STS, Specimen RP-2GFRP-MB3 also did not show failure sigh after the initiation of buckling. In other words, there was not significant difference between SBS method and conventional steel stiffener welding in terms of post buckling behaviors and to a lesser extent the maximum load levels achieved with 1% difference by both methods. The maximum loads for all specimens are given in Table 3.3. The buckled web panel for RP-2STS specimen is shown in Figure 3.7 (c)
during the post-buckling stage, which clearly demonstrated that the web panel was split into two smaller panels as is evident from the buckling of the web in both split panels.

The specimen with bonded FRP sheets to web panel, RP-2CFRP-MB3, was also tested in this study and the load deflection plot is shown in Figure 3.8. The critical web panel for this specimen was fully covered with bonded CFRP sheets on both sides. Specimen RP-2CFRP-MB3 achieving an 86% increase in load resistance in comparison with the unstiffened beam, RP-0.

The specimen behavior was similar to that observed for the steel stiffener welded beam, RP-2STS, and the two-aided SBS specimen with MB3® epoxy, RP-2GFRP-MB3. The post-buckling load capacity kept increasing gradually with almost a constant stiffness up to the deflection limit when the test procedure stopped. The buckled web panel for Specimen RP-2CFRP-MB3 is shown in Figure 3.7 (d) at time of procedure stopped. The higher load deflection curve performance of RP-2CFRP-MB3 beam can be attributed to the fact that the entire web surface was covered with CFRP sheets, hence, the epoxy layers and the CFRP sheets on both sides create additional web thickness in the critical web panel. In comparison, the SBS stiffeners
used in this study, covered an area of the web panel equal to the area of the GFRP flange; i.e., 152 x 483 mm [6x19 in.], which translates into 20% of entire web panel area. The effective parameters of the different retrofitting alternatives using composites will be scope of the future studies. Finite element models will be used for that purpose to evaluate efficiency of composite retrofitting in more detail to cover a larger range of parameters than could be tested experimentally.

### 3.6.2 Post Buckling and Ductility

As noted earlier, the use of composite materials in retrofitting applications typically relies on utilizing the uniaxial strength of the fibers, and most research efforts report gains in load capacity before failure. This gain in strength is accompanied by a reduction in ductility (Lee et al. 1967; Mays et al. 1992; Okeil et al. 2009a) with the exception of a few special applications. Several researchers studied to increase the ductility and toughness of the adhesives to obtain better bonding performance by reducing premature failures (Dean 2004; Imanaka et al. 2009; Saldanha et al. 2013). The ductility of the SBS specimens were evaluated for two different epoxies, and was found that ductile epoxies increase the performance of the retrofitted beams (Okeil et al. 2015). The behavior of ductile epoxy coupons revealed a flat plateau similar to that known for steel yielding (Okeil et al. 2015). This ductile epoxy behavior allows the redistribution of the stresses between the steel and composite substrates once the stresses reach its maximum.

<table>
<thead>
<tr>
<th>Beam Labels</th>
<th>Epoxy Type</th>
<th>Failure Load kN [kips]</th>
<th>Capacity Increase</th>
<th>Ductility $\left(\frac{E_{\text{inel}}}{E_{\text{total}}}\right)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>SP</td>
<td>-</td>
<td>278 [62.5]</td>
<td>-</td>
<td>0.86</td>
</tr>
<tr>
<td>2 GFRP$^{E1}$</td>
<td>S</td>
<td>434 [97.5]</td>
<td>56%</td>
<td>0.30</td>
</tr>
<tr>
<td>0</td>
<td>-</td>
<td>222 [50.0]</td>
<td>-</td>
<td>0.86</td>
</tr>
<tr>
<td>1 GFRP$^{P}$</td>
<td>S</td>
<td>289 [65.0]</td>
<td>30%</td>
<td>0.18</td>
</tr>
<tr>
<td>2 GFRP$^{P}$</td>
<td>S</td>
<td>302 [68.0]</td>
<td>36%</td>
<td>0.84</td>
</tr>
<tr>
<td>RP</td>
<td>1 GFRP$^{P}$</td>
<td>302 [68.0]</td>
<td>36%</td>
<td>0.83</td>
</tr>
<tr>
<td></td>
<td>2 GFRP$^{P}$</td>
<td>336 [75.5]</td>
<td>51%</td>
<td>0.82</td>
</tr>
<tr>
<td></td>
<td>2 CFRP$^{P}$</td>
<td>414 [93.0]</td>
<td>86%</td>
<td>0.85</td>
</tr>
<tr>
<td></td>
<td>2 STS$^{2}$</td>
<td>334 [75.0]</td>
<td>50%</td>
<td>0.87</td>
</tr>
</tbody>
</table>

$^{E}$ Extern®, $^{P}$ PROfrom®, $^{1}$ Diagonal stiffener orientation, $^{2}$ Welded connection
value which happens after buckling ensues as a result of the high local deformations in the
critical web panel. Therefore, the expected sudden load drops as seen in brittle epoxies was not
observed when ductile epoxies were used to bond either pultruded GFRP sections or CFRP
sheets.

The ductility of SBS, conventional, and alternative retrofitting methods were studied by
evaluating the ratio of inelastic energy absorbed by the system to the total energy up to failure.
This measure is often referred to as energy ductility index, $\mu_E$. For specimens where a clear
failure point could not be observed, energy ductility was determined at the point when the
deflection limit was reached for comparison purposes. For that purpose, if a deflection limit of
12.7 mm [0.50 in.], which was reached or passed by all the specimens without failure, to be
taken as the limit at which the energy ductility was determined. A major load drop was perceived
as a sign of failure in load deflection curves. The diagonally stiffened square panel beam, SP-
2GFRP-S, one and two GFRP bonded beams using Tyfo® S, RP-1GFRP-S and RP-2GFRP-S,
and one GFRP bonded beam using Tyfo® MB3, RP-2GFRP-MB3, showed sudden load drops,
and did not reach the 12.7 mm [0.50 in.] deflection limit. Therefore, energy calculations did not
account for resistance after these initial load drops.

The total energy, $E_{tot}$, was calculated as the area under the load deflection curve until the
failure or deflection limit. The inelastic energy, $E_{inel}$, was found by subtracting the elastic energy,
$E_{el}$, from the total energy, $E_{tot}$. The elastic energy, $E_{el}$, is estimated from the triangular area
formed between the failure/limit displacement and the linear unloading line of the load deflection
curves. Linear unloading slope was assumed same as the slope of initial linear part of the load
deflection curve. One example of total, $E_{tot}$, and elastic, $E_{el}$, energy areas are shown in Figure
3.9. The energy ductility index, $\mu_E$, was then defined as the ratio of $E_{inel}/E_{tot}$ and given in Table
3.3.

All the ductility index values without and with retrofitting schemes were calculated to be
82% or above except for the diagonally stiffened specimen, SP-2GFRP-S and for the specimen
with one GFRP stiffener bonded using Tyfo® S, RP-1GFRP-S where the ductility index was 30
and 18%, respectively). The first specimen, SP-2GFRP-S, failed abruptly once the diagonally orientated stiffener debonded from the web panel. The load drop was massive and sudden as a result of the stiffener orientation, which makes it a force bearer as a compression strut. Such an orientation puts higher loads demands on the stiffener, which makes it more susceptible to the premature failure (Okeil et al. 2011). Consequently, diagonal stiffener orientation was abandoned and the vertical stiffener orientation was chosen for all subsequent SBS experiments to avoid such failures with low ductility despite the larger gain in strength. The second specimen with low ductility, RP-2GFRP-S, was stiffened with vertical stiffener orientation, but only one stiffener was bonded using Tyfo® S epoxy, which is a brittle adhesive (Okeil et al. 2015). As was discussed earlier in the load-deflection curves section, the failure mode of the one stiffener specimen is different than that or double-sided stiffeners, which help each other in resisting the stress state whether it is tension or compression. Therefore, stiffening beams with one stiffener should be done with extreme care, especially if a brittle adhesive is used. The proposed SBS method where the stiffeners were bonded with ductile epoxy performed successfully when

---

**Figure 3.9** Energy ductility index ($E_{\text{total}} = E_{\text{inelastic}} + E_{\text{elastic}}$)
compared with the traditional and alternative retrofitting applications in terms of ductility measure.

3.6.3 Strain Readings

Results from the rectangular panel beams that were instrumented with strain gages are presented in this section. As built, the elastic buckling of web failure was the expected failure mode according to the AASHTO LRFD bridge design specifications (AASHTO 2012). The longitudinal strains in the tension and compression flanges under the load were recorded at top and bottom surfaces of the tension and compression flanges. The sign convention in the following plots is positive for tension strains and negative for compression strains. As expected, the maximum tension and compression strains were recorded at the bottom of the tension flange and the top of the compression flange, respectively. The strain plots in Figure 3.10 (a) and (b) are for two of the tested SBS beams, namely the specimen with two GFRP stiffener bonded with Tyfo® S epoxy, RP-2GFRP-S, and the specimen with one GFRP stiffener bonded with Tyfo® MB3 epoxy, RP-1GFRP-MB3. The maximum strains were obtained in tension flanges for both beams and given in Table 3.4 as 660 and 600 microstrains for Specimens RP-1GFRP-MB3 and RP-2GFRP-S, respectively. These strain values are very similar to the ones measured for the beam strengthened by welding an additional steel stiffener, RP-2STS, for which 560 microstrains were recorded (see Figure 3.11 (a)). The maximum measured strain for the beam with CFRP sheet, RP-2CFRP-MB3, was 630. It should be noted that the top flange strains of RP-2STS and

![Figure 3.10 Flange strain readings of (a) RP-2GFRP-S and (b) RP-1GFRP-MB3](image-url)
RP-2CFRP-MB3 measured lower than the expected values as can be seen in Table 3.4. This may be attributed to problems with strain gage installation. The maximum strain readings confirm that the designed beam’s flanges stayed in elastic stress range until the web buckled. The strain values at the top of the tension flange started with a positive tensile strain. Once the web buckling initiated, the measured strains shifted from positive; i.e., tension, to negative strain; i.e., compression. The opposite is also true for the bottom strains of the compression flange. Once the web buckles, the tension field action takes place in post buckling phase and plastic hinges starts forming on the flanges around the tension field area which cause the sway frame mechanism to become a substitute load path for the applied loads up to failure. The challenge here is the quantification of the percentage of load being carried by the web panel versus the sway-frame in the linear and post buckling phases. Determining this percentage experimentally requires massive instrumentation of test specimens; therefore, it will be addressed using a calibrated finite element model in future studies.

Finally, a rosette was placed on each side of the RP-2CFRP-MB3 beam’s web panel for strain measurement. A reference axis strain was captured at 0° vertical placement and two others
at ±45° from the reference axis at the center of the web plate. The tension tie (T) is roughly
aligned with +45° axis and the compression strut (C) is roughly aligned with -45° of the rosettes
in Figure 3.12. The sign convention for tension and compression strains was positive and
negative, respectively. The figure shows that the sign of the strains remained unchanged whether
the web buckled or not for the -45° and +45° strain readings kept the same initial signs with
higher strain rate increment after yielding. Conversely, the strain sign changed at the reference
axis, which indicates that the web buckled backward. Once the web buckled backward, the back
strains turned to tension strains, and the front strains remained under compression with higher
strain rate increment after yielding similar to the -45° and +45° strain readings but about 84% lesser than tension and compression strains. Therefore, the resistance demand in tension and
compression ties are more pronounced than the vertical resistance of fibers.

The maximum principal strains (ε₁ and ε₂) and principal axis rotation (α) can be derived
using fundamental strain transformation equations. Using the relations in Eq.( 3.1 ), the final
form of the principal strain equations and principal axis rotation, α, can be obtained as given in
Eqs. ( 3.2 ) and ( 3.3 ).

The calculated principal strains are shown in Figure 3.12. The maximum principal strain
was 1580 microstrain around the yield load, 285 kN [64 kips], and 16100 microstrain at the
maximum load, 415 kN [93.27 kips]. The principal axis rotation was calculated to be less than 1°
in linear loading stage.

\[
\varepsilon_{-45} = \frac{1}{2} (\varepsilon_1 + \varepsilon_2) + \frac{1}{2} (\varepsilon_1 - \varepsilon_2) \cos 2\alpha
\]

\[
\varepsilon_0 = \frac{1}{2} (\varepsilon_1 + \varepsilon_2) + \frac{1}{2} (\varepsilon_1 - \varepsilon_2) \cos 2(\alpha + 45°)
\]

\[
\varepsilon_{+45} = \frac{1}{2} (\varepsilon_1 + \varepsilon_2) + \frac{1}{2} (\varepsilon_1 - \varepsilon_2) \cos 2(\alpha + 90°)
\]

\[
\varepsilon_{1,2} = \frac{(\varepsilon_{-45} + \varepsilon_{+45})}{2} \pm \frac{1}{\sqrt{2}} \sqrt{\left(\varepsilon_{-45} - \varepsilon_0\right)^2 + \left(\varepsilon_0 - \varepsilon_{+45}\right)^2}
\]

\[
\alpha = \frac{1}{2} \arctan \left( \frac{\varepsilon_{-45} - 2\varepsilon_0 + \varepsilon_{+45}}{\varepsilon_{-45} - \varepsilon_{+45}} \right)
\]
3.7 Comments on Strengthening Alternatives

It is important to learn their advantages and disadvantages to help in choosing an appropriate strengthening alternative. The following comments should help in making such a decision.

The conventional steel stiffener technique requires welding which is well established and extensively used method in the field. However, that brings the well-known issues such as fatigue due to stress concentrations, heavy material and equipment necessity in field applications, certified labor to conduct the welding on site, which may not be easy if large initial imperfections exist. On the other side, composites offer a light material alternative for site handling, ease of application, and less stringent labor requirements. The main difference between the two composite alternatives, SBS and bonding composite sheets, is the way fiber resistance is utilized. The pultruded composite sections used in SBS mainly depend the geometrical properties of the section as an out-of-plane resistance with minor contribution of uniaxial resistance of...
fibers. Conversely, composite sheets can only contribute in-plane resistance in strengthening applications as their out of plane contribution is practically minimal.

One of the advantages of the fully bonded web plates using composite sheets is that it provides an additional protection against environmental attacks and corrosion which reduces the preventative maintenance in web plates. This is due to the fact that the impregnated sheets serve as a tight barrier that prevents oxygen from reaching the metal. However, fully bonded composite sheets in web panel requires more effort in preparing the entire deficient web panel area and requires more expensive material; i.e. Carbon FRP, than the proposed SBS method.

3.8 Conclusions

In this paper, three different retrofitting techniques, Strengthening-By-Stiffening or SBS, CFRP sheets, and welded steel stiffener, were investigated experimentally by testing built-up steel beams. The alternative strengthening techniques are based on stiffening buckling prone shear deficient web panels. The beams were loaded monotonically in unsymmetric three point loading setup creating a critical web panel. The main difference between the proposed SBS retrofitting method and others is the utilization of composites where the out-of-plane resistance of the pultruded sections were the main contribution for enhancing the strength of shear deficient regions. The experimental investigation covered with different configurations of bonded pultruded GFRP sections. Two different epoxy materials, Tyfo® S and Tyfo® MB3, were also used to bond the pultruded GFRP sections. Bonding biaxial CFRP sheets to a web panel was also investigated, and as opposed to the SBS method, in-plane resistance of fibers is the main contribution for enhancing the shear strength of the web. The CFRP sheets were saturated with Tyfo® S, and bonded with Tyfo® MB3 epoxy material. Finally, the conventional steel stiffeners were welded in the web panel. The advantages and disadvantages of the retrofitting alternatives were discussed. Based on the experimental results, the following conclusions may be drawn:

1. There was not a significant difference in maximum load capacities of SBS retrofitted beams using different epoxy types; however, the use of ductile epoxy type in SBS method performs better than generic brittle epoxy type in term of ductility measure.
It was found that the ductile epoxy allows the redistribution of stresses at the interface, which is the critical part in the system.

2. The maximum load capacity increased by 86% compared to the unstiffened control beam when retrofitted using biaxial CFRP sheets. Welding steel stiffeners and SBS method with two GFRP stiffeners bonded with Tyfo® MB3 reached about the same load capacity increase, 51 and 50% within the specified crosshead displacement. There was not any premature failure observed when the web plate retrofitted using welded steel stiffener, bonded CFRP sheets, or bonded two-sided GFRP sections within the displacement limit.

3. Using one stiffener in SBS retrofitting makes the stiffeners more susceptible to early debonding in shear strengthening applications. The existence of a second stiffener on the other side of the web plate mitigates the premature failures because the epoxy layers that faces each other in two stiffener bonded case complement the tension side that is susceptible to failure with compression stresses on the opposite side.

4. Once the bonded pultruded sections failed and the web plate bulged out of plane, the tension field action took place in the buckled web panels which sustained the unstrengthened load capacity of the beam. In later stage of loading, plastic hinges formed in the flanges, and sway frame failure mechanism was observed.

In summary, the experimental study explored the differences in behavior between possible retrofitting alternatives of the deficient thin walled steel sections using composites materials and conventional welded steel stiffeners. The proposed SBS method is appealing because of it only requires relatively inexpensive composite materials such as GFRP as the major strength contribution is generated out of plane stiffness determined by the geometrical properties of the pultruded sections. Bonding CFRP sheets on the web plate provided promising results as well as the SBS method. More investigations are required for the use of planar composites in strengthening the shear deficient regions. The adhesive is the most important link in bonding applications, and advanced adhesives for steel bonding such as ductile epoxy types will improve
the bonding performance in retrofitting the steel members. Finally, the long term post buckling and ductility performance of the SBS method was under investigation of our research team when the SBS retrofitted beam exposed harsh environment.

3.9 Nomenclature

- $C_V$ Coefficient of variation
- $E_{el}$ Elastic ductility energy
- $E_{ine}$ Inelastic ductility energy
- $E_k$ Inelastic modulus of materials
- $E_{tot}$ Total ductility energy
- $P$ Applied load on the control panel
- $\alpha$ Principal axis rotation
- $\Delta$ Vertical displacement at the load line
- $\varepsilon_0$ Rectangular rosette vertical strain
- $\varepsilon_{1,2}$ Maximum principal strains
- $\varepsilon_{45}$ Rectangular rosette compression tie strain
- $\varepsilon_{+45}$ Rectangular rosette tension tie strain
- $\mu$ Mean of sample
- $\mu_E$ Energy ductility index
- $\sigma$ Standard deviation of sample
- $\sigma_u$ Ultimate failure load

3.10 References


The ISIS Canada Research Center (2004). "FRP Rehabilitation of Reinforced Concrete Structures." *Design Manual No. 4* Quebec, Canada.


Chapter 4. Mixed Mode Fracture Properties of Adhesives for FRP Strengthening of Steel Structures

4.1 Introduction

Structural retrofitting techniques offer more economical and practical solution for extending the service life of aging and deficient structures. Of the many alternative materials, composite materials such as fiber reinforced polymers (FRP) are in high demand for retrofitting civil structures as a result of their light weight, corrosion and fatigue resistance. Typically, composite sheets, plates, or strips are externally bonded to the structure. Advanced manufacturing technologies of composite materials provide alternative utilizations of this relatively new material for structural retrofitting applications. Several of these applications for concrete and steel structures can be found in (Buyukozturk et al. 2004; El-Sokkary et al. 2013; Fam et al. 2006; Nanni 1995; Okeil et al. 2009; Patnaik et al. 2008; Ritchie et al. 1991; Sen et al. 2001; Triantafillou et al. 1992).

One novel retrofitting application is bonding pultruded composite sections to web panels of shear deficient regions of steel girder. This technique, referred to as Strengthening-By-Stiffening or SBS, was proposed and experimentally tested under monotonic loading in a pilot study (Okeil et al. 2009). The test configuration and geometric properties of the retrofitted beams and section details are shown in Figure 4.1 and Figure 4.2. The main mode of failure for the retrofitted beams was observed to be debonding of the adhesive layer between the pultruded stiffener and the steel plate. Only one out of fourteen SBS retrofitted beams experienced partial ply delamination at one corner of the GFRP stiffener. Therefore, understanding the failure mechanism of the adhesive layer is deemed important for capturing post buckling behavior of the strengthened beams.

Simulation of the experimental tests was the first step to validate a FE model that was developed for the proposed retrofitting method. The FE results for the tested beams were not expected to produce acceptable post buckling data without accounting the fracture mechanism of the adhesive materials. For example, experimentally observed sudden load drops for the tested
full scale beams cannot be simulated with perfect GFRP/steel bonded sections at adhesive layer. Furthermore, it was concluded, based on experimental observations, that the possibility of delamination failure in the GFRP sections is much less than the debonding failure of the adhesive layer. Therefore, the main fracture failure considered in this study was modeled to take place in the adhesive layer, where the interface was modeled as a cohesive material. Failure of the cohesive material can be simulated as a normal separation due to tension stresses, tangential separation due to the shear stresses or a combination of both tension and shear stresses. The amount of energy that is required to separate the substrates defines the critical fracture energy, and the ratio of shear stresses to tension stresses in cohesive zone models defines the phase angle. The complexity of the stress field in and around the bonded region of the SBS stiffened thin steel plates requires the identification of the mixed mode fracture parameters.

In this paper, the fracture phase angles of the epoxy layer between the bonded GFRP and the steel plates was investigated utilizing the submodelling technique built in ANSYS commercial finite element (FE) program. A full model of the analyzed structures was first built using solid elements. The displacement field around a single solid epoxy element layer in critical debonding regions were mapped to the submodel from the full model of a strengthened beam. Several effective parameters such as steel plate and epoxy layer thickness were then studied to investigate their effect on the fracture properties of the interface. Different planar crack sizes were introduced into the submodel to determine the phase angles under different load levels acting on the full scale FE model was subjected.

