RETROFITTING OF COMPOSITE STEEL BEAMS PRE-DAMAGED IN FLEXURE USING FIBER REINFORCED POLYMERS

by

Ehab Clovis Karam

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Approval Signatures

We, the undersigned, approve the Master’s Thesis of Ehab Clovis Karam

Thesis Title: Retrofitting of Composite Steel Beams Pre-Damaged in Flexure using Fiber Reinforced Polymers

<table>
<thead>
<tr>
<th>Signature</th>
<th>Date of Signature (dd/mm/yyyy)</th>
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</thead>
</table>
| Dr. Rami Hawileh  
Associate Professor, Department of Civil Engineering  
Thesis Advisor | _____________________________ |
| Dr. Jamal Abdalla  
Professor, Department of Civil Engineering  
Thesis Co-Advisor | _____________________________ |
| Dr. Tamer El Maaddawy  
Associate Professor, Department of Civil and Environmental Engineering  
UAE University  
Thesis Co-Advisor | _____________________________ |
| Dr. Mohammad Al Hamaydeh  
Associate Professor, Department of Civil Engineering  
Thesis Committee Member | _____________________________ |
| Dr. Essam M. Wahba  
Associate Professor, Department of Mechanical Engineering  
Thesis Committee Member | _____________________________ |
| Dr. Aliosman Akan  
Head, Department of Civil Engineering | _____________________________ |
| Dr. Mohamed El-Tarhuni  
Associate Dean, College of Engineering | _____________________________ |
| Dr. Leland Blank  
Dean, College of Engineering | _____________________________ |
| Dr. Khaled Assaleh  
Director of Graduate Studies | _____________________________ |
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Abstract

The demand for the use of carbon-fiber-reinforced polymer (CFRP) composite sheets and plates is exponentially increasing in the rehabilitation of deteriorating steel-concrete composite beams across the Globe. The aim of this study is to investigate the performance of pre-damaged steel-concrete composite beams retrofitted with CFRP sheets and mechanically anchored with pultruded composite plates with high stiffness and bearing strength. A total of 10 composite steel-concrete beams were prepared and tested under two-point loading till failure. One beam was left undamaged to serve as a control specimen, while the remaining beams were divided into three groups, each consisting of three specimens and artificially damaged by cutting different notch depths of 5 mm, 8 mm and 11 mm in the bottom flange simulating corrosion damage levels of 45%, 73% and 100%, respectively. In each group, the first beam was tested without strengthening, the second specimen was externally retrofitted in flexure with CFRP sheets bonded with epoxy adhesives, and the third beam was strengthened in a similar fashion and mechanically fastened using the proposed composite pultruded plate system. The test results showed that the load-carrying capacity of the deteriorated specimens with different notch depths of 5 mm, 8 mm and 11 mm was reduced by 10.57, 21.52, and 49.1%, respectively. The strength of the repaired beam specimens with CFRP sheets ranged from 74.19 to 104.48% of that of the control undamaged beam, and ranged from 77.42 to 108.06% for the beams repaired with the proposed mechanical anchorage system. It was concluded that the proposed mechanically fastened repair system could fully restore the strength of damaged steel-concrete beams if the notch size at midspan is less than 50% of the bottom flange thickness. A finite element (FE) model was also developed that accurately simulated the response of some selected specimens. The developed models can be used in a future parametric study to investigate the effect of several parameters on the performance of the proposed retrofitting system.

Search Terms: Flexural repair, steel-concrete composite beams, rehabilitation, girders, carbon fiber reinforced polymers, plates.
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Chapter 1: Introduction

1.1 Background

Many steel structures are now considered to have an alarming capacity deficiency due to the constant increase of live load and the never ending phenomenon of steel corrosion. Accordingly, a large number of structural members such as beams and columns in bridges, offshore platforms, and buildings need repair and strengthening. Many conventional solutions can be presented to strengthen steel beams in flexure such as replacing the member itself or attaching external steel plates to the soffit of the damaged member. The first solution is very costly, time consuming and requires skilled labor. The second solution of adding steel plates would not tackle the fundamental problems of durability since the steel plate will be corroded eventually and lose its characteristics. In addition, the attached steel plates may induce fatigue cracking at the edges of the added plates.

Fiber Reinforced Polymer (FRP) composite materials are found to be the material of choice in rehabilitating and strengthening steel beams due to their lightweight, high strength, fatigue and corrosion resistance properties, and ease of externally attaching them to steel surfaces via epoxy adhesives. FRP strengthening systems have proven to be effective in enhancing both the flexural and shear capacity of steel beams. Typically, carbon-FRP (CFRP) composite sheets and laminates are used to externally strengthen steel beams in flexure, due their high tensile strength (about three times the tensile strength of steel) and elastic modulus. In addition, CFRP composite materials have superior fatigue and corrosion resistance properties.