The second part of the study presented herein involved the experimental determination of the critical fracture energies by testing single leg bending (SLB) specimens. The traction-separation curves of two different epoxy materials were determined using Digital Image Correlation (DIC) techniques. Finally, the experimental data were validated using FE element models of the tested SLB specimens.

The main purpose for the estimation of phase angles and determination of the traction-separation curves is to establish a cohesive zone model (CZM) that can be included in a more
accurate full FE model of SBS strengthened beams. Such a full FE model should have a better ability to predict the post buckling behavior of experimentally tested beams.

4.2 Literature Review

The investigation of stress intensity factors (SIFs) still hold its importance in linear fracture mechanics. The most common of the SIFs calculation techniques under the complex loading conditions are: M-integral, virtual crack closure technique (VCCT), displacement interpolation and interaction integral method (de Morais 2007). The implementation of the alternative SIFs estimation methods for various crack types can be investigated using FE models.

Mixed mode SIFs in a cracked compact tension specimen were investigated experimentally using the interaction integral method, and digital image correlation techniques were utilized to capture displacement and strain fields around the crack tip (Rethore et al. 2005; Sutton et al. 1983). Similarly, the interaction integral approach was adopted for SIFs calculation at a surface crack at weld toes in circular K joints (Qian et al. 2006) where a three-dimensional
(3D) FE model was used to study variations in mixed mode SIFs considering different crack locations, loading conditions, and brace geometries. An embedded elliptical crack front was investigated with a 3D FE model (Ghajar et al. 2013). Due to the asymmetric geometry of the elliptical crack front, resultant mixed mode SIFs were obtained using contour integrals, and the corresponding phase angles were extracted for different crack sizes. Crack propagation in unidirectional fiber composite materials was simulated using extended finite element method, and the corresponding SIFs were calculated using interaction integral method for different crack geometries and materials (Cahill et al. 2014). Other than the interaction integral method, researchers have estimated SIFs using displacement extrapolation method under mixed loading conditions of an edge cracked plate (Souiyah et al. 2007), peak stresses method for a welded joints using 2D (Nisitani et al. 2000) and 3D finite element models (Meneghetti et al. 2014), and force method (de Morais 2007), among others.

The determination of the interlaminar fracture toughness in laminated and bonded joints is one of the most researched areas in fracture mechanics (Brunner 2000; Szekrényes et al. 2006). Fracture energy calculations were developed for different fracture modes using beam theory based solutions (Szekrényes et al. 2006). Hojo et al. (Hojo et al. 1995) investigated Mode I interlaminar fracture toughness in unidirectional laminates using double cantilever beam (DCB) specimens. Pure Mode I, II and III fracture toughness in bonded joints were studied experimentally using DCB specimens for various bond thicknesses (Chai 1995). The mixed mode fracture toughness between similar and dissimilar bonded substrates were determined with single leg bending (SLB) specimens (Davidson et al. 1996). Theoretically, different formulations were presented for Mode I and various mixed mode specimens (SLB, end node split (ELS) and mixed mode bending (MMB) (Szekrényes et al. 2006). Using the derived theoretical formulation and experimental data, fracture toughness for Mode I, Mode II and mixed mode failures were determined by Silva et al. (da Silva et al. 2011). The previous short review covers some of the published work in this area. Similar studies can be found in literature.
Other than global fracture toughness studies, local parameters defining the traction-separation laws were experimentally investigated, and theoretical formulations were derived treating the interfacial bond as springs (Ji et al. 2012; Olsson et al. 1989; Sørensen 2002). The experimental traction-separation behavior for pure Mode I fracture was studied using bonded DCB specimens, and the normal opening at the crack tip location was recorded with an extensometer (Sørensen 2002) and video capturing technique (Andersson et al. 2004; Ji et al. 2010). Similar bonded specimens using different loading and boundary conditions were tested to extract the pure local Mode II fracture parameters using end notched flexure (ENF) specimens (Alfredsson 2004; Leffler et al. 2007; Ouyang et al. 2009). Mixed mode fracture parameters were investigated using DCB specimens subjected to unequal end moments (Sørensen et al. 2006), DCB sandwiched specimens (Lundsgaard-Larsen et al. 2008), SLB specimens (Ji et al. 2012), and mixed mode bending MMB specimens (Cui et al. 2014).

The local fracture parameters obtained from the aforementioned experiments and/or formulations can then be implemented into FE models to define CZM material properties and efficiently simulate crack propagation numerically (Camanho et al. 2003; de Morais 2014; Turon 2007; Xu et al. 1993). This approach is also followed in the current work. This paper presents the first part of the approach where the local fracture parameters are determined by testing SLB specimens and theoretical stress-strain formulations. The obtained parameters are then used in enhanced FE models employing CZM to simulate the bonding interface in full scale SBS-strengthened beams, which is the subject of another paper under preparation by the authors.

4.3 Mixed Mode Fracture Investigation

In this section, the theory of interaction integral method is first briefly reviewed. The ANSYS commercial finite element program (ANSYS) was used to calculate of SIFs using path-independent contours. Details of the developed FE model and the assumptions made in the calculation of SIFs are presented. The complexity of the stress fields often leads to wrong predictions of failure modes that may happen at various locations. However, by studying experimental results (Okeil et al. 2009; Ulger et al. 2016), it was possible to identify the critical
locations where failure took place. In general, the failure mode in beams strengthened using the SBS technique started at the outer perimeter of the bonded GFRP plate. The majority of the deboning failure in SBS retrofitted beams was observed to take place on the concave side of the web panel where the epoxy material experienced the largest separation. Figure 4.3 shows the debonding location of the epoxy material at failure for a typical shear deficient beam whose critical web panel is strengthened using SBS. Based on experimental observations, a number of possible failure locations in the FE model was identified by finding the locations of the maximum distortion occurring to the epoxy between the web panel and the FRP plate. Figure 4.4 (a) shows the deformed shape contours resulting from the full beam FE. The locations where maximum distortion occurred (see Figure 4.4 (b)) were used to investigate fracture properties for adhesive failure in the current study. Within the critical failure location, three different edge epoxy elements were selected for the study using refined submodels. Each submodel represented as a single epoxy element located at the center of each of the three selected elements. A planar edge crack was inserted in the submodel in the middle of the epoxy thickness. The refined epoxy submodel was first validated before being used to investigate the effect of different parameters on the fracture properties. These parameters included web plate thickness, epoxy thickness, epoxy types, and crack size.

![Figure 4.3 Buckled panel failure in SBS retrofitted beams](image)

Figure 4.3 Buckled panel failure in SBS retrofitted beams
Calculation of Phase Angles Using Theoretical Formulations

Inclusion of a crack zone in FE models is widely used to investigate different fracture related problems. As stated earlier, several methods for estimating the mixed mode stress intensity factor (SIFs) have been used in the literature. The interaction integral method is one of the most widely used methods because it allows the capture of SIFs for different mode partitioning. This study adopts the interaction energy integral as described next. In the developed submodel, the epoxy material is assumed to be a homogenous linear elastic material. The formulation of interaction integral used to determine the SIFs is given in Eq. (4.1), and the relation between the interaction integral and SIFs is given in Eq. (4.2). The derivation details and definitions of the fields can be found elsewhere (Dolbow et al. 2002; Qian et al. 2006; Walters et al. 2006). Finally, the phase angle of each load step using SIFs was calculated using Eq. (4.3).

In these equations, $q_{ij}$ is a crack extension vector, $\sigma_{kl}$, $\epsilon_{kl}$, and $u_{k,i}$, are stress, strain, and displacement, and $\sigma_{kl}^{aux}$, $\epsilon_{kl}^{aux}$, and $u_{k,i}^{aux}$, are stress, strain, and displacement of auxiliary field. $E'$ is Young’s modulus, and $G$ is the shear modulus ($E' = E$ for plain stress and $E' = E/(1 - v^2)$.
for plain strain). $K_I, K_{II}, K_{III}$ and $K_I^{aux}, K_{II}^{aux}, K_{III}^{aux}$ are the SIFs and auxiliary SIFs for Mode I, II and III failures, respectively. $\Psi$ is the phase angle corresponding to the mixed mode fracture.

### 4.3.2 Validation of Epoxy Submodel

Before investigating the SIFs under a complex displacement field, the SIFs of a well-known edge crack problem in a finite plate under uniform tension stress was calculated using a 3D FE model assuming plane strain condition. The crack front was modeled using degenerated SOLID185 prism elements and surrounded with a total of nine elements which was more than the required minimum number of elements (Cao et al. 1998). Figure 4.5 shows the front view of an upper symmetric 3D FE model and a unit (1.0) thickness was assumed in FE model analysis. It is known that calculation of the $J$ integral is path independent; therefore, the average of nine layers was used to obtain the SIFs. The numerically obtained SIF results for the edge crack problem in a finite plate were compared with the values obtained using the closed-form expressions Eq. (4.4) and (4.5) proposed by Rooke (1976) for model validation. SIF results from the numerical simulations and Rooke’s closed form expressions are given in Table 4.1 for different crack length/plate width ratios ($a_c'/b$). The results here showed that the SIFs are in good agreement with an average of 2.25%. In the rest of this paper, similar meshing of a crack front with nine layers of surrounding elements was carried out in all epoxy submodel simulations. This validated model was adopted to investigate SIFs in in the adhesive layer at critical locations where it is subjected to complex displacement fields.

### 4.3.3 Full Beam Model

The FE beam and GFRP stiffener components of the full beam model were constructed using 3D structural SOLID185 element type from the ANSYS element library.
enhanced strain formulation was activated for SOLID185 element to eliminate possible shear locking effect within the thin sections of steel beams. Initially, the GFRP stiffener was assumed to be fully bonded to the web plate including the epoxy element in between the web plate and GFRP section. As stated earlier, the purpose of this step is to identify critical locations where the adhesive is subjected to the highest distortions and to obtain the corresponding displacement field. The experimentally obtained mechanical properties of the steel, GFRP composite, and epoxy material were included in the full beam model to account for actual material properties including nonlinearity. An initial distortion simulating manufacturing imperfections was imposed on the beam’s geometry in lieu of applying a perturbation force to trigger buckling. The assumed distorted shape was based on mode shapes obtained by solving the Eigen value problem. The maximum element size in the bonded region was 12.7 x 12.7 mm (0.5 x 0.5 in.) for the full beam model. This mesh size was selected after conducting a mesh sensitivity study whose results showed an acceptable 1.4% difference between ultimate load capacities of the FE simulations and experimentally tested beams. Mesh refinement was applied in bonded region for epoxy submodel analysis to extract the SIFs.

Figure 4.5 Front view of edge cracked plate under uniform stress
The epoxy layer thickness was measured to be between 2.0 and 2.5 mm (0.08 and 0.1 in.) in the beam specimens that the authors tested experimentally (Okeil et al. 2009; Ulger et al. 2016). In literature, the common bond line thickness reaches a maximum of 1 mm (0.04 in.) in thickness in ideal conditions. However, the field conditions generally cause variations in epoxy layer thickness because of environmental corrosion, distortion, or geometric difference between the two rigid adhering surfaces (steel web plate and GFRP stiffener). Furthermore, the adhesive type of choice for bonding steel components is extremely viscous, which makes it extremely difficult to have it successfully applied in thicknesses below 1 mm (0.04 in.). Therefore, the epoxy layer was investigated with 2.0, 2.5 and 3.0 mm thicknesses when determining the SIFs. This approach allowed the computation of the interaction integral with a planar crack assumption in homogenous materials.

The dimensions of validated 3D epoxy submodel were adjusted to match those of the selected single epoxy elements from the full beam model. The crack was assumed to be planar and splits the epoxy element in two at the mid-thickness of epoxy layer. The crack front was constructed using degenerated quadratic hexahedron SOLID185 elements to SOLID185 prism elements. The surrounding elements around the degenerated elements were then meshed with additional eight layers of quadratic hexahedron elements. The mean SIF was obtained by averaging results from nine layers of elements using interaction integral method. Figure 4.6 (a) shows one of the selected epoxy elements between the steel web plate and the GFRP stiffener. The meshing of the corresponding refined submodel of this epoxy element including planar edge crack is shown Figure 4.6 (b).

<table>
<thead>
<tr>
<th>$a'_c/b$</th>
<th>0.05</th>
<th>0.1</th>
<th>0.3</th>
<th>0.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K^*_{I}$</td>
<td>0.226</td>
<td>0.236</td>
<td>0.317</td>
<td>0.535</td>
</tr>
<tr>
<td>$K_{I,FE}$</td>
<td>0.226</td>
<td>0.236</td>
<td>0.317</td>
<td>0.535</td>
</tr>
<tr>
<td>$K^*<em>I / K</em>{I,FE}$</td>
<td>1.00</td>
<td>1.01</td>
<td>1.03</td>
<td>1.05</td>
</tr>
</tbody>
</table>

Table 4.1 SIFs of in a finite plate with an edge crack

4.3.4 Refined Epoxy Submodel
The crack front length was meshed with 3, 10 and 20 elements in the submodel to investigate the sensitivity of the mesh. The mean of the SIFs was calculated using each crack node along the crack front. For example, the mesh with 3 element along the crack front had 4 nodes along the crack front whose results were averaged to obtain the mean SIF. The difference between the calculated phase angles from the meshes with the considered number of elements along the crack front was less 1%. The element size beyond the ring elements surrounding the crack tip did not show any effect on the SIF results because it was far enough from the crack front to affect the SIFs. Therefore, the model that was used in subsequent analyses was meshed using 3 elements along the crack front, 10 elements beyond the ring elements along the crack plane, and 8 elements beyond the ring elements in normal direction of the crack plane as can be seen in Figure 4.6 (b).

After creating the epoxy submodel, the displacement field surrounding the selected epoxy element from the full beam FE model analysis were mapped to the finely meshed epoxy submodel using linear interpolation. The mapping was repeated for each load step up to failure load as determined from the full beam model. The phase angles were then determined using the epoxy submodel.

4.4 Mixed Mode Single Leg Bending (SLB) Tests

Engineering applications involving bonded components are subjected to complex stress and displacements fields in bonded regions. In this study, the fracture energy at the interface of

\[
K_I = \sigma \sqrt{\pi a_c'} \left[ 1.12 - 0.23 \left( \frac{a_c'}{b} \right) + 10.6 \left( \frac{a_c'}{b} \right)^2 - 21.7 \left( \frac{a_c'}{b} \right)^3 + 30.4 \left( \frac{a_c'}{b} \right)^4 \right]
\]

when \( h/b \geq 1 \) and \( a_c'/b \leq 0.6 \)

\[
K_I = \sigma \sqrt{\pi a_c'} \left[ \frac{1 + 3 \frac{a_c'}{b}}{2 \sqrt{\frac{a_c'}{b} \left( 1 - \frac{a_c'}{b} \right)^{3/2}}} \right]
\]

when \( h/b \geq 1 \) and \( a_c'/b \geq 0.3 \)
bonded dissimilar materials was investigated by experimentally testing SLB specimens to account for the mixed mode failure.

The fracture energy can be separated into two components; namely, traction and separation. The traction in mixed mode fracture is represented by stresses in normal and tangential directions, while the separation corresponds to the normal and tangential directions for Mode I and Mode II fractures. The fracture at the interface of bonded dissimilar materials in FE element models can be simulated using cohesive zone models (CZM) where the cohesive material follows one of the traction and separation laws. The traction and separation laws in an SLB test can be defined as a combination of pure Mode I and pure Mode II failure modes (mixed-mode).

It was reported that the fracture energies were closely dependent with the initial crack length, and the crack length should be selected equal to or larger than the 70% of the half-length for stable crack propagation (Kageyama et al. 1991). Therefore, the crack length, \( a_c \), was set to 78% of the half-length, \( L/2 \), for all the SLB specimens.

The estimated phase angles was about 41° (degrees) for SLB specimens bonding similar substrates (da Silva et al. 2011). For dissimilar substrates, the phase angles can be estimated to be 46° and 37° (degrees) for the GFRP (top)/steel (bottom) and steel (top)/GFRP (bottom) SLB specimens.

Figure 4.6 (a) Selected element location inside the critical location in full beam model (b) FE epoxy submodel with a planar crack
specimen configurations (Davidson et al. 1996). In addition to the approximate phase angles from literature, the preliminary phase angle estimation using linear fracture mechanics in SLB FE model and accounting for the epoxy thickness showed that the estimated phase angles will be around $61^\circ$ and $24^\circ$ (degrees) for GFRP/steel and steel/GFRP SLB specimen configurations, respectively.

As will be seen later, the current mixed mode fracture analysis using refined epoxy submodel showed that the expected range of phase angle is between $59^\circ$ and $29^\circ$ for debonding at the GFRP-steel bonded interface. GFRP (top)/steel (bottom) SLB substrate configuration was selected for the SLB experiment in this section because pure Mode I fracture parameters were taken from uniaxial tension test results; hence, a fracture envelope can be created between pure mode I and mixed mode failures.

4.4.1 Extraction of Fracture Energies Using Theoretical Formulation

The governing equations for the SLB tests are found in the literature (Alfredsson 2004; Ouyang et al. 2009). Generic boundary conditions were assumed in the derivations of these equations. It should be noted that the equations were derived for bimaterial joints; however, the coupling condition given in Eq. (4.6) for the normal and tangential components of the generic equation can be uncoupled (Ouyang et al. 2009). In Eq. (4.6), $D$ is the bending stiffness per unit width and $t$ is the thickness of the substrates. Subscripts $t$ and $b$ denote top and bottom substrates respectively.

It is noted that the geometrical and mechanical properties of the steel and GFRP substrates given in Table 4.2 satisfy the uncoupling condition. The GFRP laminate and steel substrates were assumed to be linear elastic materials thus allowing the nonlinear properties of epoxy layer to be obtained with path independent $J$ integral (Rice 1968). The SLB specimens were monotonically loaded during the tests, and the $J$ integral was related to the critical fracture energies as given in Eqs. (4.7) and (4.8). The derived equations are for unit width specimens. The load, $P$, was applied at the mid-span, $L/2$, of the specimen. The crack tip location was fabricated away from the support at a crack length distance, $a_c$, as shown in Figure 4.7.
\[
\frac{D_t t_b}{D_b t_t} = 1 \quad (4.6)
\]

The previous formulation for Mode I traction-separation relation is given in Eq. (4.7) was path independent, and multiplication of force and rotation under the applied load gives Mode I fracture energy (Andersson et al. 2004). Once the original equation proposed by Andersson et al. (2004) for Mode I fracture is decomposed into its components, the rotation under the applied load can be rewritten as the summation of crack rotation, \( \theta \), and upper substrate rotation, \( \varphi \), at the crack tip because the lower substrates will not rotate following the crack tip rotation, rather it follows an unrestrained linear extension of crack the tip rotation. Therefore, the final form of Mode I traction-separation equation was obtained as given in Eq. (4.7). Similarly, the derivation of Mode II traction separation equations can be rewritten for SLB specimens with dissimilar material substrates (Ji et al. 2012). The final form of Mode II traction-separation equation is given in Eq. (4.8). In these equations, \( \delta \) is the normal and \( \xi \) is the tangential separations at the crack tip. It should be noted that plain strain formulation was used for all the SLB calculations presented later.

| Table 4.2 Material properties of GFRP and steel sections |
|-------------------|-------------------|-------------------|-------------------|
| Substrates        | GFRP              | Steel             |
| \( \mu \) (mean)  | 24.38 [3536]      | 390 [56.4]        | 218 [31,621]      | 382 [55.4] |
| \( C_V \) (%)     | 6.11              | 7.10              | 9.00              | 4.40       |

4.4.2 Material Properties

The substrates were bonded using two different epoxies, Tyfo® S and Tyfo® MB3. Both types are prepared by mixing two components following the manufacturer’s instructions. Tyfo® S is mainly used for generic strengthening applications, and Tyfo® MB3 is recommended for steel structures in structural bonded systems. The main difference between these two types is that even though the ultimate strength of Tyfo® MB3 is lower than that of Tyfo® S, it exhibits a more
ductile behavior than Tyfo® S epoxy, which is essentially a purely brittle material. Furthermore, Tyfo® MB3 is an extremely more viscous than Tyfo® S, which requires a different set of procedures for its application. The effect of the differences between these two adhesives on the behavior of SBS strengthened beams is discussed in detail elsewhere (Okeil et al. 2015).

The mechanical properties for GFRP substrates was determined experimentally following ASTM-D7264 (2007) “Standard Test Method for Flexural Properties of Polymer Matrix Composites” and ASTM-E8/04 (2004) “Standard Test Methods for Tension Testing of Metallic Materials” for the steel substrates. Table 4.2 summarizes the mechanical properties of the substrates that were used in the SLB experiments.

\[ G_I = \int_0^\delta \sigma(\delta)d\delta = -\frac{P}{4b}[\theta(a_c) + \varphi(a_c)] \]  
\[ G_{II} = \int_0^{\delta_0} \tau(\xi)d\xi = \frac{1}{2}\left(\frac{t_t}{2D_t}\frac{P_d c}{2} + \frac{(t_t + t_b)P}{4D_t + D_b}\right) \]  

\[ \frac{1}{A_t} + \frac{1}{A_b} + \frac{1}{4(D_t + D_b)} \]  

\[ \delta \]

\[ P, \Delta_{SLB} \]

\[ L/2 \]

\[ a_c \]

\[ E_t, t_t \]

\[ E_b, t_b \]

Figure 4.7 SLB test configuration

4.4.3 Experimental SLB Test Procedure

4.4.3.1 Specimen Preparation

The specimens were fabricated to have a unit inch width (25.4 mm). The clear span length, \( L \), of SLB substrates was set to 229 mm [9 inch]. The length of upper (GFRP) and lower (steel) specimens were 254 (10 in.) and 203 mm (8 in.), respectively. The thickness of the GFRP and steel substrates were 9.55 and 3.18 mm (3/8 and 1/8 in.), respectively. These thicknesses
were chosen to be the same as the GFRP stiffener’s flange thickness and the steel web plate thickness from the full SBS beam specimens. The surface of the substrates was roughened using sand paper. After revealing the white metal on the steel side, the surface was wiped with acetone to remove any debris before applying the adhesive and bonding the two substrates. SLB specimens were then bonded creating three different epoxy thicknesses between the substrates; namely 1, 2 and 3 mm. Thin metal spacers were placed between the substrates during bonding to achieve the desired thickness. One spacer was placed at the bonded end, and the other one was placed around mid-span. The two substrates were pressed towards each other with clamps at spacer locations. After the substrates were clamped firmly, the starter crack was created by inserting a 3mil (0.003 in.) thick Teflon sheet at the center height of the epoxy layers. Finally, the specimens were left to cure at room temperature for at least 3 weeks before the test. This procedure was repeated for each epoxy type. A total of 6 SLB specimens (2 epoxy types, 3 epoxy thicknesses) were tested with three-point loading test configuration.