CFRP plates or sheets are typically attached to the flexural surface of damaged steel members to increase the beam’s stiffness and flexural capacity. The bond between the CFRP sheets and steel is normally assured through the use of epoxy adhesives. However, if the bond between the CFRP and steel surfaces is gone, then the composite action of the strengthening system is lost, and that would lead to a sudden failure of the strengthened beam specimen. Thus the bond between the steel and CFRP surfaces should be thoroughly investigated experimentally and numerically, before this strengthening technology is fully utilized in strengthening damaged steel structural members. This research will investigate the performance of
pre-damaged steel beams when externally strengthened with CFRP composite sheets and mechanically fastened with pultruded composite strips.

1.2 Research Significance

Many existing steel structures, such as bridges and buildings require strengthening and retrofitting. Induced fatigue-crack propagation and corrosion significantly affect the durability and life span of steel members. The state of the art technology of repairing and retrofitting deteriorating steel beams is by externally attaching CFRP sheets to the beam’s soffit via epoxy adhesives. The main concern is the bond between the CFRP sheets and steel surfaces that will assure the full utilization of the composite action in the strengthening system. Debonding of the CFRP sheets from the steel surface could be enhanced by anchoring the sheets at specified locations within the beam’s shear span using different mechanical systems. The current state of the art is to anchor the CFRP sheets to the flexural flange of the beam using CFRP plates and steel bolts. The main disadvantage of such mechanically anchored systems is the deterioration and associated imperfections of the CFRP plate material around the holes, thus creating zones of stress concentrations that will lead to premature failure of the plate.

The research significance of this study is to investigate the performance of pre-damaged steel beams when externally strengthened with CFRP composite sheets and mechanically fastened with pultruded composite strips, composed of carbon tows sandwiched between layers of fiberglass mats and rovings. To the author’s best knowledge, the literature is lacking information on the performance of damaged steel beams when retrofitted using such strengthening systems. Thus, this research will shed light and answer many questions related to the improvement of the bond performance between the CFRP composite sheets and steel surfaces.

1.2 Research objectives

This study aims to investigate the effectiveness of using externally bonded CFRP sheets with mechanically fastened pultruded composite plates in retrofitting pre-damaged composite steel beams. The main objectives of this research are to:

1. Investigate the performance of pre-damaged steel-concrete composite beams with different levels of damage in the tension flange at midspan.
2. Study the effectiveness of using CFRP composite sheets in repairing the pre-damaged steel beams.
3. Examine the effect of the number of CFRP layers and bond length used in the repair of the pre-damaged steel beams.
4. Investigate the performance of the repaired steel beams with CFRP composite sheets when mechanically fastened using flexible pultruded composite plates.
5. Study the effect of using mechanical anchorage along the length of CFRP on flexural strength gain and ductility.
6. Develop three-dimensional finite element (FE) models using the finite element software, ANSYS that can simulate the response of a selected group of tested specimens.

1.3 Thesis Organization

The contents of this thesis are briefly summarized below:

Chapter 1 presents the background, research significance and objectives of this thesis. Chapter 2 presents a literature review related to the repair and strengthening of steel structures in flexure. Chapter 3 describes the experimental program and the test procedure conducted in the Construction Laboratory of the American University of Sharjah. Chapter 4 presents the results of the beam tests, including the load-midspan deflection responses, strain gauge results, along with the associated failure mode of the tested specimens. Chapter 5 presents the development of the FE models that captured the response of a selected group of specimens. Chapter 6 presents a summary of the work conducted in this thesis along with the conclusions drawn from the experimental and numerical results. Recommendations for future research are also presented in this chapter.
Chapter 2: Literature Review