After curing, the specimens were marked with four easily distinguishable colored markers on one of the crack tip sides of the specimens. The color code of the inserted markers was chosen to facilitate object tracking during post processing using digital image correlation techniques. One of two color markers was placed on the top substrate right above the crack tip, and the second one placed on the bottom substrate below the crack tip. The key point in this instrumentation was that the imaginary line connecting the centers of the inserted markers had to pass through the crack tip. Therefore, the relative movement of the objects’ center can be used to determine the normal and tangential separations of the crack tip.

In addition to the relative separations at the crack tip, the relative rotations of the substrates around the crack tip is needed to calculate traction separation curves. Two other sets of color markers were placed parallel to the first couple on the unbonded/open interface side. The distance between these two parallel couples was selected to be 2.54mm (0.1 in.). The relative linear rotation between the substrates was then calculated using trigonometrical relations.
In performing the digital image processing, a scaled grid paper was glued near the attached markers as a reference. The relative pixel separations in each direction can be converted to the numeric value using the grid scale. The inserted markers and scaled grid paper can be seen in Figure 4.8 (a).

4.4.3.2 Optical Data Capturing and SLB Tests

Obtaining the fracture energy properties using Eqs. (4.7) and (4.8) requires the determination of normal, $\delta$, and tangential, $\xi$, separations at the crack tip. These quantities were determined by processing optical images captured during the tests of the area around the crack tip. The macro images of the crack tip location were captured using a HD camera whose results appeared to be adequate for achieving the goal of this study as will be seen later. The camera was positioned about 10-15 mm away from the specimen, perpendicular to the crack tip side that has optical marks. The MTS 810 hydraulic testing machine which has 245 kN (55-kip) tension/compression capacity was used to load the specimens at a constant deflection rate of 4.4 and 8.5 \( \mu \text{m} \) per second for Tyfo\textsuperscript{®} and Tyfo\textsuperscript{®} MB3 epoxies, respectively.

![Image processing steps](image)

Figure 4.8 Image processing steps (a) raw image (b) contrast enhancement (c) binary image/thresholding (d) morphological operation
4.4.3.3 Image Processing

The digital images were processed in the MATLAB computation environment (image processing toolbox for 2D images) using code specially developed for this problem on. The main objective was to track the coordinates of installed marker’s center at a rate of one frame per second. First, the recorded video images were extracted, and stamped with the camera time. Then, selected images were subjected to contrast enhancement algorithms to sharpen the boundary of the installed markers. Then, a threshold of RGB values was manually assigned to determine the area of the markers within the frame, from which a raw binary image was obtained. Typically, there will be unwanted objects other than the intended markers in the raw binary image due to the fact that some RGB pixel values fall within the manually set thresholds. These objects need to be eliminated from the image before tracking. The elimination process can be completed before or after the extracting the markers’ properties. In our study, the first option was selected to avoid having to process the irrelevant data, which may require manual intervention. Therefore, the following step in the image processing algorithm was to filter these unwanted pixels in the raw binary image file using morphological operations. Finally, the raw binary image frame was processed to obtain only the centroids of the four identified areas representing the installed markers. The same steps were repeated for one selected frame per second. The centroids of the markers and the corresponding frame time were recorded for all the images. As will be explained later, these time stamps were mapped to the load history from the MTS testing machine to find the load magnitude corresponding to each frame. Figure 4.8 shows a typical captured optical image before and after contrast enhancement and the subsequent RGB thresholding and the morphological operation to eliminate unintended pixels from the raw binary image.

Initial time lag between the actual loading of the specimen (e.g. due to initial gaps in the test setup) and onset of image capturing time was recorded and subtracted from the corresponding frame time during the image processing. Once the real loading time and captured image time were matched, MTS load data was mapped to image data using linear interpolation.
The captured image data includes coordinate changes of the installed markers for each frame due to the separations. However, this change also includes variations because of the change of RGB pixel values during the test. The first reason for the unwanted noise was that the boundary of the inserted object were sensitive to the surrounding light sources. A small change in an RGB value of a pixel determines whether that pixel will be filtered or not based on the predefined RGB threshold value. The second reason was the morphological operations can cause additional noise in the boundary of the objects. In this study, the relative separations were estimated using best fit polynomial functions before they were used in the calculations. The selected polynomial functions and the raw data from digital image processing will be shown later for crack tip normal and tangential separation.

4.5 Phase Angle Results

The change in phase angle for different epoxy thicknesses, element locations, steel plate thicknesses, and crack size will be presented in this section. The phase angle will be presented in degree (°) units. According to Eq. (4.3), a phase angle equal to 0° corresponds to pure Mode I, while a 90° degree phase angle corresponds to pure Mode II failure. Any phase angle in between these two angles is defined as a mixed mode.

4.5.1 Effect of Epoxy Thickness

The effect of epoxy thickness on the load-displacement history of the full beam FE model was studied assuming three different epoxy thicknesses, 2.5, 3.0, and 3.5 mm with one web plate thickness equal to 3.18 mm [1/8 in.]. The results did not show any significant change in the load-displacement histories for different epoxy thicknesses. Displacement fields from these full beam FE analyses were mapped to the submodel to study whether epoxy thickness has a different effect locally on the phase angle. It was found that the maximum phase angle difference between the three considered epoxy thicknesses was 1.51% around peak load level. Therefore, one epoxy thickness (3 mm) was taken in the refined epoxy submodel studies.
4.5.2 Effect of Element Location

As stated earlier, three different element locations were studied in the bonded region subjected to the largest web deformations to compare the location effects on the phase angles. The thickness of steel web and epoxy thickness were assumed 3.18 mm (1/8 in.) and 3 mm (0.118 in.), respectively. The thickness of the GFRP section was 9.53 mm (3/8 in.) and was kept unchanged during phase angle investigation. The length of the planar crack was 1.27 mm (0.05 in.) for each location. The three selected locations were named top, middle, and bottom elements as can be seen in Figure 4.4. The phase angle change for each load step is shown in Figure 4.9, where the secondary ordinate (y) axis shows the load deflection history of the full scale beam.

It is clear from Figure 4.9 that the phase angle was about 90°; i.e. pure Mode II, in the linear loading range for all three locations. During this stage, the plate does not bear more than its buckling load and there is no tendency to buckle out of plane. Once the web reaches its buckling load limit and beyond, the phase angle starts to drop to 45°-50° indicating a clear mixed-mode condition. This is due to the fact that as the web plate buckles out of plane away from the epoxy and the GFRP stiffener, tensile stresses start acting on the interfaces. Thus, the dominant Mode II; i.e. pure sliding shear, shifts towards mixed mode. All three different

![Figure 4.9 Phase angles for 3 locations as the girder was loaded](image-url)
elements exhibited a similar descending phase angle change that stabilized at a value ranging between 45°-50°.

As a result, the phase angle investigation can be reduced to one single element, which was chosen to be the one subjected to the maximum out of plane web deformation. Therefore, the results presented next are for only one location which was the middle epoxy element.

4.5.3 Effect of Web Thickness

The effect of web thickness on fracture properties was investigated during the linear and post buckling stages using displacement fields from full beam FE models with four different web thicknesses, 3.18, 3.97, 4.76, and 6.35 mm (4/32, 5/32, 6/32, and 8/32 in.). These displacement fields were mapped to the submodel as explained earlier. The epoxy layer was assumed to be 3 mm-thick in this study, and the maximum deflection was limited to the 12.7 mm (0.5 in.) under the applied load. The length of the planar crack was assumed to be 1.27 mm (0.05 in.).

The load-deflection curves for the four beams with different web thicknesses are shown in Figure 4.10. The full beam models with perfectly bonded GFRP/steel interface demonstrated a nonlinear behavior starting with linear segment prior to buckling.

![Figure 4.10 Load deflection history with different web thicknesses](image-url)
The relationship between phase angles and load history for these web thicknesses is plotted in Figure 4.11. As before, the phase angle prior to buckling showed pure Mode II behavior, while a clear mixed mode behavior was determined for post buckling. The shift in phase angles is more obvious for the beams that have thinner web plates, which are more susceptible to web buckling causing an increase in Mode I, and hence the clearer mixed mode behavior. For the thickest web, the phase angle did not show any change and almost stayed in 90° phase angle indicating that buckling is not a major issue; i.e., failure is caused by full plasticization of the web or flange buckling.

It should be noted that the investigated epoxy elements were selected on the concave side of the buckled web plates. The beginning of the nonlinear behavior in the plots shown in Figure 4.10 corresponds to initiation of web buckling or yielding. Therefore, the shifts in the phase angle became more pronounced as the outward deformation of the web panels increased. The outward displacement of the web panels at the critical region versus the vertical deflection under the load for different web thicknesses is shown in Figure 4.12. As expected, the thinnest web panel showed the earliest buckling sign. In other words, the crack front stress field was shifted
from pure shear stresses to mixed shear-tension stresses. It was concluded that the buckled panel forced the failure mechanism of the epoxy element closer towards pure Mode I delamination. The cases with thicknesses of the mid-range web panels; i.e., 3.97 mm (5/32 in.), 4.76 mm (6/32 in.), showed similar phase angle shift following panel buckling order. As can be seen in Figure 4.11, the phase angles eventually stabilized at about 45° mixed mode phase angle at the end of the investigated deflection limit of 12.7 mm (0.5 in.). Finally, the thickest web panel; i.e., 6.35 mm (8/32 in.), did not experience significant out of plane buckling before the deflection limit, 12.7 mm (0.5 in.) was reached. Consequently, there was not a clear phase angle shift determined from the epoxy submodel.

4.5.4 Effect of Crack Length

The effect of different crack lengths on the phase angle was investigated using the 3.18 mm (4/32 in.) web panel thickness and 3 mm epoxy thickness. Five different crack lengths; 0.3175, 0.635, 1.270, 2.540, and 3.810 mm (0.0125, 0.025, 0.05, 0.1, and 0.15 in.) were analyzed using the epoxy submodel. Since the web panel and epoxy thickness were not changed, the same displacement field was imposed on all the submodels with different crack lengths.

![Figure 4.12 Web plate's lateral deflection at critical location](image)
The crack length effect on the phase angle variation is shown in Figure 4.13. Web panel buckling was the main reason for the shift in phase angle from pure Mode II towards a mixed mode behavior. The phase angles stabilized at different levels based on the crack size. The smaller crack size showed a phase angle of around 29° at the predefined deflection limit. As the crack size increased, the final stabilized phase angle also increased, resulting in a Mode II dominant mixed mode behavior. For the largest crack length 3.810 mm (0.15 in.), the phase angle stabilized around 59°.

4.5.5 Effect of Epoxy Type

The GFRP stiffeners were bonded to the web panel using two different epoxy types, and their experimentally determined mechanical properties were used to investigate phase angle change using the same epoxy submodel for three different crack sizes, 1.27, 2.54, and 3.81 mm (0.05, 0.10, and 0.15 in.). All the previously presented results were calculated for Tyfo® S epoxy properties; the generic adhesive that is often used in strengthening applications. In this section, the phase angle change was studied for two different epoxy types using the beam with web panel thickness equal to 3.18 mm (4/32 in.) and a 3 mm epoxy thicknesses.

![Figure 4.13 Phase angle variation with different crack length, \( a_c \)](image-url)
Both epoxy types showed a phase angle shift with initiation of web buckling. The difference between the phase angles was 7°, 11° and 13° for 1.270, 2.540, and 3.810 mm (0.05, 0.10, and 0.15 in.) crack lengths, respectively. Overall, Tyfo® MB3 showed slightly lagging phase angle shift after yielding but at later loading stage it showed more dominant Mode I failure when compared with Tyfo® S as can be seen in Figure 4.14.

The experimental test observations showed that delamination failure at the GFRP/steel interface can initiate after web panel yielding. The mechanical properties of the bonding material plays an important role in the fracture mechanism. If the epoxy material is brittle, the failure may occur after the web panel experiences minor buckling. At this stage, it can be said that the behavior is a Mode II dominant mixed mode. On the other side, if the epoxy material is ductile (e.g. Tyfo® MB3), elongation of the adhesive allows for stress redistribution and delays cracking, which allows for web panel buckling to proceed beyond what has been typically observed in the case of brittle adhesives. Consequently, the failure becomes Mode I dominant mixed mode failure because the buckled web plate in post buckling loading will cause peeling type failure on the concave side of the buckled web panel.

Figure 4.14 Effect of different epoxy type on the phase angle

![Phase angle vs. beam deflection graph](image_url)
4.6 SLB Test Results

4.6.1 Image Data

The relative displacements in normal and tangential directions, and relative rotation between upper and lower substrates at the crack tip were collected from digital images and analyzed using image processing techniques. Relative separations in normal and tangential directions are shown in Figure 4.15 for 1 mm-thick Tyfo® S epoxy. A polynomial function was fitted to the raw image data points. The separations were assumed to follow the polynomial functions in subsequent calculations. Similarly, the relative rotation between the substrates that were obtained from digital image processing was approximated using polynomial functions. Figure 4.16 shows a typical captured image rotation and the fitted polynomial function for 1 mm-thick Tyfo® S epoxy. Image results from polynomial data, and corresponding MTS load data were used to calculate normal and tangential fracture energies, $G_I$ and $G_{II}$, respectively.

4.6.2 Fracture Properties of Epoxies Obtained from SLB Test

The fracture energy curves were obtained by fitting polynomial functions to the equations (4.7) and (4.8) results. Figure 4.17 shows the normal and tangential fracture energies and their
fitting polynomial functions. Traction separation curves were then obtained by finding the derivative of the fracture energy function with respect to the separation. Normal and tangential stress-separation curves were obtained for both epoxy types. The normal and tangential separation curves for Tyfo® S and Tyfo® MB3 epoxies are shown in Figure 4.18 and Figure 4.20. As can be seen, the total fracture energies, peak stresses, and separation in both delamination directions increased as the thickness of epoxy layers increased. Another observation was that the normal peak stresses showed an increase from 1 to 3 mm epoxy thickness; 3.7 MPa and 4.8 MPa for Tyfo® S and Tyfo® MB3 epoxies, respectively. However, the tangential peak stresses gained only 1.3 and 2.4 MPa for these epoxies, respectively. The dashed lines in Figure 4.18 and Figure 4.20 represents the idealized bilinear CZM behavior that is recommended for detailed FE models in future studies. The average of the areas under the traction separation curves were chosen to be equivalent to the triangular area under the CZM. In other words, CZM behavior was obtained by conserving the average normal and tangential fracture energies and peak stresses. The maximum separation was then obtained by equating the fracture energies.

Figure 4.16 Relative substrate rotation around crack tip (Tyfo® S 1mm)
In the following discussions, the results will be presented as an average of the behavior of the 1, 2 and 3mm epoxy thicknesses to draw more broad traction separation law sample which can be investigated with parametric studies for a CZM applications. Therefore, the combined properties of three different epoxy thicknesses were treated as one thick epoxy layer representing a typical application with varying adhesive thickness due to field conditions such as initial distortion or non-uniform corrosion. Table 4.3 summarizes the total fracture energies, maximum stresses, and maximum separation for the normal and tangential constitutive relationship for each epoxy type. The total fracture energy, $G_I + G_{II}$, of Tyfo® MB3 was determined to be 2 times higher than the total fracture energy Tyfo® S. Even though there was not a noticeable increase at maximum peak stresses with Tyfo® MB3 epoxy, the significant increase in the total energy difference was due to the higher elongation ability of Tyfo® MB3 epoxy which was about 1.7 times higher than Tyfo® S’s elongation. Experimental fracture energies showed that mixed mode phase angle for SLB experiment was 51 and 46° for Tyfo® S and Tyfo® MB3 epoxies. It should be noted that the phase angle was determined to be 61° for GFRP/steel specimen configuration using FE model of SLB configuration. While this is the same trend obtained numerically using

![Figure 4.17 Mode I ($G_I$) and Mode II ($G_{II}$) fracture energies (Tyfo® S 1mm)](image-url)
the FE epoxy submodel, it is clear that the experimentally observed difference between the phase angles from the SLB tests is less.

Finally, the crack initiated at the preformed crack tip and propagated from crack tip to GFRP substrate diagonally in a brittle manner for specimens bonded using Tyfo® S epoxy. The separation continued at the GFRP interface as the load was increased. Similar failure behavior was observed with Tyfo® MB3 epoxy, however, rather than a clear discrete crack propagation, there was a noticeable color contrast change around the crack tip before fracture. The progressive evolution of the crack tip opening for Tyfo® MB3 specimen can be seen in Figure 4.21.

### 4.6.3 FE Simulation of SLB specimens

To test the performance of the recommended CZM properties given in Table 4.3 in a FE model, they were used as an input for to model the tested SLB specimen for the purpose of simulating load-deflection plots and crack propagations. The SLB FE model was built using 2D PLANE182 elements from the ANSYS element library with plane strain formulation for the substrates. The FE model of the SLB specimen is shown in Figure 4.19.

<table>
<thead>
<tr>
<th>Epoxy Type</th>
<th>$G_{II}$ (kN/m)</th>
<th>$G_{II}$ (kN/m)</th>
<th>$\sigma_{avg}$ (MPa)</th>
<th>$\delta_{avg}$ (mm)</th>
<th>$\tau_{avg}$ (MPa)</th>
<th>$\xi_{avg}$ (mm)</th>
<th>$\Psi$ (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tyfo® S</td>
<td>0.19</td>
<td>0.24</td>
<td>15.8</td>
<td>0.0243</td>
<td>16.9</td>
<td>0.0280</td>
<td>51</td>
</tr>
<tr>
<td>Tyfo® MB3</td>
<td>0.40</td>
<td>0.42</td>
<td>17.3</td>
<td>0.0465</td>
<td>18.9</td>
<td>0.0444</td>
<td>46</td>
</tr>
</tbody>
</table>

Figure 4.18 Normal and tangential stress-separation curves of Tyfo® S with customized CZM
The adhesive was modeled using two approaches. In the first approach, INTER202 element, which is a cohesive zone material element, was used, while in the second approach CONT171/TARG169 elements technologies were used to simulate crack propagation. Experimentally obtained MTS load-deflection curves for three SLB specimens are plotted in Figure 4.22 for Tyfo® S and Tyfo® MB3 epoxies. Delamination simulation of the SLB FE models are plotted with the dashed lines along with the experimental results in the same figure. There was not significant difference between the peak loads at start the crack propagation. The average loads before crack propagation were obtained to be 0.70 and 0.91 kN from the experiments for Tyfo® S and Tyfo® MB3 epoxies, respectively. The maximum error between the results obtained from FE simulation using contact elements and the experimental results was 6.6% for Tyfo® MB3 epoxy. It was also noted that the FE plots showed slightly stiffer behavior in linear loading before crack propagation; however, the post crack stiffness showed similarity with experimental results. Overall, the load at initial crack propagation was accurately estimated with FE model simulations using experimentally obtained CZM material properties. The
averaged results of SLB experiments and FE simulations using interface (INTER202) and contact (CONT171/TARG169) elements are given in Table 4.4.

4.7 Conclusion

Fracture properties of adhesives employed in retrofitting steel girders using the SBS technique where GFRP stiffeners are bonded to deficient web plates was investigated using numerical FE models and experimental testing. The maximum out of plane deformation in the web due to buckling caused delamination of the epoxy adhesive on concave side of the web plate. Numerically, a full finite element model of the SBS-strengthened steel beams was constructed to determine the displacement field at the critical epoxy elements in failure region. A planar crack was introduced in an epoxy submodel to capture the phase angle variations using different epoxy types, epoxy thicknesses, epoxy locations, web thickness, and crack length.

Based on this part of the study, it was found that:

<table>
<thead>
<tr>
<th>SLB</th>
<th>Tyfo® S P (kN [lbs])</th>
<th>∆SLB (mm [in.])</th>
<th>Tyfo® MB3 P (kN [lbs])</th>
<th>∆SLB (mm [in.])</th>
</tr>
</thead>
<tbody>
<tr>
<td>EXP (average)</td>
<td>0.70 [157]</td>
<td>2.09 [0.082]</td>
<td>0.91 [205]</td>
<td>3.15 [0.124]</td>
</tr>
<tr>
<td>FE (INTER202)</td>
<td>0.70 [157]</td>
<td>1.75 [0.069]</td>
<td>0.88 [197]</td>
<td>2.12 [0.083]</td>
</tr>
<tr>
<td>FE (CONT171/TARG169)</td>
<td>0.69 [155]</td>
<td>1.64 [0.065]</td>
<td>0.85 [191]</td>
<td>2.02 [0.080]</td>
</tr>
</tbody>
</table>

Figure 4.21 Crack tip opening at different loads
In general, there is a consistent fracture mode shift from Mode II dominant behavior prior to the buckling in the initial loading stages to mixed mode behavior once buckling ensues.

1. The considered epoxy thicknesses (2.5, 3.0, and 3.5 mm), and epoxy types did not show significant effect on mixed mode failure and phase angle.

2. The size of the crack length had significant effect on the phase angle. The mixed mode failure phase angle varied between $59^\circ$ to $29^\circ$ for 0.3175 and 3.810 mm (0.0125 and 0.15 in.) crack lengths.

Based on these results, it can be concluded that the buckling of the steel plate causes peeling type of failure with the progression of loading as Mode I becomes more dominant on the behavior. The peeling effect is more dominant in beams with thinner web plates and smaller crack sizes. Different sizes and forms of cracks may exist in epoxy layers in real applications. However, in general, cracks usually start as small anomalies that grow with time, which explains the experimentally observed mode of failure in epoxy layer or at interface of epoxy and substrate that is often reported in the literature.

To extend the mixed mode phase angle study, local fracture parameters for two different epoxy types were experimentally investigated using SLB specimens with GFRP/steel substrates. Three different epoxy thicknesses were considered (1, 2, and 3mm). Digital image correlation (DIC) techniques were used to capture relative normal and tangential separations, and rotations at the crack tip. From these results, traction-separation laws for the given epoxies were extracted.

Figure 4.22 Load deflection curves of SLB experiments with FE simulation using contact and interface element
using theoretical formulations. The averaged fracture energies and peak stress values of three epoxy thicknesses were then idealized for future use as a bilinear CZM material in FE simulations. From the experimental results it was concluded that:

1. The mixed mode phase angles at failure and the total fracture energies were obtained 51° and 46° degrees and 0.43 and 0.83 kN/m for Tyfo® S and Tyfo® MB3 epoxies, respectively.
2. Tyfo® MB3 showed about twice the fracture energy, and 1.7 times separation in normal and tangential directions compare to Tyfo® S. The increase in fracture energy is a result of the higher elongation ability of Tyfo® MB3 epoxy, which exhibits a ductile behavior with a yield-like plateau.
3. The increase in epoxy thickness caused slightly higher normal fracture energies but almost did not have any effect on tangential fracture energy.

Finally, the crack propagation in SLB specimen was simulated with a finite model using experimentally obtained CZM properties. Two different CZM delamination methods were considered by incorporating interface and contact element technologies. The initial crack propagation in FE simulation started slightly earlier than in the experiment; however, the simulated peak load and post peak behavior were in good agreement with experimental results. The average experimental peak load was 0.70 and 0.91 kN, whereas 0.70 and 0.87 kN were obtained using FE interface element simulations.

Thus, the idealized fracture mechanism and properties of the specific epoxy types will provide valuable information to simulate the delamination failures of full beam model or any similar bonding applications.

4.8 Nomenclature

\[ a_c', a_c \]  \quad \text{Crack length in submodel and SLB specimen}

\[ A_t, A_b \]  \quad \text{Axial stiffness of top and bottom substrates for plain strain}

\[ b \]  \quad \text{Plate width}

\[ C_V \]  \quad \text{Coefficient of variation}
D_t, D_b  Bending stiffness of top and bottom substrates for plain strain
E  Elastic modulus
E_c, E_b, E_t  Elastic modulus of epoxy, bottom and top substrates
G  Shear modulus
G_I, G_II  Mode I and II fracture energies
I  Interaction integral value
K_I, K_II, K_III  Stress intensity factors for Mode I, II and III fractures
K_I^{aux}, K_II^{aux}, K_III^{aux}  Auxiliary stress intensity factors for Mode I, II and III fractures
L  SLB specimen length
P  Applied load at mid-length of SLB specimen
SD  Standard deviation
q_i  Crack extension vector
t_c, t_b, t_t  Thickness of epoxy, bottom and top substrates
δ, ξ  Normal and tangential separations
δ_z  Web panel lateral deflection
Δ_{SLB}  Deflection at mid-length of SLB specimen
θ, φ  Relative substrate and crack tip rotation
μ  Mean of sample
σ_{kl}, ε_{kl}, u_{k,i}  Stress, strain, displacement
σ_{kl}^{aux}, ε_{kl}^{aux}, u_{k,i}^{aux}  Auxiliary stress, strain, displacement
σ, τ  Normal and shear stresses
Ψ  Phase angle

4.9 References


ANSYS "ANSYS ® Academic Research, Release 16."


Chapter 5. Numerical Analysis of SBS Retrofitted Beams and Design Considerations

5.1 Introduction

Rehabilitation and retrofitting methods offer economical and feasible alternatives for upgrading aged and deficient structures to reach original or higher load capacities. Structural strengthening has been gaining attention because it can extend the service life of an existing structure with relatively limited capital investment and service interruptions. This is especially true for strengthening using composite materials. Fiber reinforced polymer (FRP) composites are widely used in research and in strengthening applications of deficient structures. Typically, FRP materials are employed in the form of laminates, sheets or strips. The mechanical properties of FRPs compared to those of traditional structural materials are appealing because they offer high corrosion and fatigue resistance in addition to their lighter weight, which makes their installation easier.

The majority of research efforts on strengthening of deficient structures using composite materials focused on concrete and masonry structures due to their elastic modulus, which is lower than that of composite materials. Successful applications in literature showed that FRPs are efficient for retrofitting relatively low modulus materials (ACI-440.2R 2008; Ulger et al. 2016). Planar composite materials (e.g. sheet, laminate or plate) are typically bonded to the deficient regions where additional tensile forces are needed to increase the member’s flexure, shear or axial resistance. For the latter, indirect tensile capacity in the hoop direction improves core confinement; leading to the higher capacity. Design codes and guidelines for strengthening concrete structures have been published (ACI-440.2R 2008), which is a sign of the level of maturity of these methods and the confidence the engineering community has in them. A study of these documents reveals that the use of composites in strengthening applications has relied on the in-plane strength of the fibers to increase the load capacity of the structures.

The same retrofitting concept, in-plane resistance contribution of FRP fibers, has been implemented for strengthening steel structures; however, as a result of the higher elastic modulus
of steel, a similar outcome as that obtained from retrofitting concrete or masonry structures was reached only when larger amounts of FRPs (Sen et al. 2001) was used. To improve the performance of FRP-strengthened steel structures, the use of high or ultra-high modulus (HM or UHM) FRPs was recommended (Schnerch 2007). Several researchers investigated bonding carbon FRPs (CFRP), HM or UHM-CFRPs to the tension flanges of the steel girders (Al-Saidy et al. 2007; Kim et al. 2012; Miller 2001; Patnaik et al. 2008; Photiou et al. 2006; Sen et al. 2001). Anchoring CFRP laminates to the ends of the tension flange with and without bonding CFRP to the flange was also investigated (Galal et al. 2012). In other retrofitting examples, axial members and joints connecting two or more axial members working under tension or compression loads were retrofitted by wrapping FRPs around the steel sections (Fam et al. 2006; Gao et al. 2013; Xiao et al. 2012). The shear load bearing sections were also strengthened by bonding thin sheets to the web panels of the steel beams to enhance shear capacity of the steel beams (Narmashiri et al. 2010; Okuyama 2012; Patnaik et al. 2008). One can conclude that the concept of utilizing the in-plane strength of thin FRP products has been also applied to the steel structures but not as extensively as concrete or masonry structures. High modulus FRPs (Carbon FRP or CFRP, HM and UHM CFRP) are more dominant retrofitting materials in literature compared to other FRP products; e.g., Glass FRP (GFRP), because of its higher mechanical properties.

In this work, we propose a new strengthening method, *Strengthening by Stiffening (SBS)*, which utilizes the relatively cheaper GFRP material in the form of pultruded GFRP sections to strengthen buckling prone steel structures. One specific use of SBS method was investigated experimentally by bonding pultruded T-shaped GFRP stiffeners to shear deficient web panels of steel beams for enhancing the performance and capacity (Okeil et al. 2009b; Ulger et al. 2016). A schematic illustration of the proposed method is shown in Figure 5.1. Contrary to the aforementioned retrofitting concept where the in-plane strength of the composite fibers is the main contributor to the strength enhancement, the out-of-plane resistance of the GFRP stiffeners resulting from their geometrical properties contributes to the overall strength of the retrofitted
section. Therefore, the SBS method does not solely depend on the uniaxial resistance of fibers within the matrix of the pultruded stiffener. Consequently, the commonly preferred composites (i.e. CFRP, HM or UHM CFRP) can be replaced with less expensive fibers (i.e. GFRP) in manufacturing the pultruded sections.

In this paper, the SBS method was numerically investigated by constructing a finite element (FE) model in the commercial software ANSYS. The developed model was validated using results from experiments conducted by the authors (Okeil et al. 2009b; Ulger et al. 2016). The failure of adhesive layer was modeled by implementing an interlaminar cohesive zone model (CZM) to simulate debonding failure. Delamination failure within the FRP stiffener was defined by Hashin’s failure criteria for layered composites in the FE model. The mechanical properties of the CZM model were based on mixed mode fracture experimental tests conducted by the authors in Chapter 4. The SBS method’s ability to enhance the shear behavior of a built-up steel I-section divided into equal length shear deficient web panels was investigated. The behavior of the shear deficient beams under critical shear loading were obtained experimentally and simulated numerically using the FE model. The calibrated model was then used to investigate the effective parameters in SBS design.

![Figure 5.1 (a) Schematic of SBS method showing T-shaped FRP stiffener (b) stiffness contribution of FRP](image)
5.2 Experimental Investigation

5.2.1 Test Set-up and Specimens

The goal of the experimental program is to study the shear capacity enhancement of built-up I-shaped steel beams using the proposed SBS retrofitting method. The beams were fabricated with welded transverse steel stiffeners to create web panels. Two different panel aspect ratios (panel length to panel height \(l_p/h_p\)), 1.0:1.0 and 1.5:1.0, were considered in the experimental program. An asymmetric three-point loading setup was used for the beam tests, and the load was applied on the first intermediate steel stiffener to create a critical shear failure zone. Details of the tested beams are given in Table 5.1. The beams are grouped in two categories based on panel aspect ratios (1.0:1.0 and 1.5:1.0); i.e., having a square panel (SP) or a rectangular web panel (RP). Each group was then sub-grouped based on web panel thickness; namely SP1, SP2, RP1 and RP2. The retrofitting scheme and contact area for the tested beams are provided in the ‘stiffener’ and ‘contact’ columns in Table 5.1. The proposed SBS method employed two different GFRP stiffener configurations in the shear critical panel; namely in the first configuration one GFRP stiffener was bonded on one side of the web panel and two stiffeners were bonded opposite each other; one on each side of the web panel in the second configuration. The prefix number 1 or 2 in the ‘stiffener’ column represents the number of stiffeners in retrofitted web panel. The GFRP stiffeners were extended between the weld toes of the flanges to cover the full web height. The contact area of the GFRP section can be defined as the ratio of GFRP stiffener’s flange width to the web panel length \(f_w/l_p\) as they both share the same height. The contact ratio, \(A_r\), web thickness, and panel dimension of the retrofitted girders are given in Table 5.1. The size of the GFRP stiffeners used in SBS retrofitting were 152 x 152 x 9.53 mm [6 x 6 x 3/8 in.] and 76 x 152 x 9.53 mm [3 x 6 x 3/8 in.].

The three point loading configuration and the beams’ dimensions can be seen in Figure 5.2. In the figure, section A-A show the steel stiffener dimensions, and section B-B shows the dimensions of the GFRP stiffeners used for SBS retrofitting.
Another retrofitting method involving composite materials is one where bi-axial CFRP fabrics are bonded to the web panel. This configuration was also investigated experimentally in order to compare the performance of SBS retrofittting with an alternative method. The CFRP fabric sheets were cut to 508 x 762 mm [20 x 30 in.] dimensions to cover the entire web panel. They were then saturated with Tyfo® S epoxy before bonding them to both sides of the shear critical web panel using Tyfo® MB3 epoxy. The number of orthogonal fibers, which were aligned horizontally and vertically (0°/90°), in the used bi-axial CFRP sheets were identical. The web panel retrofitted bonding CFRP sheet can be seen in section C-C of Figure 5.2.

Finally, one beam was retrofitted using the traditional approach of welding additional steel stiffeners to both sides of the critical web panel. The purpose of this specimen was to offer a reference by which the performance of SBS can be evaluated. Like in the beams retrofitted with SBS, the steel stiffeners were welded at mid-length of the web panel. The dimension of the steel stiffeners, 114 x 508 x 9.53 mm [4 1/2 x 20 x 3/8 in.], were identical to the transverse steel stiffeners dividing the web panels, and they were fully welded to web panel and flanges.

<table>
<thead>
<tr>
<th>Beam</th>
<th>Stiffener</th>
<th>Contact type</th>
<th>Contact ratio $A_r$</th>
<th>Web thickness $t_w$ (inch)</th>
<th>Panels</th>
</tr>
</thead>
<tbody>
<tr>
<td>SP1</td>
<td>0</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>20x20.5x4</td>
</tr>
<tr>
<td></td>
<td>2 GFRP$^E$</td>
<td>Tyfo® S</td>
<td>0.30</td>
<td>0.1205</td>
<td>20x20.5x4</td>
</tr>
<tr>
<td></td>
<td>2 GFRP$^{E,1}$</td>
<td>Tyfo® S</td>
<td>0.30</td>
<td>0.1205</td>
<td>20x20.5x4</td>
</tr>
<tr>
<td>SP2</td>
<td>0</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>19x20.5x4</td>
</tr>
<tr>
<td></td>
<td>2 GFRP$^E$</td>
<td>Tyfo® S</td>
<td>0.15</td>
<td>0.1154</td>
<td>19x20.5x4</td>
</tr>
<tr>
<td></td>
<td>1 GFRP$^E$</td>
<td>Tyfo® S</td>
<td>0.15</td>
<td>0.1154</td>
<td>19x20.5x4</td>
</tr>
<tr>
<td>RP1</td>
<td>0</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>19x30.5x3</td>
</tr>
<tr>
<td></td>
<td>2 GFRP$^P$</td>
<td>Tyfo® MB3</td>
<td>0.20</td>
<td>0.1305</td>
<td>19x30.5x3</td>
</tr>
<tr>
<td></td>
<td>2 GFRP$^P$</td>
<td>Tyfo® S</td>
<td>0.20</td>
<td>0.1305</td>
<td>19x30.5x3</td>
</tr>
<tr>
<td>RP2</td>
<td>0</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>19x30.5x3</td>
</tr>
<tr>
<td></td>
<td>2 STS</td>
<td>Welded</td>
<td>1.00</td>
<td>0.1142</td>
<td>19x30.5x3</td>
</tr>
<tr>
<td></td>
<td>2 CFRP</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>1 GFRP$^P$</td>
<td>Tyfo® MB3</td>
<td>0.20</td>
<td>0.1142</td>
<td>19x30.5x3</td>
</tr>
<tr>
<td></td>
<td>2 GFRP$^P$</td>
<td>Tyfo® S</td>
<td>0.20</td>
<td>0.1142</td>
<td>19x30.5x3</td>
</tr>
<tr>
<td></td>
<td>1 GFRP$^P$</td>
<td>Tyfo® S</td>
<td>0.20</td>
<td>0.1142</td>
<td>19x30.5x3</td>
</tr>
</tbody>
</table>

$^E$ Extern®, $^P$ PROFom®, $^1$ Diagonal stiffener orientation
In all, fourteen beams were tested to investigate different behavioral aspects. More details about the experimental program can be found elsewhere (Okeil et al. 2009a; Okeil et al. 2015; Ulger et al. 2016). The experimental results from these tests will be used to validate the developed FE model.

5.3 Material Properties

5.3.1 Steel

The mechanical properties of the A36 steel used in fabricating the beam specimens were determined by conducting uniaxial tension tests following ASTM E8-04. The experimentally obtained engineering material properties were converted to the true material properties using logarithmic stress strain relation given in Eq. (5.1) where $\sigma_{tr}$ and $\sigma_{en}$ are true and engineering stresses, and $\varepsilon_{tr}$ and $\varepsilon_{en}$ are true and engineering strains, respectively. A multi linear true stress-strain relation was defined for the FE model to simulate the non-linear material behavior of steel beam’s web, flanges, and stiffeners.

![Figure 5.2 Eccentric three-point load test configuration with (a) steel stiffener, (b) SBS and (c) CFRP sheet retrofitting](image-url)
Two epoxy types were used to bond the GFRP stiffeners in SBS retrofitted specimens, namely Tyfo® S and Tyfo® MB3. The first epoxy type, Tyfo® S, is a general purpose epoxy whose behavior is brittle. The other epoxy type, Tyfo® MB3, has a tolerance for high elongation prior to rupture and is promoted mainly for steel bonding applications. In SBS, the epoxy layer is subjected to complex stress-strain field at the steel / GFRP stiffener interface. Therefore, mixed mode fracture material properties was needed in order to simulate adhesive debonding in the FE model. The fracture energy of both epoxy materials under mixed mode stress field was studied by conducting single bending specimen (SLB) tests bonding two different substrates, GFRP (top substrate) /steel (bottom substrate) (Chapter 4). Digital image correlation (DIC) techniques were used to capture relative normal and tangential separations, and rotations at the crack tip. Traction-separation laws for the given epoxies were then extracted using theoretical formulations (Chapter 4). The averaged fracture energies and peak stress values were reported for an average epoxy thickness of 2 mm, which is close to the average epoxy thickness obtained (2.5 mm) in SBS retrofitted beams (Okeil et al. 2015).

A bilinear cohesive zone model (CZM) was defined in the FE model to simulate the steel / GFRP stiffener interface behavior using interface elements, INTER200. The generic bilinear CZM material model is shown in Figure 5.3 and the traction separation results are tabulated in Table 5.2.

5.3.3 Composites

The T-shaped stiffeners used in SBS retrofitting were cut from commercially available wide flange (WF) pultruded GFRP sections. Two products were used in the experimental program, namely Extern® and PROForm®. One of the flanges of the WF sections was cut to
obtain the T-shaped stiffener. The flexural properties for the pultruded sections were obtained experimentally following ASTM-D7264 (2007) and are given in Table 5.3. It was possible to simplify the FE model by not having to simulate the GFRP stiffener failure for the experimentally tested beams as no delamination of the GFRP stiffener was observed during the tests. This allowed for saving valuable run time and memory space during the FE simulations reported in this paper for validation and for the parametric study runs. Nevertheless, in a complementary investigation exploring the use of SBS in new construction, the FE model was modified to account for the possibility of delamination within the GFRP stiffener material itself following Hashin’s failure criteria for laminated composites. The GFRP section and matrix resin properties for GFRP stiffener were obtained from the manufacturer’s design guide and literature (M. Davallo M. 2010). PROForm® WF beam properties were used in all further parametric transverse weld free (TWF) bonded stiffener studies.

Coupons from the bi-axial (0°/90°) CFRP sheet used for strengthening Specimen RP2-2CFRP-MB3 were also tested to determine its mechanical properties following ASTM-D3039/D3039M (2014). The number of yarns in both directions was identical, therefore, the same material properties were applied to both directions. The uniaxial material properties of CFRP coupons are given in Table 5.3. It should be noted that since the thickness of saturated FRP resins usually vary, the results are provided per unit width.
Experimentally tested beam specimens were modeled in ANSYS (ANSYS) to validate the FE model. The steel and pultruded GFRP sections were modeled using three-dimensional (3D) SOLID185 elements from ANSYS element library. The element is defined with eight nodes; each having three translational degrees of freedom in x, y and z directions. Simplified enhanced strain formulation was used for all components; i.e. steel and GFRP, with the exception of the layered formulation that was accounted for in modeling the GFRP stiffeners in the aforementioned exploratory study investigating the use of SBS in new construction. The weld thickness was neglected in the FE model of welded connections between the steel parts (e.g. flanges and web), which were assumed to be perfectly connected. Figure 5.4 shows a typical mesh of the FE model of one of the experimentally tested specimens that was developed for validation purposes.

### Table 5.2 Epoxy material properties obtained from SLB test

<table>
<thead>
<tr>
<th>Epoxy Type</th>
<th>$G_I$ (N/mm)</th>
<th>$G_{II}$ (N/mm)</th>
<th>$\sigma_{avg}$ (MPa)</th>
<th>$\delta_{avg}$ (mm)</th>
<th>$\tau_{avg}$ (MPa)</th>
<th>$\xi_{avg}$ (mm)</th>
<th>$\xi / \xi_u$</th>
</tr>
</thead>
<tbody>
<tr>
<td>S</td>
<td>0.19</td>
<td>0.24</td>
<td>15.8</td>
<td>0.0243</td>
<td>16.9</td>
<td>0.0280</td>
<td>0.26</td>
</tr>
<tr>
<td>MB3</td>
<td>0.40</td>
<td>0.42</td>
<td>17.3</td>
<td>0.0465</td>
<td>18.9</td>
<td>0.0444</td>
<td>0.22</td>
</tr>
</tbody>
</table>

### Table 5.3 Material properties of GFRP stiffeners and CFRP sheets

<table>
<thead>
<tr>
<th>Composites</th>
<th>$E_k$, GPa [ksi]</th>
<th>$\sigma_u$, MPa [ksi]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Extern</td>
<td>(\mu) (mean)</td>
<td>13.20 [1915]</td>
</tr>
<tr>
<td></td>
<td>$SD$</td>
<td>1.85 [268]</td>
</tr>
<tr>
<td></td>
<td>$C_V%$</td>
<td>14.0</td>
</tr>
<tr>
<td>PROForm</td>
<td>(\mu) (mean)</td>
<td>24.38 [3536]</td>
</tr>
<tr>
<td></td>
<td>$SD$</td>
<td>1.50 [216]</td>
</tr>
<tr>
<td></td>
<td>$C_V%$</td>
<td>6.1</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Composite</th>
<th>$E_k$, kN/mm [kip/in]</th>
<th>$\sigma_u$, N/mm [kip/in]</th>
</tr>
</thead>
<tbody>
<tr>
<td>CFRP</td>
<td>(\mu) (mean)</td>
<td>36.35 [150.5]</td>
</tr>
<tr>
<td></td>
<td>$SD$</td>
<td>2.51 [14.32]</td>
</tr>
<tr>
<td></td>
<td>$C_V%$</td>
<td>9.5</td>
</tr>
</tbody>
</table>
Initial Imperfections

The first step in the analysis is to build an initial FE model does not account for imperfections due to fabrication of steel plates, built-up I steel beams, and bonded GFRP sections. These initial imperfections are inevitable in real applications and are essential for triggering buckling modes of failure analytically and numerically. This initial model was first run to extract the first three Eigen buckling mode shapes, which were used to impose initial imperfections in the final FE analysis. Preliminary results from the final FE analyses of the control steel beam; i.e. without any strengthening strategies, showed that the magnitude of the assumed initial deformation has negligible effect on the final results. This was found to be true whether separate mode shapes, or any combination of mode shapes, were considered as an initial imperfection. A similar conclusion was drawn for the retrofitted beams’ initial imperfections. Therefore, it was deemed unnecessary to impose higher buckling modes in the initial imperfections, thus, only the deformation of the 1st buckling mode was introduced in the FE model. The magnitude of the initial imperfections was estimated using ASTM A6-A 6M-05, Standard Specification for Carbon Structural Steel, in which the limiting waviness of steel plates was determined to be 2.6‰ for length of 3.66 m [12 ft.] plate. Therefore, once the normalized deformation results of the 1st Eigen mode were obtained, a scaling factor was applied to the entire displacement vector such that the maximum out-of-plane movement 25.4 mm (1 in.) is
adjusted to 1/1000 of the web panel’s height. The nominal web height of the tested steel beams was 508 mm [20 in.]; therefore, the maximum deformation in the web panel was assumed to be about ±0.508 mm [±0.02 in.] for all specimens. The 1st mode deformed shape of the panel can be visualized as a half-sine wave and full-sine wave for un-retrofitted and SBS retrofitted web panels, respectively, and their normalized modal deformation in FE model are shown in Figure 5.5

5.4.2 Adhesive Model

Bonded applications are typically subjected to complex stress fields and corresponding failure modes (Mode I, II, III and mixed mode) at the bonded interface. In most cases, one of the failure modes becomes dominant, and is therefore assumed to be the only failure mode for the adhesive. Shear loads on I-beams are mainly resisted by the web panels. In the linear loading stage, the out-of-plane web panel deformation is minimal. Therefore, the adhesive is expected to be subjected to shear stresses causing Mode II or Mode III. However, once the panel buckling ensues, the peeling effect due to the web panel buckling shifts the failure mode to a mixed mode debonding with the introduction of Mode I.