Over the past few years, there have been a lot of studies on the use of fiber reinforced polymer (FRP) sheets and plates in strengthening reinforced concrete structural members. This led many researchers to widen their scope and investigate the possibility of strengthening structural steel members with composite FRP plates and sheets. Photiou et al. [1] studied the behavior of a hybrid combination of carbon/glass (CFRP/GFRP) fiber reinforced composite sheets placed at the tension face of artificially damaged steel beams. Two types of unidirectional CFRP composite sheets were used: one with a high elastic modulus (HM-CFRP) that had almost an identical amount of stiffness as that of steel, and the other with an ultra-high modulus (UHM-CFRP), that had higher stiffness than steel. Two geometrical shapes for the composite system were studied. One was a normal flat composite system, and the other was a U-shaped system where the GFRP extended to the modified axis of the beam. Four rectangular hollow section (RHS) beams were tested in the experimental program. Two beams had UHM-CFRP laminates with an alternative geometrical shape of composites. The other two beams had HM-CFRP with an alternative geometrical shape of composites. The test results showed that the UHM-CFRP-U-shaped hybrid composite system failed at the location of maximum moment in the CFRP, were it reached its ultimate strain without bond failure. Nevertheless, the capacity of the artificially damaged reinforced beam exceeded the full RHS beam at normal state. The HM-CFRP-U-Shaped composite exhibited better capacity than that of the UHM-CFRP-U-shaped system and the CFRP did not fail. The UHM-CFRP-flat shaped composite exhibited similar results to that of the U-shaped composite system but the CFRP composite failed and debonding occurred. Surprisingly, the HM-CFRP-flat shaped composite exhibited similar results to that of the HM-CFRP-U-Shaped composite without any failure or debonding in the CFRP composites.

Tavakkolizadeh and Saadatmanesh [2] studied the effect of CFRP composite sheets on steel-concrete composite girders. The beams were artificially damaged to three different levels which consisted of cutting 25, 50 and 100% of the cross sectional area of the tensile flange at midspan. For the 25% pre-damaged beam, the concrete had a compressive strength of 29.1 MPa and was reinforced with one layer of CFRP composite sheet at the tensile flange. The load-carrying capacity of the
repaired beam increased by 120% of the original pre-damaged beam. The 50% pre-damaged beam had a concrete topping with a compressive strength of 16.6 MPa, and was reinforced with three layers of CFRP composite sheets at the face of the tensile flange and the results showed an increase in load-carrying capacity of 180% to that of the pre-damaged beam. As for the 100% pre-damaged composite beam with a concrete topping that had a compressive strength of 29.1 MPa and was strengthened with five layers of CFRP sheets, the results showed an increase in load-carrying capacity of 110% to that of the pre-damaged beam. In all cases, debonding of the CFRP composite sheets was the major mode of failure in the tested specimens. The elastic stiffness of the girders was also improved with results exceeding that of the original pre-damaged beam by more than 85%.

Al-Saidy et al. [3] studied the effectiveness of using CFRP plates to repair steel-concrete composite beams. A total of six beams were tested with changing levels of artificial damaging. CFRP reinforcement was applied on the face of only the tensile flange only or to the tensile flange and the web. The beams were artificially damaged at their midspan, to 50% or 75% of their tensile flange and repaired with CFRP plates. The results showed an increase in the overall stiffness and load-carrying capacity of the repaired specimens. It was concluded that damaged beams can restore their original capacity (strength of original beam) if properly repaired and strengthened with CFRP plates.

Shaat and Fam [4] tested six artificially damaged composite steel-concrete beams. Five beams were severely damaged by removing their tensile flanges at the beam’s midspan using a saw cut. The results indicated a loss of 60% and 54% in the strength and stiffness, respectively. Four beams were repaired by attaching CFRP composite sheets to their tensile flanges to restore their strength and stiffness. The repaired beam specimens failed by the initiation of debonding of the CFRP sheets from the steel surfaces. Overall, the stiffness of the repaired beams was restored and was negligibly affected by the bond length of the CFRP sheets. However, the load-carrying capacity of the repaired beams was affected by the bond length of the CFRP sheets and ranged from 72% to 116% of the control undamaged beam specimen.

In another study, Fam et al. [5] investigated the effect of the elastic modulus of CFRP sheets on enhancing the flexural performance of steel-concrete composite
girders, pre-damaged with different levels and configurations in the tension flange. The results indicated an increase in the overall strength and stiffness of the repaired specimens over the damaged ones by 51% and 19%, respectively. In addition, the repaired beams restored the load-carrying capacity of the damaged specimens up to 79%. The authors also developed an analytical model that was able to predict the response of the tested specimens. It was concluded that CFRP sheets with higher elastic modulus would result in an increase in the overall stiffness and reduction in the strength, due to the reduction in the tensile strength of the CFRP sheets with higher elastic modulus at the same strain level in the FRP laminates.

Another study on the flexural performance of steel girders retrofitted using CFRP materials conducted by Galal et al. [6] showed that an anchorage system is needed when repairing damaged steel beams using CFRP composite sheets. The investigators used CFRP composite sheets and plates to strengthen the pre-damaged beam specimens. The mode of failure was either debonding or rupture at higher loads than those of the pre-damaged beam specimens. The proposed ductile anchorage system by the investigators showed that it would increase both the strength and ductility of the repaired specimens and could reduce early debonding or peeling of the CFRP composite sheets.