A detailed investigation of this progression of failure modes was given in Chapter 4 for beam’s retrofitted using SBS. The FE model study of stress intensity factors to estimate the

![Figure 5.5 Initial deformations obtained 1st Eigen mode FE analysis (a) control and (b) SBS retrofitted beams](image)

Figure 5.5 Initial deformations obtained 1st Eigen mode FE analysis (a) control and (b) SBS retrofitted beams
phase angle changes at steel / GFRP bonded interface between linear and post buckling stages using linear fracture mechanics were performed and the results are reported in Chapter 4. The variations in mixed mode phase angles for different element locations, web panel thickness, crack length, and epoxy types were investigated. It was estimated that the phase angles in post buckling stage were 59° and 29° for 3.8 and 3.2 mm [0.15 and .0125 in.] crack lengths. The study showed that; 1) maximum outward deflection initiates the failure due to peeling effect, 2) peeling effect is dominant with thinner web plates, 3) micro crack length caused more Mode I dominant mixed mode failure, and 4) Tyfo® MB3, which has more elongation capacity, and it also showed mixed mode failure with a more dominant Mode I.

To model this behavior, an adhesive layer at the bonded interface was introduced in the FE model using the CZM bi-linear traction-separation material properties presented earlier. The interface was included in the FE model using INTER205 element to simulate adhesive debonding by increasing the separation between each two corresponding nodes that were initially coincident. One CZM input parameter, shear mode failure (i.e. Mode II and III) contribution factor, \( \beta \), was needed in addition to the material properties to simulate debonding successfully for the SBS test set-up. The shear mode failure contribution factor, \( \beta \), within the mixed mode delamination was studied for both epoxy types. The displacement at failure for RP2-2GFRP-S and RP2-2GFRP-MB3 beams obtained from the experimental tests are compared with the failure prediction of FE model simulations for three different \( \beta \)-values equal to 0.95, 0.5 and 0.0. The result showed that the factor can be assumed between 0.0 and 0.5 for Tyfo® S epoxy because of the experienced failure, and closer to 0.0 for Tyfo® MB3 epoxy because of complete failure of the adhesive did not occur within the predefined deflection limit. The effect of \( \beta \) can be seen in Figure 5.6.

5.4.3 GFRP Stiffeners

The pultruded GFRP section contains a number of fiber layers in rowing direction. The layered structure of the GFRP section was modeled using SOLID185 layered structural thick shell element which supports the linear anisotropic material properties. The thickness of GFRP
section’s web and flange (0.375 mm [3/8 in.]) had a total of five layers of fibers. For GFRP stiffeners of different size used in the parametric study, the number of layers was assumed to be proportional to the thickness and to be equally distributed through the thickness. For example, the 12.7 mm [1/2 in.] thick GFRP stiffener considered in transverse weld free (TWF) structures studies were modeled with seven fiber layers.

The failure of the GFRP stiffener was modeled using Hashin’s failure criteria. Once the failure criteria was satisfied for the GFRP element in FE model, a stiffness reduction factor applied to the material stiffness to simulate the degradation of the GFRP stiffener. The stiffness reduction factor was limited to 80% percent of stiffness loss in the developed models.

5.4.4 Mesh Size

A mesh size study was conducted on one of the experimentally tested beams with SBS retrofitting, RP-2GFRP-MB3, using the final FE model that accounts for the debonding failure at the adhesive layer at steel/GFRP interface. Non-linear behavior of CZM is highly dependent on element size and needed to be captured accurately in FE simulations. The specimen’s web panel thickness for the case study was selected from one of the thinner steel webs because the stiffness contribution of the adhesive to the global stiffness will be more pronounced than that in thicker web panels (Chapter 4).
The mesh size was first approximated using the theoretical critical cohesive zone length (CCZL), $L_{CCZL}$, equation (Turton 2007) given in Eq. (5.2) to develop progressive delamination where $M$ is cohesive zone dependent parameter and is assumed to be 1.0, $E$ is the elastic modulus of the epoxy material, $G_c$ is the critical energy release rate, and $\tau_{max}$ is the maximum interfacial strength. It is important to consider at least one element within the CCZL because the mesh size larger than $L_{CCZL}$ will disturb the accuracy of the CZM behavior and FE results (Turton 2007).

Based on the properties of the adhesives used in this study, the CCZL was estimated to be 5 mm (0.2 in.) for both epoxy types. Therefore, it was concluded that the selection of 5 x 5 mm [0.2 x 0.2 in.] mesh size sufficed for accurately modeling progressive delamination.

The second mesh size study was performed numerically using the full FE model. The experimental results of the RP2-2GFRP-MB3 and RP2-2GFRP-S specimens were simulated with the FE model for four different mesh sizes 25 x 25, 12.5 x 12.5, 6 x 6, and 3 x 3 mm (1 x 1, 0.5 x 0.5, 0.25 x 0.25, and 0.125 x 0.125 in.) in the contact region. The peak load obtained from the experimental and numerical results are compared in Table 5.4 for both epoxy types. The coarser mesh sizes, 25 x 25, and 12.5x 12.5 mm (1 x 1, and 0.5 x 0.5 in.), do not result in accurate predictions of the peak load for both epoxy types. The smallest mesh size (3 x 3 mm [0.125 x 0.125 in.]) provides accurate failure load predictions with less than 1% error. A slightly larger mesh size (6 x 6 mm [0.25 x 0.25 in.]) resulted in predicted peak loads with less than 3% error, while still having a computationally manageable mesh. Therefore, a mesh size of not larger than 6 x 6 mm [0.25 x 0.25 in.] was selected for modeling contact region for adhesive layer to simulate adhesive debonding and to predict load carrying capacity of the SBS retrofitted beams accurately. The size of the elements in regions other than the contact region was relaxed but were not larger than 25.4 x 25.4 mm [1 x 1 in.]. The FE element mesh in contact region and surrounding sections can be seen in Figure 5.7.

$$L_{CZL} = MEG_c(\tau_{max})^{-2}$$ (5.2)
The effective SBS design parameters in addition to the experimentally investigated parameters were extended with parametric studies using the validated FE model. The properties of control beam, RP2, was selected as the basis for all parametric studies. Similar to the SBS retrofitted and experimentally tested beams, the GFRP stiffeners were bonded between the weld toes of the top and bottom flanges. Therefore, the contact area ratio, \( A_r \), was defined as the ratio of GFRP flange width, \( f_w \), to the panel length, \( l_p \). GFRP stiffener size selection was studied by coupling the effects of the stiffener’s moment of inertia and the contact area ratio, \( A_r \). From this first parametric results, the proper size of GFRP stiffener was modeled in further parametric studies.

In the second study, different panel aspect ratios and web thicknesses were numerically investigated keeping the total span length constant, 3.05 m [10 ft.], in the FE model simulations. Similar to the experimental test set-up, the load was applied on the first intermediate stiffener creating a web panel vulnerable to shear buckling. However, the applied shear load was different as the panel dimensions difference and, hence, the shear span. Therefore, un-retrofitted steel beams for different panel aspect ratios were also run in the FE model to normalize the SBS retrofitted beams’ results.

### 5.5 GFRP Stiffeners

To our knowledge, the selection of an optimum GFRP stiffener for SBS retrofitting is not specified in any design specifications or guidelines. Therefore, the concepts behind established design requirements for traditional welded transverse steel stiffener was used as a starting point for establishing similar guidelines for selecting GFRP stiffeners. Steel stiffeners are mainly

<table>
<thead>
<tr>
<th>Epoxy type</th>
<th>Experiment P (kips)</th>
<th>Mesh size (mm x mm [in x in])</th>
<th>25 x 25 [1.0 x 1.0]</th>
<th>12.5 x 12.5 [0.5 x 0.5]</th>
<th>6 x 6 [0.25 x 0.25]</th>
<th>3 x 3 [0.125 x 0.125]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tyfo® S</td>
<td>66.0</td>
<td>76.1</td>
<td>67.6</td>
<td>66.7</td>
<td>66.3</td>
<td></td>
</tr>
<tr>
<td>Tyfo® MB3</td>
<td>66.8</td>
<td>71.6</td>
<td>69.0</td>
<td>68.8</td>
<td>67.4</td>
<td></td>
</tr>
</tbody>
</table>

Table 5.4 Maximum loads obtained from mesh size studies
designed to postpone web buckling in initial loading stages, and sustain the compression forces due to the tension field action in post buckling stages (Rahal 1990). Similarly, the GFRP stiffener will provide bending resistance to the web panel once the web buckling ensues; however, axial resistance to the compression forces within the GFRP stiffeners was not provided because, as stated earlier, the GFRP stiffeners were not bonded to the flanges, but rather were only extended between the weld toes. Therefore, the bending stiffness of the selected GFRP stiffener should at least satisfy the minimum bending stiffness requirements for an intermediate steel stiffeners, $I_{s,\text{min}}$, defined in design codes. For that purpose, AASHTO LRFD Bridge Design Specifications was used as a reference (AASHTO 2012). The minimum equivalent moment of inertia, $I_{F,\text{min}}$, for GFRP stiffener was equated to the minimum required bending stiffness of a similar intermediate steel stiffener, $I_{s,\text{min}}$, given in section 6.10.11.1.3 and using the modular ratio between the steel and GFRP stiffeners as given in Eq. (5.3) where $E_s$, and $E_F$ are the elastic moduli for steel and GFRP stiffeners, respectively.

The obtained flexural rigidity, $EI_{F,\text{min}}$, of the GFRP stiffener was scaled by 0.5, 1.0, 5.0, 15.0, and 30.0 to study its effect on the maximum load capacity of SBS retrofitted beams. Keeping the scaled bending stiffnesses constant, the contact area ratio, $A_r$, was selected as 0.1,

$$I_{F,\text{min}} \geq \frac{E_s}{E_F} \cdot I_{s,\text{min}} \quad (5.3)$$
0.13, 0.2 and 0.3 by varying the GFRP stiffener’s flange width, \( f_w \). As a result, it was possible to establish the interaction between the GFRP stiffener’s flexural rigidity and contact area using the parametric study results. The different size GFRP stiffeners used in the parametric study are listed in Table 5.5, where the listed values correspond to the web height x flange width x thickness in inch units.

5.5.2 Panel Aspect Ratio

Web panel aspect ratios are known to fall within practical limits. Previous work by Rahal (Rahal 1990) considered four different panel aspect ratios (i.e. 0.5, 1.0, 1.5, and 2.0) for transversely stiffened steel girders. In this study, we considered web panel aspect ratios equal to 1.0, 1.5 and 2.0 as one of the parameters for studying SBS retrofitted steel beams. The smallest aspect ratio, 0.5, was not considered in this section because the selected contact ratios would not yield practical stiffener sizes for the investigated beams. The clear web height, \( h_p \), of three different panels was set to 508 mm [20 in.], which translated into 6, 4, and 3 panels for 1.0, 1.5, and 2.0 panel aspect ratios, respectively, within the chosen constant span length, 3.05 m [10 ft.].

<table>
<thead>
<tr>
<th>Web thickness ( t_w )</th>
<th>Contact ratio ( A_r )</th>
<th>Slenderness ( \lambda (h_p/t_w) )</th>
<th>1.0</th>
<th>1.5</th>
<th>2.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.2 mm (1/8&quot;)</td>
<td>0.1</td>
<td>3x2x3/8</td>
<td>4x3x3/8</td>
<td>4x4x1/2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.2</td>
<td>3x4x3/8</td>
<td>4x6x3/8</td>
<td>4x8x1/2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.3</td>
<td>3x6x3/8</td>
<td>4x9x3/8</td>
<td>4x12x1/2</td>
<td></td>
</tr>
<tr>
<td>4.0 mm (5/32&quot;)</td>
<td>0.1</td>
<td>3x2x3/8</td>
<td>4x3x3/8</td>
<td>4x4x1/2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.2</td>
<td>3x4x3/8</td>
<td>4x6x3/8</td>
<td>4x8x1/2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.3</td>
<td>3x6x3/8</td>
<td>4x9x3/8</td>
<td>4x12x1/2</td>
<td></td>
</tr>
<tr>
<td>6.4 mm (1/4&quot;)</td>
<td>0.1</td>
<td>3x2x3/8</td>
<td>4x3x3/8</td>
<td>4x4x1/2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.2</td>
<td>3x4x3/8</td>
<td>4x6x3/8</td>
<td>4x8x1/2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.3</td>
<td>3x6x3/8</td>
<td>4x6x3/8</td>
<td>4x12x1/2</td>
<td></td>
</tr>
</tbody>
</table>

*given dimensions are web height, flange width and thickness in inches (25.4 mm = 1 in.)*

5.5.3 Slenderness

The SBS method adds additional stiffness as a result of bonding the GFRP stiffener(s) to the web panel. The relative stiffness contribution of the GFRP stiffener is highly dependent on the stiffness of the bare web steel plate to delay any premature local web failure. The
performance of SBS retrofitted beams was investigated for different relative stiffness contribution of GFRP stiffener to the retrofitted webs by defining three web panel thicknesses 3.2, 4.0 and 6.4 mm (1/8, 5/32, and 1/4 in.). The panel slenderness, \( \lambda \), corresponding to these web thickness were 160, 128 and 80 as can be seen in Table 5.5.

5.6 FE Model Validation

The developed FE model with CZM adhesive properties and GFRP stiffener delamination capabilities was run to validate the FE model using experimental results. Two criteria were chosen to assess the performance of the FE model. The first validation criterion was the magnitude of the load at the end of the linear elastic limit behavior. The behavior during the post buckling stage constituted the second validation criterion. Only beams with rectangular panels, RP1 and RP2, were considered in the validation study. The rectangular configuration was chosen as it is the case that would likely be in need for strengthening more than beam configurations with square panels. In each of the plots that will be presented in this section, the solid and dashed lines represent the experimental and FE results, respectively.

The first group of rectangular panel beams, RP1, was only strengthened using the proposed SBS retrofitting method. The load deflection curves from the experiment and FE model simulations are given in Figure 5.8 for control and retrofitted RP1 beams. The error in the model’s prediction for the yield and maximum post buckling load capacity of the control beam, RP1 was 7% and 9%, respectively. Welded steel connections allow the beam to maintain a stable load capacity in post buckling stage with increased deflection. This is due to the fact that once buckling occurs, the load path switches from the web panel to the sway frame mechanism. This phenomenon was observed by the authors and is discussed in more detail elsewhere (Ulger et al. 2016). The errors for the SBS retrofitted beams where the stiffeners were bonded using Tyfo\textsuperscript{®} MB3 adhesive, RP1-2GFRP-MB3, were found to be 12% for the elastic limit load, and 4% for the post buckling stage. The final set in this group of specimens is for the beams whose stiffeners were bonded using Tyfo\textsuperscript{®} S adhesive, RP1-2GFRP-S. The error for this beam was 11% in post
buckling loading stage corresponding to a deflection equal to 6.4 mm (0.25 in.). The FE model simulation predicted that failure should happen at around 6.5 mm (0.26 in.).

Based on the plots in Figure 5.8, it can be said that the experimentally obtained post buckling behavior with and without retrofitting was successfully simulated using the developed FE model with the CZM material properties for both epoxy types, namely Tyfo® MB3 and Tyfo® S. It should be noted that difference in the initial stiffness within the linear elastic loading stage for Beam RP1-2GFRP-MB3 was not expected and is probably due to a measurement error for the LVDTs that recorded support deformations during the tests.

A similar comparison of load deflection curves for beams in the RP2 group, which includes welded steel stiffeners, RP2-2STS, bonded GFRP stiffener, RP2-2GFRP-MB3, and CFRP sheet, RP2-2CFRP-MB3, is shown in Figure 5.9. The ultimate load capacities and the post buckling behaviors were successfully predicted using the developed FE models. Similar to what was observed for the RP1 specimens, a difference in the initial slope between experimental and

Figure 5.8 Graphical representation of experiments and FE model simulations for RP1-0, RP1-2GFRP-MB3, and RP1-2GFRP-S beams
FE model results is also noted. While some of this discrepancy may be due to the support LVDT readings, it is also clear that the FE model consistently results in a stiffer response. From both figures, Figure 5.8 and Figure 5.9 it can also be said that this difference is bigger for the beams with thinner web panel, RP2. The predicted stiffness for beams with higher initial web stiffness, such as RP2-2CFRP-MB3, are almost identical to experimental results.

Finally, the experimental and FE model simulation results for beams retrofitted using one GFRP stiffener are presented in Figure 5.10 for both epoxy types and the corresponding control specimen. The discrepancy in the initial web stiffness is also apparent for specimens with thin initial web thickness (2.90 mm [0.1142 in.]). As before, the peak loads and the progression of failure were the main validation criteria. The FE model was able to detect key points in the progression of failure during the post-buckling stage. The sudden drops in load resistance, which correspond to debonding for a substantial portion of the interface, can be seen for both SBS-retrofitted beams. Two such drops were observed experimentally and captured numerically for
Beam RP2-1GFRP-S, but only one drop took place for RP2-1GFRP-MB3. The peak loads at the end of the elastic limit stage were also successfully predicted with a maximum error of 10% and 5% error, respectively, for Beam RP2-1GFRP-MB3.

In addition to comparing the load-deflection plots, the complete experimental and FE model results for square and rectangular panel beams are presented in Table 5.6. For each tested beam, the experimentally obtained load at the end of the linear elastic limit, the peak load in the post-buckling stage, and the displacement at global failure are given with their corresponding FE results. Since a clear yielding point did not exist for all the experimentally tested beams, the intersection of the initial linear loading portion of the curve and a linearly fitted segment for the post buckling portion of the curve were used to determine the end of the linear elastic limit to identify an idealized yield point. It should also be noted that the experiments were conducted using test procedures that ended for several of these tests at predefined deflections to limit the damage to the beams, which were tested twice; one time for each side. These limits were relaxed.
in later experiments to capture advanced post buckling behavior. Therefore, the deflections reported in Table 5.6 do not necessarily represent the failure of the beam and are indicated in the table with superscript ‘dl’ if the specimen did not fail until the predefined deflection limit.

The ratio between the FE predicted and experimentally observed loads at yielding and post buckling stages were computed. The maximum error was 16.5 and 11% in predicting yielding and post buckling peak loads for SP2-2GFRP-S and RP1-2GFRP-S beams, respectively. On average, the FE model overpredicted yielding by 7% and underestimated the peak load by 3%. The coefficient of variation for all results including control and retrofitted beams was 5.6 and 4.5% for yielding and post buckling stages, respectively, which is considered to be low for predictions of structural behavior.

In summary, it can be said that predicting the global failure of the retrofitted beams is challenging because the possibility of existence of random micro cracks in the adhesive layer

### Table 5.6 Experimental and FE results at maximum load capacities at yield point and in post buckling stage

<table>
<thead>
<tr>
<th>Beam</th>
<th>Stiffener</th>
<th>Contact type</th>
<th>Experiments</th>
<th>FE Simulation</th>
<th>( \frac{P_{FE}}{P_{EX}} ) (yield)</th>
<th>( \frac{P_{FE}}{P_{EX}} ) (post)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>( P_{yield} ) (kN)</td>
<td>( P_{post} ) (kN)</td>
<td>( \Delta_{fail} ) (mm)</td>
<td>( P_{yield} ) (kN)</td>
</tr>
<tr>
<td>SP1</td>
<td>0</td>
<td>-</td>
<td>251</td>
<td>278</td>
<td>-</td>
<td>277</td>
</tr>
<tr>
<td></td>
<td>2 GFRP</td>
<td>S</td>
<td>343</td>
<td>389</td>
<td>6.9 (^{dl})</td>
<td>373</td>
</tr>
<tr>
<td></td>
<td>2 GFRP</td>
<td></td>
<td>383</td>
<td>434</td>
<td>4.6</td>
<td>431</td>
</tr>
<tr>
<td>SP2</td>
<td>0</td>
<td>-</td>
<td>240</td>
<td>240</td>
<td>-</td>
<td>227</td>
</tr>
<tr>
<td></td>
<td>2 GFRP</td>
<td>S</td>
<td>289</td>
<td>322</td>
<td>6.4 (^{dl})</td>
<td>337</td>
</tr>
<tr>
<td></td>
<td>1 GFRP</td>
<td></td>
<td>271</td>
<td>311</td>
<td>7.1</td>
<td>296</td>
</tr>
<tr>
<td>RP1</td>
<td>0</td>
<td>-</td>
<td>278</td>
<td>289</td>
<td>-</td>
<td>298</td>
</tr>
<tr>
<td></td>
<td>2 GFRP</td>
<td>MB3</td>
<td>347</td>
<td>409</td>
<td>17.8 (^{dl})</td>
<td>388</td>
</tr>
<tr>
<td></td>
<td>2 GFRP</td>
<td>S</td>
<td>382</td>
<td>409</td>
<td>6.4</td>
<td>385</td>
</tr>
<tr>
<td>RP2</td>
<td>0</td>
<td>-</td>
<td>216</td>
<td>222</td>
<td>-</td>
<td>229</td>
</tr>
<tr>
<td></td>
<td>2 STS</td>
<td></td>
<td>245</td>
<td>334</td>
<td>17.8 (^{dl})</td>
<td>284</td>
</tr>
<tr>
<td></td>
<td>2 CFRP</td>
<td></td>
<td>301</td>
<td>414</td>
<td>19.1 (^{dl})</td>
<td>298</td>
</tr>
<tr>
<td></td>
<td>1 GFRP</td>
<td>MB3</td>
<td>276</td>
<td>302</td>
<td>11.2</td>
<td>302</td>
</tr>
<tr>
<td></td>
<td>2 GFRP</td>
<td></td>
<td>298</td>
<td>336</td>
<td>16.5 (^{dl})</td>
<td>308</td>
</tr>
<tr>
<td></td>
<td>1 GFRP</td>
<td>S</td>
<td>288</td>
<td>289</td>
<td>3.56</td>
<td>299</td>
</tr>
<tr>
<td></td>
<td>2 GFRP</td>
<td></td>
<td>296</td>
<td>302</td>
<td>11.4</td>
<td>308</td>
</tr>
</tbody>
</table>

\( \mu \) (mean) 1.07 0.97
\( \text{SD} \) 0.06 0.04
\( \text{CV}\) (%) 5.6 4.5

\(^{dl}\) indicates if the specimen did not fail until the predefined deflection limit.
that can grow in unpredictable stages of loading in unexpected regions. Nevertheless, it can be stated that the FE model is capable of predicting the adhesive failure accurately in post buckling stage using the CZM to represent the steel/GFRP interface.

5.7 Results and Discussions

5.7.1 GFRP Stiffener Size Study

In this section, we investigate the effect of the GFRP stiffener size on the behavior of SBS-retrofitted beams. The size study was conducted parametrically using a beam whose dimensions are based on RP2 specimens. The panel aspect ratio was 1.5:1.0 and the web thickness was 3.2 mm (1/8 in.). The relative stiffness demand from the bonded GFRP stiffener will be higher for thinner steel web panels. The maximum loads were obtained for different variations of the GFRP stiffener’s flexural rigidity as a function of the minimum required value, $E_I F_{min}$, (i.e. 0.5, 1, 5, 15, and 30). Also, various GFRP stiffener contact area ratios, $A_r$, (i.e. 0.1, 0.13, 0.2 and 0.3) were considered. A total of 20 cases were analyzed to study the effect of GFRP stiffener size on SBS retrofitted beams. The predicted maximum load was then normalized using the maximum load capacity of the control beam, RP2. The results showed that when the provided bending stiffness is less than the minimum required stiffness; i.e., $0.5 E_I F_{min}$, the strength enhancement is limited compared to cases with higher flexural rigidity values as can be seen in Figure 5.11. It can also be said from the figure that once a bending stiffness ratio, $E_I F / E_I F_{min}$, of 5 or more is used, the capacity enhancement is almost identical. Conversely, capacity enhancement kept increasing as the contact area ratio increased. The main goal of this part of the study was to assist in the selection of GFRP stiffeners that can achieve the full potential of SBS retrofitting with an optimum GFRP stiffener size. Further optimization is needed in order to ensure that the integrity of the GFRP stiffener’s laminas during pre and post-buckling loading stages is maintained. It should be noted that the selection of the GFRP stiffener can be made from commercially available pultruded FRP products that satisfies the aforementioned minimum bending stiffness requirement. For the experimentally tested beams in
the RP2 group, the provided bending stiffness was about 270 times higher than minimum bending stiffness, $E I_{F, \text{min}}$, and the contact area ratio, $A_r$, was 0.2.

### 5.7.2 Panel Aspect Ratio and Slenderness

Another set of beams with 1.0:1.0, 1.5:1.0 and 2.0:1.0 panel aspect ratios was analyzed for GFRP stiffener contact ratios, $A_r$, equal to 0.1, 0.2, and 0.3. The RP2 beam was also chosen as the control specimen, whose capacity was used to normalize the FE load capacity increases to investigate relative effectiveness of SBS. Three nominal web thicknesses were considered 3.2, 4.0, and 6.4 mm [1/8, 5/32 and 1/4 in.], resulting in slenderness values of 160, 128 and 80, respectively. A total of 9 different cases will be presented for each panel aspect ratio covering three slenderness values and three GFRP stiffener contact ratios.

Three plots are shown in Figure 5.12 to demonstrate the effect of panel aspect ratio, panel slenderness, and contact area ratio on the efficiency of SBS. It can be seen from the plots that the rate of enhancement to the load capacity gradually reduces as the web panel aspect ratio increases from 1.0 to 2.0 for the same contact ratios. The square web panel, panel aspect ratio 1.0:1.0, provides the higher load capacity increase (60% for the most slender case) with 0.3
contact ratio as shown in Figure 5.12 (a). The trend of increase in load capacities for different contact area ratios of 0.1, 0.2, and 0.3 can be seen in Figure 5.12 with dashed lines. Similar to the GFRP stiffener size study, the larger the GFRP stiffener contact area ratio, \( A_r \), results in larger load capacity increases. The trend shows that the rate of load capacity increases reduces with the increase in GFRP stiffener contact area ratio. Finally, the SBS method is more effective on thinner web panel, 3.2 mm (1/8 in.) where the slenderness ratio is 160, and the load capacity increase gradually reduces with the increase of web panel thickness; i.e., decrease in web slenderness.

For the extreme end of the investigated parameters, panel aspect ratio of 2.0 with a slenderness value of 80, SBS does not seem to enhance the load capacity of the retrofitted beam as can be seen in Figure 5.12 (c). The reason for this behavior is that this beam was not controlled by shear buckling of the web panel, but rather the failure mode was controlled by local flange buckling Therefore, it can be said that even though SBS is capable of altering the expected failure mode for all other beams retrofitted with SBS, for this case another failure mode limited the capacity of the beam.

![Graph showing normalized load vs. contact ratio](image)
Investigation of Possible Use of SBS in New Construction

The scope of the SBS method is extended in this section to replace transversely welded bearing and intermediate steel stiffeners with bonded pultruded GFRP stiffeners for new construction.

\[ \frac{l_p}{h_p} = 1.5 \]

\[ \frac{l_p}{h_p} = 2.0 \]

Figure 5.12 Parametric study of different panel aspect ratios, \( \frac{l_p}{h_p} \), (a) 1.0, (b) 1.5, (c) 2.0.

5.8 Investigation of Possible Use of SBS in New Construction

The scope of the SBS method is extended in this section to replace transversely welded bearing and intermediate steel stiffeners with bonded pultruded GFRP stiffeners for new construction.
construction. The main driver behind this study is to utilize the superior properties of composite materials beyond structural retrofitting and into new construction to completely eliminate welded transverse steel stiffeners. Transverse welds are known to result in a fatigue category that penalizes the design more than longitudinal welds (AASHTO 2012). Therefore, eliminating transverse welds will translate into a better fatigue performance for steel beams constructed using SBS solely as the only transverse stiffening element. Composite materials have also been shown to extend fatigue life in retrofitting applications by limiting crack growth steel (Tavakkolizadeh 2003).

In order to study the feasibility of transverse weld free (TWF) structures, the material and geometrical properties of RP2 beam were selected. The transverse steel stiffeners were replaced with a C-shaped GFRP (C-GFRP) stiffener whose dimensions were selected to perfectly fit between the web, and top and bottom flanges of the steel beam. Even though this imaginary C-GFRP stiffener shape is not readily available, it was chosen for this study as an initial attempt to create TWF structures. If the concept is successful, it can be commercially pultruded in standardized dimensions like other FRP shapes. Since the C-GFRP stiffeners were extended to the top and bottom flanges, they would be expected to bear the compression forces due to the tension field action like steel stiffeners. In the case of C-GFRP stiffeners, ply degradation within the pultruded GFRP stiffener due to combined compression force and bending moment needs to be checked for achieving successful designs. The performance of the C-GFRP stiffener is affected by the same attributes discussed in the earlier parametric study; e.g. contact area and moment of inertia. However, a detailed study similar to the earlier parametric study was not the goal of this exploratory work.

The maximum commercially available pultruded FRP thickness (12 mm [1/2 in.]) was chosen as the basis for the proposed C-GFRP stiffener’s thicknesses, and the flange width of the stiffener was selected to be 76 and 152 mm [3 and 6 in.]. C-GFRP stiffeners with two different contact areas, C-3 x 6 x 1/2 and C-6 x 6 x 1/2, were considered for this exploratory study using dimensions similar to Specimen RP2 and retrofitted Specimen RP2-2STS discussed earlier for
FE model validation. The additional intermediate steel stiffener that was welded in the middle of the critical web panel in the SBS study discussed earlier was replaced with a T-shaped GFRP stiffener as before; i.e., without bonding the stiffener to the top and bottom flanges. The flange width and thickness were taken similar to the C-GFRP stiffener (12 mm [1/2 in.]) . The beams in this exploratory study were designated based on C-GFRP stiffener’s flange width [3 and 6 in.]. For example, the control beams in this exploratory study were called 3C-GFRP-0 and 6C-GFRP-0, and the SBS retrofitted beams bonding T-shaped GFRP stiffeners are called 3C-GFRP-3T and 6C-GFRP-6T. Figure 5.13 shows the FE models for the control and retrofitted critical web panels considered in this exploratory study for TWF steel structures.

![Figure 5.13 Bonded transverse GFRP stiffener FE models; (a) 3C-GFRP-0, (b) 3C-GFRP-3T, (c) 6C-GFRP-0, and (d) 6C-GFRP-6T](image)

5.8.1 Results

Results from the exploratory study where the welded transverse stiffeners were substituted with the bonded C-GFRP stiffeners for a control beam, RP2, and a retrofitted beam, RP2-2STS, will now be presented. The load deflection plots for 3C-GFRP and 6C-GFRP beams with and without SBS retrofitting were shown in Figure 5.14.

The effect of the failure of GFRP stiffeners can be seen as a gradual load capacity decrease in post buckling stage when the predefined Hashin’s failure criteria was satisfied. The
results show that the initial stiffness of the 3C-GFRP and 6C-GFRP beams are slightly lower than that for the beams with welded stiffeners, RP2-0 and RP2-2STS. Based on the FE model results, it is estimated for the load capacity for 3C-GFRP-0 and 6C-GFRP-0 was 222 and 249 kN (50.0 and 56.0 kips). These results are 2.8% less and 9.0% more than the maximum load capacity for the corresponding control beam with steel stiffeners, RP2, respectively. However, unlike the beams with steel stiffeners, a gradual load capacity decrease is observed for the TWF beams due to the gradual loss of stiffness for the GFRP stiffener and adhesive layer. The analyses were stopped for various reasons including convergence issues for the beams with highly nonlinear behavior (e.g. C-GFRP beams). Nevertheless, all analyzed beams exhibited a ductile failure behavior with a distinct plateau after reaching their respective elastic limits. A distinct difference between the 3C-GFRP and 6C-GFRP beams is their post buckling behavior. Beams 6C-GFRP and 6C-GFRP-6T exhibited a gradual capacity increase after the initiation of buckling, whereas the capacity for 3C-GFRP and 3C-GFRP-3T decreased slightly.

Figure 5.14 C-shaped 3 and 6 inch wide bearing and transverse GFRP stiffeners and SBS retrofitted beams with bonding T-shaped 3 and 6 inch wide GFRP stiffener
Now we compare the behavior of the TWF control beams, 3C-GFRP and 6C-GFRP, with the SBS retrofitted TWF beams, 3C-GFRP-3T and 6C-GFRP-6T. The maximum estimated load capacity is 294 and 369 kN [66.1 and 83.0 kips], for 3C-GFRP-3T and 6C-GFRP-6T, respectively. These capacities are 7% less and 17% more than maximum load capacity of RP2-STS beam. It is important to note that the clear panel length; i.e. unsupported distance between stiffener edges, of the critical panel for the RP2-STS beam is 381 mm (15 in.), was reduced to 305 and 229 mm (12 and 9 in.) when the C-GFRP stiffeners were bonded instead of the welded steel stiffeners. Consequently, higher load capacities should be expected with wider C-GFRP flange widths as a result of reducing the panel unsupported length. Finally, like with most other FRP applications, the deflection under the applied load for 3C-GFRP-3T and 6C-GFRP-6T beams at failure was less than that for beam retrofitted with steel stiffeners, RP2-2STS. While this is not a desirable outcome, the fact that a ductile behavior is obtained is promising and can be further improved in future studies.

In summary, the proposed C-GFRP stiffener can be pultruded for some standardized dimensions that allow it to be used in fabrication of large optimized TWF steel beams. Alternatively, it may be possible to fabricate C-GFRP stiffeners by bolting readily available products to produce the required shape. The positive attributes highlighted in this study for creating TWF steel beams using C-GFRP stiffeners are promising and warrant further investigation to optimize the C-GFRP section dimensions, adhesive properties among others.

5.9 Conclusions

A numerical investigation of the proposed “Strengthening-by-Stiffening” (SBS) method to strengthen shear deficient steel beams by bonding GFRP stiffeners to web panels was conducted using a validated FE model. The beams were loaded monotonically in asymmetric three point loading setup creating a critical web panel. In addition to the GFRP stiffened beams, an alternative retrofitting method where CFRP sheets were bonded to the entire critical web panel and a conventional retrofitting method in which steel stiffeners were welded to the critical web panel were also studied experimentally and numerically. Two different epoxy types, Tyfo®
S and Tyfo® MB3, were also used to bond the pultruded GFRP stiffeners to the web panel. To bond the CFRP sheets to the web panel, Tyfo® S was used for saturating the carbon fibers and Tyfo® MB3 was used to bond the saturated CFRP sheets to the web panel. The mechanical properties of steel, pultruded FRP sections, and adhesives were experimentally determined and presented in this study. These properties were incorporated in the FE model, which takes into account geometric and material nonlinearities, adhesive debonding at steel/GFRP interface and fiber delamination in GFRP stiffeners. Debonding failure modes of the adhesives and delamination of GFRP stiffeners in post buckling stage were captured using bi-linear CZM material models and activating Hashin’s failure criteria in FE model. Load-deflection curves from FE simulations were compared with the experimentally obtained curves to validate the FE model. The validated model was then used to investigate the minimum required bending stiffness of a GFRP stiffener by conducting a parametric study for different panel aspect ratios and GFRP stiffener contact area ratios. The impact of initial web slenderness on the efficiency of SBS was also investigated. Finally, the FE model was used to explore the feasibility of developing new transverse weld free (TWF) steel beams by replacing all the transversely welded steel stiffeners with bonded GFRP stiffeners to reduce this type of weld, which is known to create fatigue problems. The following conclusions can be drawn from the results obtained from these simulations:

1. The experimental results showed that 51, 86 and 50% load capacity increase can be achieved in comparison with corresponding control beams by bonding GFRP stiffener (SBS method), bonding CFRP sheets and welding steel stiffeners in critical web panels, respectively. It should be noted the amount of contact area for the CFRP sheet retrofitting method is 80% more than for GFRP stiffeners in SBS. This translates into higher costs because more adhesives will be required for bonding in addition to more time and labor for surface preparations, which is required before bonding. Also, a fully covered web panel may help with protecting against corrosion as a result of oxygen deprivation, however, inspections of the condition under a fully cover panel is challenging.
2. The progression of failure observed in experimentally tested beams was also captured in post buckling stages using the adopted CZM material model in FE simulations. The failure of beams with a single GFRP stiffener configuration was more sudden than that for double GFRP stiffener configuration in experimental and FE simulations. Furthermore, SBS-retrofitted beams whose GFRP stiffeners were bonded using a brittle epoxy, Tyfo® S, are more susceptible to sudden failure than ones that used a ductile epoxy type, Tyfo® MB3.

3. The mean error in predicting the linear elastic limit load and the maximum post buckling peak load was 7% and 3%, respectively. The corresponding coefficient of variations were 5.6% and 4.5%, respectively, which is considered acceptable for structural applications and allows using it for further parametric investigations.

4. In SBS, the GFRP stiffener mainly bears bending forces as the web panel buckles, and is not subjected to the tension tie end forces. Therefore, GFRP stiffener size can be determined based on existing minimum bending stiffness requirements for transverse steel stiffener design in current design codes. The GFRP size has negligible effect on the load capacity increase once a certain bending stiffness for the GFRP stiffener (approximately 5 times $EI_{F,min}$) is exceeded. However, the selection of the flange width which determines the contact area has significant effect on load capacity enhancements. The availability of GFRP sections in market may limit the GFRP stiffener selection. In such situations, multiple GFRP stiffeners may be bonded to the deficient web panel to achieve the required properties for larger beams.

5. The SBS method splits the web panel in two sub-panels, however, it reduces the unsupported panel length by less than half because the GFRP stiffener’s flange width further stiffens the web plate beyond the centerline of the stiffener. Therefore, higher load capacity enhancement were observed for beams with smaller panel aspect ratios. The highest load capacity increase was 60% for beams with a square panel (1.0:1.0 panel
aspect ratio) and with the largest considered contact area. Load capacity enhancements were gradually reduced as the panel aspect ratio increased from 1.0:1.0 to 2.0:1.0.

6. SBS is more effective on beams with slender webs. The gains in load capacity for beams with thinner web thickness, 3.2 mm (1/8 in.), was more than that for beams with thicker webs, 4.0 and 6.4 mm [5/32 and 1/4 in].

Results from the exploratory study to create transverse weld free (TWF) steel beams showed that pultruded composite stiffeners are a promising alternatives for welded steel stiffeners. The load capacity of a beam where web panels were split with bonded C-GFRP stiffeners in addition to T-shaped GFRP stiffener in the middle of the critical panel reached 17% higher load capacity than beams with welded steel stiffeners. As expected, the behavior of this new steel construction method is less ductile than traditional all-steel construction. The failure of GFRP stiffeners with alternative stiffeners sizes can be further investigated to improve ductility while benefitting from longer fatigue life resulting from the elimination of transverse welds.

5.10 Nomenclature

- $A_r$: Contact ratio of bonded region
- $C_V$: Coefficient of variation
- $d_l$: Predefined deflection limit
- $E_F$: Elastic modulus of GFRP stiffener
- $EI_F$: Equivalent bending stiffness
- $E_k$: Elastic modulus of materials
- $E_s$: Elastic modulus of steel
- $f_w$: GFRP stiffener flange width
- $G_{II}, G_{III}$: Energy release rate of Mode I and Mode II failures
- $h_p$: Clear panel height in panel
- $I_F$: Moment of inertia of GFRP stiffener
- $I_s$: Moment of inertia of intermediate steel stiffener
- $L_{CCZL}$: Critical cohesive zone length
$l_p$  Control panel length in a beam  
$P$  Applied load on the control panel  
$SD$  Standard deviation of sample  
$t_w$  Thickness of steel web  
$\beta$  Mode II failure contribution factor in CZM definition  
$\delta_{\text{avg}}, \xi_{\text{avg}}$  Interlaminar average normal and tangential separations at peak stresses  
$\delta^u, \xi^u$  Maximum normal and tangential separations at failure  
$\Delta$  Vertical displacement at the load line  
$\lambda$  Slenderness  
$\mu$  Mean of sample  
$\sigma_{\text{avg}}, \tau_{\text{avg}}$  Interlaminar average peak normal and tangential stresses  
$\sigma_{\text{en}}, \varepsilon_{\text{en}}$  Engineering stress and strain values  
$\sigma_{\text{tr}}, \varepsilon_{\text{tr}}$  True stress and strain values  
$\sigma_u$  Ultimate failure load

5.11 References


ANSYS "ANSYS ® Academic Research, Release 16."