Another study Hmidan et al. [7] conducted an experimental program that investigated the effect of the initial damage level of notched steel beams and their repair using CFRP composite sheets. The authors had three different notch levels at the tensile flange. The experimental results were modeled and simulated using the finite element (FE) software ANSYS (2013). The modes of failure varied between web cracking, partial debonding, complete debonding, and rupture of CFRP sheets. An increase in ultimate load in all tested beams was observed, especially with the increasing level of damage. It was also observed that the degree of damage of the steel beam did not affect its mode of failure. In addition, the level of damage influenced the rate of web cracking in the repaired specimens. The developed FE model was capable of predicting the response of the tested specimens.

In another study, Ghafoori et al. [8] examined the difference between the use of bonded and prestressed-unbonded CFRP plate fatigue strengthening systems in
repairing damaged steel beams. The test results showed that both repair systems have similar load-carrying capacities but failed in different modes.

Deng et al. [9] developed analytical and FE numerical models that were able to predict the flexural strength of strengthened composite beams with CFRP composites. The developed models were validated via comparison with experimental data published in the literature. Good matching between the predicted and measured load versus midspan deflection results was obtained at all stages of loading till failure of the beam specimens. It was also concluded that the flexural strength is not influenced by the permanent load nor the prestressing force, especially when failure is controlled by concrete crushing.

Narmashiri et al. [10] investigated experimentally and numerically the flexural performance of twelve externally strengthened steel I-beams with CFRP composite sheets. It was observed that load-carrying capacity increases by increasing the thickness and length of the CFRP plate. However, a thicker plate would result in premature debonding of the CFRP plate. Moreover, using a length longer than the CFRP effective length is not effective in increasing the flexural strength of the repaired specimens.

In another study, Sallam et al. [11] investigated the flexural behavior of strengthened steel-concrete composite beams by CFRP composite sheets or with bonded steel plates. The CFRP sheets were attached to the tension flange of the steel beams using two different configurations, namely with wrapped CFRP sheets around the flange and with wrapped CFRP sheets around the flange and part of the web of the steel beam. In addition, two beams were strengthened without bonding 50% of the CFRP sheets along the length of the tension flange. It was observed from the experimental results that the CFRP strengthening systems enhanced the load-carrying capacity of the pre-damaged steel beams up to 22.5%. However, the increase in the yield strength of the repaired composite beams was up to 32.2% of the damaged specimens. In addition, it was observed that the damaged steel beams failed due to the opening of a notch that was followed by propagation of cracks in the web towards the upper flange of the steel beam. The repaired specimens failed by local debonding of the CFRP sheets at the notch location. It was concluded that the CFRP composite sheets managed to reduce the crack initiation and delay crack propagation along the
web of the steel section. In addition, it was observed that only after yielding, the beams strengthened with partially bonded CFRP sheets showed a different behavior and lower strength that their counterpart with fully bonded CFRP sheets along the bottom flexural flange.

Corrosion by itself is a great danger to steel. However, if carbon is present with steel, then corrosion can be a far greater threat due to the expediting rate of the corrosion process. Tavakkolizaded et al. [12] discussed the serious threat of galvanic corrosion of carbon and steel in aggressive environments. The extensive research conducted by the authors showed that carbon (CFRP) and steel indeed exhibit galvanic corrosion tendencies. Nevertheless, these tendencies can be significantly reduced through the use of epoxy adhesives.

Other similar studies (Zhao and Zhang [13], Zhao and Li [14], Akbar et al. [15], Seleem et al. [16], Kim and Harries [17], Sweedan et al. [18], Lenwari et al. [27]) evaluated experimentally and numerically the flexural performance of steel beams externally strengthened with FRP composite materials. However, debonding of the CFRP laminate from the steel surface of the bottom steel flange in those studies was the major mode of failure in the repaired specimens. This study aims to investigate the effect of retrofitting pre-damaged composite steel-concrete beams that resemble a localized corrosion application with bonded CFRP composite sheets. The CFRP sheets will be attached to the soffit of the tensile flange via epoxy adhesives and mechanically fastened at specified locations within the beam’s shear span with flexible pultruded composite plates. Thus, the CFRP-sheets will be bonded using adhesive material to the steel flange and mechanically fastened by a hybrid material that does not cause galvanic corrosion if in contact with steel.
Chapter 3: Experimental Program

3.1 Test Specimen Properties
A series of ten steel-concrete composite beams were fabricated in this study to examine the effectiveness of using the proposed externally hybrid bonded-mechanically anchored CFRP composite system to retrofit pre-damaged composite steel girders. The section type of the steel beam used is UC 203x203x46, with a total length of 2 m and clear span of 1.7 m between supports. Figure 1 shows the details of a typical sample of the tested composite beam specimen. Stiffeners were placed at the location of applied loads and at support as shown in Figure 1. A 100 mm thick and 450 mm wide concrete cover was casted monolithically using shear studs on the top compression flange as shown in Figure 2.