Chapter 6. Summary and Conclusions

The present dissertation investigates in detail a new retrofitting method for deficient thin walled steel structures in which pultruded GFRP composite stiffeners are bonded to buckling prone thin plates. The new method is referred to as “Strengthening-by-stiffening” or SBS. Unlike the typically used planar composite materials for strengthening applications, the proposed method relies mainly on out-of-plane stiffness of bonded GFRP stiffener which is a function of its cross-sectional and material properties. An extensive experimental program was executed to investigate the effectiveness of the SBS technique in shear deficient thin walled steel beams. The steel beams were designed to account for two different panel aspect ratios and nominal web thicknesses. The experimental program was designed to investigate the effect of several major factors on the efficiency and performance of SBS. These factors are: GFRP stiffener contact area (i.e. controlled by varying the GFRP flange width), GFRP stiffener configuration, and adhesive type (brittle versus ductile). The results and performance of one of the SBS-retrofitted beam was compared with experimental results from two alternative retrofitting schemes; namely using conventionally welded steel stiffeners and bonding CFRP sheets to the entire critical web panel. The experiments showed that epoxy failure was dominant in all tested beams, and that it should be the controlling parameter in design of SBS method. Therefore, an FE model was developed to investigate epoxy debonding failure based on linear elastic fracture mechanics theory. Mixed mode phase angles were determined from parametric studies for inclusion in SBS FE models that account for the epoxy debonding failure. The approximated phase angle between GFRP stiffener and steel were then experimentally investigated by conducting single leg bending (SLB) tests to define CZM properties for the two epoxy types used in this study. The developed FE model was validated using experimental results from the tested full-scale beams. The developed FE model accounts for epoxy debonding and GFRP stiffener delamination in addition to geometric and material nonlinearities. Parametric studies where then conducted to investigate the effective GFRP stiffener size, and its impact on SBS efficiency for different panel aspect ratios and web slendernesses. Finally, an exploratory study was conducted to investigate the potential of using
SBS concept for fabricating transverse weld free (TWF) steel beams by replacing traditional steel stiffeners with GFRP stiffeners for the purpose of increasing their fatigue life.

The main results and findings of the previously mentioned studies that were completed as part of the scope of this dissertation are summarized in the following:

1. The proposed retrofitting method, SBS, increases the load capacity of deficient thin walled steel structures subjected to shear induced buckling failure. In addition to load capacity enhancement, the SBS method also enhances the limiting service load capacity of structurally deficient steel beams.

2. The properties of the adhesive type used in bonding GFRP stiffeners is the most important criterion in SBS retrofitting. A ductile adhesive is the preferred bonding agent in structural steel bonding applications even though similar load capacities can be achieved with brittle epoxies. A ductile adhesive enhances the ductility of the strengthened beams by allowing larger deflections prior to failure. As a result, the toughness of the beams retrofitted using ductile adhesives is larger than that for beams strengthened using brittle adhesives.

3. Failure of the adhesive layer in SBS method started when the web panel buckling started. The numerical investigation of adhesive layer showed that the failure of brittle epoxy is caused by a combination of normal and tangential separation failures, and that it occurs at an early stage of the buckling process due to the adhesive’s low ability to elongate. Conversely, ductile adhesives allowed full web buckling as a result of their ability to experience excessive elongations. Therefore, normal separation failure becomes more dominant in debonding failure of ductile adhesives, and the deformation on the web panel causes a peeling type failure (Mode I) for the bonded GFRP stiffener.

4. SBS retrofitting is more effective in strengthening slender sections by increasing both the elastic limit load and the post buckling maximum overall load capacity. Similar load capacity enhancement for SBS retrofitted compact steel sections may not be
possible; however, SBS can still alter the expected mode of failure from shear buckling to other local or global failure mechanisms (e.g. flange local buckling).

5. In SBS, the pultruded GFRP stiffeners are only bonded to the web panels and are not bonded to the tension or compression flanges. Therefore, the GFRP stiffeners are not subjected to the compression and tension tie forces that develop in retrofitted web panels. They are mainly subjected to flexural demands resulting from the web panel buckling. Therefore, the geometrical properties of the pultruded FRP sections becomes the important factor affecting the behavior, which allows the use of cheaper FRP types such as GFRP. Minimum design requirements for intermediate steel stiffeners can be used to assist in establishing minimum GFRP stiffener size requirements for SBS applications.

6. The investigation of different GFRP stiffener configuration revealed that the diagonal bonded GFRP stiffener serves as a load bearing compression strut element, while vertical GFRP stiffeners serve as a bracing element and showed a more ductile post buckling load carrying performance. Using a single vertical GFRP stiffener configuration or one stiffener on each side in a double stiffener configuration were investigated. The results showed that the double stiffener configuration is more reliable than single stiffener configuration. The first reason is that the tension failure of the bonding adhesive is more likely than shear and compression failures. Therefore, the failure of the adhesive in tension on one side will be supported from the opposite side that is subjected to compression in a double stiffener configuration. Such support does not exist in the single stiffener configuration, resulting in a debonding failure that occurs earlier than it does for double stiffener configurations.

7. The GFRP stiffener contact area has a significant effect on the maximum load capacity of SBS retrofitted beams. Load capacity enhancement trends show that the flexural rigidity affects the achievable enhancement levels, however, this effect is capped after providing stiffeners with about five times the minimum required value, $EI_{F,\text{min}}$.  

152
Lesser load capacity increases were obtained for beams with higher web panel aspect ratio.

8. Retrofitting shear deficient beams using double GFRP stiffeners bonded using ductile adhesive results in similar toughness and load capacity enhancements as those obtained with traditional welded steel stiffeners.

9. An alternative retrofitting method was applied by bonding biaxial CFRP sheets to the critical web panel. The load capacity of the beam strengthened using this method was higher than that of steel welded and GFRP bonded beams. Similar deflection limits were achieved without any global failure in all three retrofitting schemes. It should be noted that CFRP sheets requires more adhesives in addition to more labor and time for surface preparation to cover the entire web panel. Inspection of fully covered web panels may also be a challenge. Nevertheless, further investigations are warranted because of the superior performance, which may also provide additional protection to the web panel against corrosion due to oxygen deprivation.

10. Fabricating steel beams with longer fatigue lives by eliminating transverse welds (transverse weld free or TWF) is possible using GFRP stiffeners that substitute for traditional steel stiffeners. Further studies are needed to investigate key behavioral issues for TWF beams such as GFRP stiffener crushing and delamination under compression demands.
Appendix I. Effect of Adhesive Type on Strengthening-By-Stiffening for Shear-deficient Thin-walled Steel Structures‡

A. I  Abstract

Strengthening-By-Stiffening (SBS) is a novel technique whose purpose is to improve structural strength by stiffening buckling-prone regions in thin-walled steel structures using pultruded composite sections. A proof of concept study showed that SBS can achieve gains in shear strength of up to 56% using glass fiber reinforced polymers (GFRP) sections. This paper presents experimental results showing the effect of adhesive type on the efficiency of SBS for shear-deficient thin-walled steel beams. Specimens strengthened with two adhesive types were tested; a generic type (Type I) that is typically used for FRP-strengthening of concrete structures and a relatively new type (Type II) that is particularly promoted for steel structures. Like most FRP-strengthened structures, a debonding failure mode was observed for SBS specimens strengthened using adhesive Type I. Conversely, specimens strengthened using adhesive Type II did not fail by debonding, but rather by buckling of the smaller (less slender) shear panels. The resulting ductile failure mode is uncommon for FRP strengthening techniques and can lead to new applications of FRP strengthening for steel structures that were not possible using more brittle adhesives with lower capacity to absorb inelastic energy.

Keywords: Composites; adhesive; strengthening; buckling; steel; shear

A. I 1  Introduction

Aging steel structures suffer from inadequate capacity due to several reasons such as deterioration because of environmental attacks, increase in applied loads, among others. Extending the service life of existing structures is an economically feasible decision in comparison to full replacement of the structure provided that its strength can be increased to meet applied demands. Therefore, engineers are always exploring new strengthening techniques and materials that are sound, both structurally and economically.

Several methods can be used for strengthening deficient structures including member enlargement, external post-tensioning, and anchoring or welding of additional steel plates. In recent years, the use of adhesively bonded composite materials gained acceptance for strengthening applications because of the many advantages they offer (ACI Committee 440 2007). Composite materials are light weight and can exhibit high tensile resistance, which leads to a high strength-to-weight ratio in comparison to other materials. Additionally, composite materials are not susceptible to corrosion and can be easily handled and installed using adhesives, which allows for minimizing down times of the strengthened structure. Carbon, glass, and aramid fiber reinforced polymers (FRP) have been used in structural strengthening applications. It has been demonstrated that thin carbon FRP (CFRP) sheets, strips or laminates are efficient in strengthening concrete structures because of their high tensile strength (ACI-440.2R 2008). Glass FRP (GFRP) has also been used in strengthening applications. However, the GFRP’s lower modulus of elasticity in comparison to CFRP reduces its strengthening efficiency since the main contribution of FRP in strengthening applications is an added tensile capacity in deficient zones. Similarly, the strengthening efficiency of steel structures using CFRP was lower than that of concrete structures because of the higher modulus of elasticity of steel, which leads to the need for large amounts of CFRP to achieve similar strengthening levels as those achieved for concrete structures (Sen et al. 2001) Alternatively, the more expensive high or ultra-high modulus CFRP can also be used to strengthen steel structures (Schnerch et al. 2004).

Strengthening-By-Stiffening (SBS) is a new effective strengthening method that is suitable for thin-walled steel structures where pultruded FRP composite sections are bonded to buckling prone slender plates (Okeil et al. 2011a). In SBS, buckling resistance of the thin-walled steel members is enhanced by using the out-of-plane stiffness of FRP sections as opposed to conventional strengthening techniques using composite materials that rely on in-plane strength of FRP fibers. This method mimics conventional welded steel stiffeners where the pultruded FRP section corresponds to the steel plate and epoxy bonding corresponds to the welding (Okeil et al.
A schematic illustrating the main components of an SBS strengthened plates; i.e. steel plate, epoxy, and pultruded FRP composite section, can be seen in Figure A. I 1.

Preliminary experimental results using commercially available materials proved that the SBS concept is an efficient strengthening technique that is highly dependent on the stiffness contribution of the pultruded FRP section and to a lesser extent on the strength of the FRP fibers for structures that are prone to buckling failures (Okeil et al. 2009). The previous experimental results also showed that the mechanical properties of the adhesive used to bond the pultruded FRP section to the steel plate is the most important factor affecting SBS efficiency since the mode of failure is usually controlled by debonding. While ductile adhesives exist and have been used in other engineering fields, epoxy resins that are commonly used in structural strengthening applications fail in brittle manner (ACI Committee 440 2007; Lee et al. 1967; Mays et al. 1992). Several researchers tested plain epoxy specimens under tension, shear and compression loading at different strain rates and temperatures (Fiedler 2001; Gilat et al. 2007; Littell 2008). It was reported that specimens fail in brittle manner under tension; however, more ductile behavior was observed for the specimens tested in compression and shear. In general, shear tests exhibited a higher ductile behavior than that observed in tension tests (Gilat et al. 2007). Furthermore, tension specimens fail at smaller strain levels than the shear and compression specimens (Fiedler 2001). Elevated temperatures were also found to decrease the failure stress of tension specimens (Deb et al. 2008; Littell 2008). Gilat et al. (2007) investigated the effect of strain rate on the behavior of plain epoxy tension specimens, and concluded that a ductile response was observed at low strain rates, while a brittle response was observed at medium and high strain rates. Adding rubber particles to an epoxy mix was found to increase the deformation capacity before the failure (2004). Imanaka et al. (2009) introduced liquid rubber and cross linked rubber particles to an epoxy mix to enhance its toughness. Another additive was investigated by Zavareh et al. (2012) who included bitumen in the epoxy mix, which resulted in an increase in the toughness without changing the other mechanical properties. Saldanha et al. (2013) tested new epoxy types with enhanced deformation capacity and toughness without the need for including additional
particles. The results show that new epoxies can achieve desired deformation before fracture. Yu et al. (2012) studied both linear (brittle) and nonlinear (ductile) adhesives to characterize their bond slip model considering the adhesive thicknesses and axial strength of CFRP laminates when bonded to steel elements. The experimental tests revealed approximately triangular and trapezoidal bond slip models for brittle and ductile adhesives, respectively. Fernando et al. (2013) evaluated the interfacial fracture energy between CFRP and steel surfaces bonded using ductile nonlinear adhesives and confirmed the trapezoidal bond slip behavior exhibited by ductile adhesives.

It can be seen from the work cited earlier that improving the mechanical properties of epoxy adhesives has gained interest in recent years. Traditional brittle adhesives impose limitations on the efficiency of a very successful structural strengthening technique; i.e., external bonding of FRP composites. The limitation is due to the fact that such strengthening techniques are for the most part controlled by debonding. Therefore, improving the properties of the adhesive translates into improved structural behavior at the member level.

In this paper, SBS is chosen to study the effects of using different adhesive types on the efficiency structural strengthening. First, the mechanical properties of two adhesives were investigated. Both adhesives are then used for stiffening built-up steel beam specimen to enhance their shear capacity. Results from both experimental programs are presented and discussed.

A.1.2 Experimental Program

An experimental program was first devised to determine the mechanical properties of two commercially available epoxy adhesives and how they affect the shear capacity of steel beams. Tensile coupons of cured epoxy specimens were first tested to determine the stress strain behavior of two adhesive types employed in the strengthening of steel beams. The purpose of conducting these experiments was to explore the behavior of steel beams strengthened using the SBS technique and the effect of different adhesive types on its efficiency. Thin-walled I-shaped steel beams were then tested with and without externally bonded GFRP stiffeners to the critical
web panel under shear loading. Furthermore, two web thicknesses were considered to study the effect of initial web slenderness on the efficiency of SBS.

The following sections present the experimental program in more detail:

A.1 2.1 Epoxy Adhesives Considered in this Study

As stated earlier, two types of adhesives were investigated. These were: (1) Tyfo® Saturant Epoxy (Type I) and (2) Tyfo® MB-3 High Performance Adhesive (Type II). Both adhesives are produced by Fyfe Co. and come in two mixable components. For Type I, the final mixture of these components has a relatively lower viscosity around 600-700 cps, and its working time can be 3-6 hours. Depending on the surface orientation and required thickness, the viscosity of Type I may be increased by adding fumed silica (LLC Fyfe Co. 2012). Type II is also supplied from the same manufacturer with two components. Its viscosity is 55,000 cps which is much higher than that of Type I, and has working time, 1-2 hours; relatively shorter than that of Type I (LLC Fyfe Co. 2010). It should be noted that the Type II adhesive is more suitable for bonding metals because of its adhesion properties as well as its higher viscosity that makes its application to smoother vertical surfaces more practical. The mechanical properties of both adhesives obtained from epoxy coupons are presented later.
A. 1 2. 2 Epoxy Preparation and Bonding Procedure

Both Type I and Type II epoxies have two components (Component A and Component B). The components were mixed as per manufacturer’s recommendation (100 A: 42 B for Type I and 100 A: 29.4 B for Type II adhesive by volume). Mixing was done using a special attachment connected to a drill at 400-600 rpm for at least 5 minutes in room temperature until uniform viscosity was obtained. Both epoxy types cured in room temperature. Uniaxial tensile tests were conducted after 47 days for Type I adhesive cured and 14 days for Type II adhesive cured.

The mechanical properties for both epoxy types were obtained by conducting uniaxial tension tests following ASTM 638 Standard Test Method for Tensile Properties of Plastics (ASTM-D638 2003). A rectangular epoxy plate (254 mm [10 in] x 381 mm [15 in]) was formed for each epoxy type for both adhesive types. After the mix cured, dumbbell-shaped coupons were cut from the epoxy plates using a hydrocut waterjet machine. The average specimen thickness measured 6.68 mm (0.263 in.) and 8.79 mm (0.346 in.) for Type I and Type II adhesives, respectively. Each coupon’s thickness and width were measured at three locations prior to testing, and the average of two cross sectional dimensions that were closest to rupture location was used for the calculations. The coupon tests were conducted using MTS 810 Hydraulic Materials Testing machine. The test procedure was programmed in an MTS TestStar II controller via a connected personal computer. Strains were measured using an MTS extensometer with a 25.4 mm [1-inch] gage length. The controller recorded time-stamped test results for the applied tension force, and crosshead movement (stroke) and specimen strains through three input channels. Figure A. I 2 shows the test setup for a coupon specimen after failure, which occurred in the middle third of the constant width region.

A. 1 2. 3 Beam Specimens

Built-up I-shaped steel beams were fabricated for this part of the experimental program. Vertical steel stiffeners were welded to the web and flanges to form panels with 1:1.5 aspect ratio creating a span length of 2,438 mm (8 ft) as can be seen in Figure A. I 3. The steel web plate thickness was selected to make it prone to buckling and ensure that the failure mode will be
shear buckling. Beam tests were carried out under three-point loading. A point load was applied at one-third of the total specimen’s span length from the support; therefore, the shear force that developed in the first panel under the shown loading was twice that acting on the other two panels. Experimental and numerical test results validated the failure mode to be the desired buckling mode occurring in the first panel. The pultruded FRP stiffeners were cut to a T-shaped section by cutting one of fabricated H beam’s flanges. Two commercially available products were used; namely EXTREN 500 and PROForms. Typical mean elastic modulus and ultimate flexural strength of 24,650 and 166 MPa [3,575 and 24 ksi], respectively, were experimentally obtained for the pultruded sections. In addition to the experimental results, other mechanical properties of these sections can be found in manufacturers’ data sheets. Wide flange beam sections 152 mm x 152 mm x 9.5 mm (6 in. x 6 in. x 3/8 in.) were chosen from the available product list.

The stiffener size choice was made using engineering judgment based on previous experimental results due to the lack of any design guidelines for this type of strengthening. The glazed finishing of the bonding surface; i.e., flange, was removed to enhance bonding between the epoxy adhesive and the FRP stiffener. While the stiffener can be diagonally or vertically oriented within the first panel, vertical orientation is preferred for convenience and because earlier studies showed that it results in a more ductile behavior compared to the diagonal orientation. More discussion on stiffener orientation can be found elsewhere (Okeil et al. 2011b).

Both epoxy adhesives, Type I and Type II, were used to bond the T-shaped stiffeners to the steel web. The epoxy bonding procedure is one of the most important steps that can greatly affect the efficiency of the strengthening technique because it is the weakest link of entire system. Baldan (2004) and (Schnerch (2007)) describe the proper surface preparation procedures for different adhesive and substrate types. Dry abrasive process using sand papers on metallic surfaces was followed to increase surface contact area and eliminate or delay debonding.

The standard bonding procedures applied throughout this research can be summarized in the following steps. The preparation of the steel beam specimens started by first cleaning the
surface from any type of contamination such as rust using a poly abrasive wheel. A dry abrasive process was then followed to prepare the surface of the steel beams where the pultruded FRP section would be bonded to increase surface contact area and eliminate or delay debonding. Coarse finishing sand paper was used for this step until the white metal was reached. The FRP surfaces were roughened with a sharp edged metal chisel to a depth smaller than the matrix cover of the first FRP layer. Finally, the roughened surfaces were cleaned using a chemical solvent (acetone) to ensure any debris from the roughening process was removed prior to applying the adhesive. The epoxy adhesive was then applied to the steel web and pultruded FRP surfaces separately, and the pultruded FRP section was placed on the prepared steel web. Small pressure was applied on the stiffener to ensure that the epoxy between the steel web and FRP is dispersed with a uniform thickness along the bonding region. Due to the variation of the initial steel web distortion, the average thickness of the epoxy varied between 2.5 and 3.5 mm, which was relatively thick but was considered practical for real structures where imperfections may not allow for applying a thinner uniform adhesive layer. Dispersed epoxy outside of the FRP flange was scraped with a spatula to avoid creating an artificially thicker web beyond the stiffened region under the FRP section.

Figure A. 12 Uniaxial tension test setup
Table A. I lists the six beams tested, which are divided into two groups based on web thickness. Each group consisted of three beams, namely control beam without SBS stiffeners, and two strengthened beams with both types of epoxy adhesives. As such the test matrix covers the effect of adhesive type on the efficiency of SBS for two web slendernesses. The beams discussed in this paper are designated based on their web thickness ($B_t$) and adhesive type ($E_n$). For example, B1/8-E1 is a specimen with a web thickness equal to 3.2 mm [1/8 in.] and a FRP stiffener bonded using Type I adhesive. Similarly, E0 stands for no adhesive; i.e. unstiffened specimen, and E2 stands for a stiffened specimen stiffened using Type II adhesive.

A. I 3 Experimental Results

A. I 3. 1 Epoxy Tests

Epoxy coupons were tested in uniaxial tension following the ASTM 638-03 standard. The loading procedure was displacement controlled as per the standard. The speed of testing was measured by crosshead movement rate. Two rates were considered in testing both adhesive types. For Type I, the considered rates were 1.27 mm/min [0.05 in./min] and 2.54 mm/min [0.10 in. /min]. For these rates, the test duration ranged between 30 and 50 seconds for the two considered strain rates, which is satisfies the acceptable range of 0.5-5 minutes according to ASTM 638-03. Table A. I 2 lists the measured elastic modulus, rupture stress and maximum elongation for each specimen tested under the two considered rates.

Figure A. I 3 Built-up I shape steel beams

Table A. I 1 lists the six beams tested, which are divided into two groups based on web thickness. Each group consisted of three beams, namely control beam without SBS stiffeners, and two strengthened beams with both types of epoxy adhesives. As such the test matrix covers the effect of adhesive type on the efficiency of SBS for two web slendernesses. The beams discussed in this paper are designated based on their web thickness ($B_t$) and adhesive type ($E_n$). For example, B1/8-E1 is a specimen with a web thickness equal to 3.2 mm [1/8 in.] and a FRP stiffener bonded using Type I adhesive. Similarly, E0 stands for no adhesive; i.e. unstiffened specimen, and E2 stands for a stiffened specimen stiffened using Type II adhesive.

A. I 3 Experimental Results

A. I 3. 1 Epoxy Tests

Epoxy coupons were tested in uniaxial tension following the ASTM 638-03 standard. The loading procedure was displacement controlled as per the standard. The speed of testing was measured by crosshead movement rate. Two rates were considered in testing both adhesive types. For Type I, the considered rates were 1.27 mm/min [0.05 in./min] and 2.54 mm/min [0.10 in. /min]. For these rates, the test duration ranged between 30 and 50 seconds for the two considered strain rates, which is satisfies the acceptable range of 0.5-5 minutes according to ASTM 638-03. Table A. I 2 lists the measured elastic modulus, rupture stress and maximum elongation for each specimen tested under the two considered rates.
For Type II, three coupons were tested at 1.27 mm/min [0.05 in./min] rate and another three were tested at 0.635 mm/min [0.025 in./min] rate. The crosshead movement rate had to be adjusted to ensure that the failure of the coupons occurred within the standard allowed duration of 0.5-5 minutes. The uniaxial tension test results for Type II adhesive are presented in Table A. I 3. Pictures of the coupons can be seen in Figure A. I 4 (a) and (b).

Overall, Type I adhesive exhibited a higher rupture stress (30-33MPa) than Type II (10-12MPa). The results show that the higher strain rate of loading resulted in a higher ultimate rupture stress by 10.75% for Type I and 20.8% for Type II. It should be noted that the rupture strength and elastic modulus results were very consistent as is evident by the computed coefficient of variation, $CV$, which ranged between 2.88% and 7.85%. Compared to traditional civil engineering materials such as concrete and steel (Andrzej et al. 2003)-, the computed value are considered low. The same cannot be said about the elongation at failure, which varied remarkably even within the same group. It was also affected by the loading rate; however, a clear trend could not be established between the strain rate and elongation at failure.

### Table A. I 1 Beams tested with Type I and Type II adhesives

<table>
<thead>
<tr>
<th>Beam labels</th>
<th>Web thickness of steel mm[in]</th>
<th>Adhesive</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1/8</td>
<td>3.2 [1/8]</td>
<td>No FRP</td>
</tr>
<tr>
<td>E0</td>
<td>Type I</td>
<td></td>
</tr>
<tr>
<td>E1</td>
<td>Type II</td>
<td></td>
</tr>
<tr>
<td>E2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B5/32</td>
<td>4.0 [5/32]</td>
<td>No FRP</td>
</tr>
<tr>
<td>E0</td>
<td>Type I</td>
<td></td>
</tr>
<tr>
<td>E1</td>
<td>Type II</td>
<td></td>
</tr>
<tr>
<td>E2</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Table A. I 2 Mechanical properties for adhesive Type I

<table>
<thead>
<tr>
<th>Specimen Number</th>
<th>Rate of Loading</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1.27 mm/min [0.05 in/min]</td>
<td>2.54 mm/min [0.10 in/min]</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Elastic Modulus $E_k$, MPa [ksi]</td>
<td>Rupture Stress $\sigma_u$, MPa [ksi]</td>
<td>Rupture Strain $\varepsilon_u$ (mm/mm)</td>
<td>Elastic Modulus $E_k$, MPa [ksi]</td>
<td>Rupture Stress $\sigma_u$, MPa [ksi]</td>
</tr>
<tr>
<td>$\mu$ (mean)</td>
<td>2575 [373.47]</td>
<td>30.13 [4.37]</td>
<td>0.0129 [0.0017]</td>
<td>2642 [383.15]</td>
<td>33.37 [4.84]</td>
</tr>
<tr>
<td>$\sigma$ (SD)</td>
<td>202 [29.32]</td>
<td>1.72 [0.25]</td>
<td>0.0012 [0.0002]</td>
<td>126 [18.24]</td>
<td>2.07 [0.30]</td>
</tr>
<tr>
<td>$C_V$ (%)</td>
<td>7.85%</td>
<td>5.66%</td>
<td>8.99%</td>
<td>4.76%</td>
<td>6.30%</td>
</tr>
</tbody>
</table>
The stress-strain curves for Type I adhesive are plotted in Figure A. I 5 (a) and (b) for the 1.270 mm/min [0.05 in./min] and 2.54 mm/min [0.10 in./min] strain rates, respectively. The plots show an initial linear trend, the curves deviated slightly at about 55% of the ultimate strength until rupture. The elastic modulus for Type I adhesive was determined using the initial linear trend, which did not vary considerably with different strain rates for this particular adhesive. The stress-strain curves for Type II adhesive are shown in Figure A. I 6. As can be seen, this adhesive exhibited yield-like plateau in metals after an initial linear segment for all coupons. This behavior is not typical for adhesives used in structural strengthening, which normally exhibit a brittle behavior similar to that observed by Type I adhesive. As will be seen later, this ductile behavior has a positive impact on SBS strengthening, and hypothetically on a large class of FRP strengthening of civil infrastructure applications whose failure mode is controlled by debonding. Debonding is often triggered because brittle adhesives cannot resist high stress concentrations at the geometric extremities of the bonding surface. A ductile response such as that exhibited by Type II adhesive will allow redistribution of stresses at such hot spots, thus delaying the debonding mode of failure.
The experimental results obtained from the beam tests confirm that SBS is an effective strengthening approach for thin-walled steel structures. Load-deflection curves are plotted in Figure A. I 7 for all six tested specimens; i.e., unstiffened bare web, stiffened using Type I and Type II adhesives for the two considered web thicknesses. For B1/8 specimens where the web thickness was 3.2 mm (1/8 in.), three load displacement curves are plotted for B1/8-E0 (unstiffened), B1/8-E1 (SBS with Type I adhesive) and B1/8-E2 (SBS with Type II adhesive). The initial stiffness (slope of linear segment at the beginning of the experiment) for B5/32-E2 was relatively lower than B5/32-E1. The authors could not identify the cause of discrepancy. Regardless of this observation, B5/32-E2 gained strength at higher deflections and reached a

<table>
<thead>
<tr>
<th>Specimen Number</th>
<th>Rate of Loading</th>
<th>Rate of Loading</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Elastic Modulus</td>
<td>Rupture Stress</td>
</tr>
<tr>
<td></td>
<td>$E_k$, MPa [ksi]</td>
<td>$\sigma_u$, MPa [ksi]</td>
</tr>
<tr>
<td>$\mu$ (mean)</td>
<td>649 [94.10]</td>
<td>10.1 [1.47]</td>
</tr>
<tr>
<td>$\sigma$ (SD)</td>
<td>31 [4.47]</td>
<td>0.27 [0.04]</td>
</tr>
<tr>
<td>$C_V$ (%)</td>
<td>4.75%</td>
<td>2.88%</td>
</tr>
</tbody>
</table>

### Table A. I 3 Mechanical properties for adhesive Type II

#### A. I 3. 2 Beam Tests

The experimental results obtained from the beam tests confirm that SBS is an effective strengthening approach for thin-walled steel structures. Load-deflection curves are plotted in Figure A. I 7 for all six tested specimens; i.e., unstiffened bare web, stiffened using Type I and Type II adhesives for the two considered web thicknesses. For B1/8 specimens where the web thickness was 3.2 mm (1/8 in.), three load displacement curves are plotted for B1/8-E0 (unstiffened), B1/8-E1 (SBS with Type I adhesive) and B1/8-E2 (SBS with Type II adhesive). The initial stiffness (slope of linear segment at the beginning of the experiment) for B5/32-E2 was relatively lower than B5/32-E1. The authors could not identify the cause of discrepancy. Regardless of this observation, B5/32-E2 gained strength at higher deflections and reached a

![Figure A. I 5 Type I stress-strain curves, (a) 1.27 mm/min [0.05 in./min] strain rate, (b) 2.54 mm/min [0.10 in./min] strain rate](image-url)
capacity that is practically identical to that observed for B5/32-E1. It should be noted that the test was stopped at a relatively low deflection for B5/32-E0. Therefore, the dashed line shown in Figure A. I 7 (b) is an expected behavior based on the previous tests of unstiffened specimens including B1/8-E0 (see Figure A. I 7 (a)). These results further confirm the ability of SBS technique to enhance the shear capacity of thin-walled steel beams. The attained strengthening levels may not be substantially different for beams using either adhesive type, however, it is clear that beams strengthened using Type II adhesive exhibit a more ductile behavior than that exhibited by beams strengthened using Type I adhesives. This ductile behavior is not very common for FRP strengthening techniques and opens the door for new applications.

The maximum load carrying capacities for B1/8 and B5/32 are tabulated in Table A. I 4. The shear strength of B1/8 was improved 36% and 51% with using Type I and Type II adhesive, respectively. B5/32, which was built with a thicker steel web, gained 40% and 39% in shear capacity using Type I and Type II adhesive, respectively. Despite the fact that the increase in capacity of strengthened specimens is almost identical regardless of adhesive type, failure occurred at a much larger displacement for the specimen whose stiffened was bonded using Type II adhesive.
From the plots shown in Figure A. I 7 (a), it can be seen that the proportional limit for the unstiffened specimen is about 200 kN [45 kip] for B1/8-E0, whereas it is 267 kN [60 kip] for B1/8-E1 and B1/8-E2. Hence, strengthening the beams using the SBS technique postpones the initiation of nonlinear behavior, which is directly translated into an increase in the allowable service loading levels. Both strengthened beams in this group behave similarly at their initial yielding region, however, B1/8-E1 resistance drops the load around 8.9 mm [0.35 in.] deflection while B1/8-E2 resistance is maintained up to 17.8 mm [0.70 in.] deflection. Similar results were obtained from B5/32 experiments, and are plotted in Figure A. I 7 (b). Even though the ultimate load carrying capacities of the two strengthened beams were almost identical, B5/32-E1 reached its ultimate capacity at around 10.16 mm [0.40 in] deflection while B5/32-E2 reached its ultimate capacity 22.9 mm [0.90 in] deflection.

Type I adhesive in comparison with Type II adhesive. Figure A. I 7 (b) shows a similar increase in the initial stiffness was observed for specimens with thicker web; i.e., B5/32, for the case when the FRP stiffener was bonded using Type I adhesive. However, the difference between the initial stiffness of the stiffened and unstiffened beams is relatively smaller that the B1/8. One exception to this trend is again the behavior of B5/32-E2. It can be seen from the plot in Figure A. I 7 (b) that the initial stiffness of Type II is even less than that of B5/32-E0, unstiffened case. The expected slope falls between the Type I and No FRP plots in Figure A. I 7 (b). As stated earlier, this lower stiffness can be attributed to many factors, however, the authors

<table>
<thead>
<tr>
<th>Beam Labels</th>
<th>SBS</th>
<th>Failure Load kN[kips]</th>
<th>Capacity Increase</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1/8</td>
<td>E0</td>
<td>No FRP</td>
<td>222 [50.0]</td>
</tr>
<tr>
<td></td>
<td>E1</td>
<td>Type I</td>
<td>302 [68.0]</td>
</tr>
<tr>
<td></td>
<td>E2</td>
<td>Type II</td>
<td>336 [75.5]</td>
</tr>
<tr>
<td>B5/32</td>
<td>E0</td>
<td>No FRP</td>
<td>294 [66.0]</td>
</tr>
<tr>
<td></td>
<td>E1</td>
<td>Type I</td>
<td>411 [92.5]</td>
</tr>
<tr>
<td></td>
<td>E2</td>
<td>Type II</td>
<td>409 [92.0]</td>
</tr>
</tbody>
</table>
will not speculate on the cause and will consider it an anomaly until the behavior is confirmed with further testing.

Unstiffened beams; i.e., B1/8-E0 and B5/32-E0, failed in a traditional way by buckling of the exterior web panel that is subjected to the higher shear forces. A single buckling wave extended from the corners of the panel in a classical shape as can be seen Figure A. I 8 (a). The mode of failure of the tested beams that were stiffened was different than that of the unstiffened beams. In the case of B1/8-E2, no indication of failure was observed during the test until adhesive cracking noise was heard. Soon thereafter, the rate of cracking noise increased gradually, which was accompanied by a lower rate of increase in the applied load. As this stage progressed, buckling of the steel web was visible on one side of the stiffener in the first panel. After the ultimate capacity was reached, large portions of the stiffener were completely debonded from the steel web panel and the buckling propagated along the diagonal of the web panel. This is when a major sudden drop in the applied load occurred as can be seen in the in load-deflection curves. Figure A. I 8 (b) shows the critical web panel for with FRP stiffeners bonded using Type II adhesive for the beam B5/32, which behaved differently. For these specimens, no or very limited cracking noises could be heard. This indicates that the bond between the FRP stiffener and the steel web panel was maintained throughout the test. As such, the web panel remained divided into two regions due to the stiffening effect of the FRP stiffener,
which limited the buckling of the steel web to one half of the panel as shown in Figure A. I 8 (b). This led to higher straining of the FRP stiffener at its extremities where the buckling wave in the divided panel ended; i.e., at the corners of the FRP stiffener. The edges of the FRP stiffener flange underwent large local deformations in its attempt to resist the web panel from buckling. Figure A. I 9 shows the local separation between the flange of the FRP stiffener and the adhesive at the end of the test for B5/32-E2. Despite this separation, a sudden drop in the applied load was not observed for B1/8-E2 and B5/32-E2 as it was for Beams B1/8-E1 and B5/32-E1. It should be

Figure A. I 9 Excessive straining at bottom flange corner of FRP stiffener
noted that in some other tests that were not reported in this paper, separation did not occur and instead, interlaminar failure took place in the FRP flange.

A. I 4 Conclusions

In this study, the mechanical properties of two adhesive types subjected to uniaxial tension were investigated. The adhesives were then employed in a new structural strengthening technique referred to as SBS that was developed by the research team to investigate their effect on the performance of shear behavior of thin-walled steel beams.

Based on the experimental results presented in this paper, the following conclusions can be drawn:

1. Type I adhesive is a more brittle material in tension than Type II adhesive. Type I coupons do not have a noticeable yielding region such as that observed for Type II adhesive, and fail suddenly. The ductility of Type II adhesive is not typical for adhesive types used in structural strengthening applications. In addition to the ductility of Type II adhesive, its elastic modulus is lower than that of Type I adhesive.

2. Both adhesive types were successful in enhancing the shear strength of built-up steel beams by up to 51% of the original capacity. These results further confirm the efficiency of SBS in strengthening thin-walled steel structures.

3. The failure mode for beams strengthened using Type II adhesive is different than that for unstrengthened beams in that buckling occurred in a smaller panel (half the size of the unstrengthened panel).

4. Beams strengthened using Type II adhesive exhibit more ductile behavior than beams with Type I adhesives.

5. Initial stiffness slopes of the stiffened beams enhanced when SBS was applied to the thin walled steel beams. The increase of the initial stiffness is slightly higher when Type I adhesive was used.
6. For the specimens where the stiffener fully debonded, the buckling mode changed abruptly finally reaching conditions similar to unstrengthened beams, which was accompanied by a large drop in applied load.

The authors acknowledge that the limited number of experiments in this paper is not enough to reach a full understanding of SBS method. Therefore, further investigation using validated finite element analysis is needed for better understanding of effective parameters such as FRP strength and bond area in order to be able to optimize the strengthening system and to derive reliable design procedure. Improvements to the adhesion performance can also be investigated using the developed model to avoid premature adhesive failures.

A. I 5 Nomenclature

\[ C_V \quad \text{Coefficient of variation} \]
\[ E_k \quad \text{Elastic modulus of adhesives} \]
\[ \text{FRP} \quad \text{Fiber Reinforced Polymer} \]
\[ \text{GFRP} \quad \text{Glass Fiber Reinforced Polymer} \]
\[ SBS \quad \text{Strengthening-By-Stiffening} \]
\[ \varepsilon_u \quad \text{Rupture strain of adhesives} \]
\[ \mu \quad \text{Mean value} \]
\[ \sigma \quad \text{Standard deviation} \]
\[ \sigma_u \quad \text{Rupture stress of adhesives} \]

A. I Acknowledgements

This research is sponsored in part by the National Science Foundation (CMMI# 1030575). The donation of materials by Fyfe Co., LLC, and Bedford Reinforced Plastics, Inc. in addition to support from Strongwell Corporation are greatly appreciated. Additional support from the Department of Civil and Environmental Engineering at Louisiana State University is also acknowledged. Any opinions, findings, and conclusions or recommendations expressed in this material are those of the authors and do not necessarily reflect the views of the sponsoring agencies.
A. I  References


ACI Committee 440 (2007). "Report on Fiber-Reinforced Polymer (FRP) Reinforcement for Concrete Structures." ACI 440R-08, American Concrete Institute Farmington Hills, MI.


Fernando, D., Yu, T., and Teng, J. (2013). "Behavior of CFRP Laminates Bonded to a Steel Substrate Using a Ductile Adhesive." Journal of Composites for Construction, 0(0), 04013040.


### Appendix II. Letters of Permissions

#### A. II 1 Chapter 2. Effect of Initial Panel Slenderness on Efficiency of Strengthening-By-Stiffening Using FRP for Shear Deficient Steel Beams

**ELSEVIER LICENSE TERMS AND CONDITIONS**

**May 10, 2016**

This is a License Agreement between Tuna Ulger ("You") and Elsevier ("Elsevier") provided by Copyright Clearance Center ("CCC"). The license consists of your order details, the terms and conditions provided by Elsevier, and the payment terms and conditions.

All payments must be made in full to CCC. For payment instructions, please see information listed at the bottom of this form.

<table>
<thead>
<tr>
<th>Supplier</th>
<th>Elsevier Limited</th>
</tr>
</thead>
<tbody>
<tr>
<td>Registered Company Number</td>
<td>1982084</td>
</tr>
<tr>
<td>Customer name</td>
<td>Tuna Ulger</td>
</tr>
<tr>
<td>Customer address</td>
<td>Louisiana State University</td>
</tr>
<tr>
<td>License number</td>
<td>3865510822329</td>
</tr>
<tr>
<td>License date</td>
<td>May 10, 2016</td>
</tr>
<tr>
<td>Licensed content publisher</td>
<td>Elsevier</td>
</tr>
<tr>
<td>Licensed content publication</td>
<td>Thin-Walled Structures</td>
</tr>
<tr>
<td>Licensed content title</td>
<td>Effect of initial panel slenderness on efficiency of Strengthening-By-Stiffening using FRP for shear deficient steel beams</td>
</tr>
<tr>
<td>Licensed content author</td>
<td>Tuna Ulger, Ayman M. Okeil</td>
</tr>
<tr>
<td>Licensed content date</td>
<td>August 2016</td>
</tr>
<tr>
<td>Licensed content volume number</td>
<td>105</td>
</tr>
<tr>
<td>Licensed content issue number</td>
<td>n/a</td>
</tr>
<tr>
<td>Number of pages</td>
<td>9</td>
</tr>
</tbody>
</table>
Appendix I. Effect of Adhesive Type on Strengthening-By-Stiffening for Shear-deficient Thin-walled Steel Structures

ELSEVIER LICENSE

TERMS AND CONDITIONS

May 10, 2016

This is a License Agreement between Tuna Ulger ("You") and Elsevier ("Elsevier") provided by Copyright Clearance Center ("CCC"). The license consists of your order details, the terms and conditions provided by Elsevier, and the payment terms and conditions.

All payments must be made in full to CCC. For payment instructions, please see information listed at the bottom of this form.
<table>
<thead>
<tr>
<th>Supplier</th>
<th>Elsevier Limited</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>The Boulevard, Langford Lane</td>
</tr>
<tr>
<td></td>
<td>Kidlington, Oxford, OX5 1GB, UK</td>
</tr>
<tr>
<td>Registered Company Number</td>
<td>1982084</td>
</tr>
<tr>
<td>Customer name</td>
<td>Tuna Ulger</td>
</tr>
<tr>
<td>Customer address</td>
<td>Louisiana State University</td>
</tr>
<tr>
<td></td>
<td>BATON ROUGE, LA 70808</td>
</tr>
<tr>
<td>License number</td>
<td>3865511232959</td>
</tr>
<tr>
<td>License date</td>
<td>May 10, 2016</td>
</tr>
<tr>
<td>Licensed content publisher</td>
<td>Elsevier</td>
</tr>
<tr>
<td>Licensed content publication</td>
<td>International Journal of Adhesion and Adhesives</td>
</tr>
<tr>
<td>Licensed content title</td>
<td>Effect of adhesive type on Strengthening-By-Stiffening for shear-deficient thin-walled steel structures</td>
</tr>
<tr>
<td>Licensed content author</td>
<td>A.M. Okeil, T. Ulger, H. Babaizadeh</td>
</tr>
<tr>
<td>Licensed content date</td>
<td>April 2015</td>
</tr>
<tr>
<td>Licensed content volume number</td>
<td>58</td>
</tr>
<tr>
<td>Licensed content issue number</td>
<td>n/a</td>
</tr>
<tr>
<td>Number of pages</td>
<td>8</td>
</tr>
<tr>
<td>Start Page</td>
<td>80</td>
</tr>
<tr>
<td>End Page</td>
<td>87</td>
</tr>
<tr>
<td>Type of Use</td>
<td>reuse in a thesis/dissertation</td>
</tr>
<tr>
<td>Intended publisher of new work</td>
<td>other</td>
</tr>
<tr>
<td>Portion</td>
<td>full article</td>
</tr>
<tr>
<td>Format</td>
<td>both print and electronic</td>
</tr>
<tr>
<td>Are you the author of this Elsevier article?</td>
<td>Yes</td>
</tr>
<tr>
<td>Will you be translating?</td>
<td>No</td>
</tr>
</tbody>
</table>
Title of your thesis/dissertation

STRENGTHENING SHEAR DEFICIENT THIN-WALLED STEEL BEAMS BY BONDING PULTRUDED GFRP SECTIONS

Expected completion date

Jul 2016

Estimated size (number of pages)

200

Elsevier VAT number

GB 494 6272 12

Permissions price

0.00 USD

VAT/Local Sales Tax

0.00 USD / 0.00 GBP

Total

0.00 USD

Terms and Conditions
Vita

Tuna Ulger was born in Nevsehir, Turkey, in 1984. He graduated from Nevsehir Anatolian High School. Then, he enrolled in Ege University and graduated in 2007 with a Bachelor of Science in Civil Engineering. After graduation, he worked as a site engineer for two years. Then, he decided to pursue a doctoral degree and applied for and was awarded a scholarship from the Turkish Education Ministry. First he enrolled at Texas A&M University where he earned his master’s degree in structural engineering in 2012. Finally, he joined Louisiana State University to pursue doctoral degree in Civil Engineering. He expects to receive his Doctor of Philosophy Degree in Civil Engineering in August 2016.