Figure 1. Typical steel-concrete composite beam detail

Figure 2. Typical steel-concrete cross section
3.2 Material Properties

3.2.1 Steel Beam

This section describes the material properties of the UC 203 x 203 x 46 steel section used in the test. The depth and width of the steel section is 203.2 mm and 203.6 mm, respectively. The thickness of the web and flange is 7.2 mm and 11 mm, respectively. The second moment of area is along axis x-x is $45.68 \times 10^6 \text{ mm}^4$ and the cross sectional area of the steel beam is $0.587 \times 10^6 \text{ mm}^2$.

A tensile coupon test conforming to BS EN 10002-1: 2001 [19] was performed on four samples of the web in order to find the actual mechanical properties of the steel section. Figure 3 shows the samples which were cut to a 300x45mm shape. The samples were then grooved and prepared for testing as shown in Figure 4.

![Figure 3. Coupon test samples](image)

![Figure 4. Coupon test sample dimension](image)

The machine used to perform the tensile test was an INSTRON 8801-Servo hydraulic dynamic testing machine. The tests were conducted at a loading rate of 10
mm/min. Table 1 shows the yield and tensile strength of each tested specimen. It is observed from Table 1 that the average yield and ultimate strength of steel is 318.75 MPa and 459 MPa, respectively. Figure 5 shows the stress-strain curves of the tested specimens.

Table 1: Tensile test results

<table>
<thead>
<tr>
<th>Specimen Name</th>
<th>Yield Strength (MPa)</th>
<th>Tensile Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>W1</td>
<td>288</td>
<td>430</td>
</tr>
<tr>
<td>W2</td>
<td>310</td>
<td>475</td>
</tr>
<tr>
<td>W3</td>
<td>330</td>
<td>456</td>
</tr>
<tr>
<td>W4</td>
<td>347</td>
<td>475</td>
</tr>
<tr>
<td>Average</td>
<td><strong>318.75</strong></td>
<td><strong>459</strong></td>
</tr>
</tbody>
</table>

Figure 5: Stress vs strain of steel section

3.2.2 Concrete Material Properties

Normal weight concrete was used to cast the 100 mm thick slab monolithically with the top UC flange. Several standard concrete cylinders with 100 mm diameter and 200 mm in height were casted at the same time of beam slab casting. The cylinders were tested after 28 days and the average compressive strength of the tested cylinders was found to be 20.1 MPa.
3.2.3 CFRP Sheet Material Properties

The carbon fiber reinforced polymer (CFRP) used to strengthen the tensile flange of the composite beam was SikaWrap® - 300C [20]. Table 2 shows the material properties of the sheets which were obtained from the technical data sheet of the manufacturer.

Table 2: CFRP Sheet Properties

<table>
<thead>
<tr>
<th>Fiber Type</th>
<th>High strength carbon fibers</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fiber Orientation</td>
<td>0° (Unidirectional)</td>
</tr>
<tr>
<td>Strain at break of fibers</td>
<td>1.5% (nominal)</td>
</tr>
<tr>
<td>Areal Weight</td>
<td>300 g/m² ± 5%</td>
</tr>
<tr>
<td>Fiber Density</td>
<td>1.8 g/m³</td>
</tr>
<tr>
<td>Fabric Design Thickness</td>
<td>0.17 mm (based on total carbon content)</td>
</tr>
<tr>
<td>Tensile Strength of Fibers</td>
<td>3900 N/mm² (nominal)</td>
</tr>
<tr>
<td>Tensile E-Modulus of Fibers</td>
<td>230000 N/mm² (nominal)</td>
</tr>
</tbody>
</table>

Five coupon tensile tests were performed according to ASTM D3039 [26] on CFRP laminates composed of one layer of CFRP sheet that was embedded between two layers of epoxy adhesives at a loading rate of 2 mm/min. The samples were prepared and the thickness was measured after curing the epoxy at six different locations within each sample and the width was measured at three different locations before the average was evaluated. Figure 6 shows a plot of stress versus strain of each specimen tested. Figure 6 shows that each specimen behaved in a similar fashion with an elastic brittle behavior due to micro-cracking of the epoxy.

Figure 6. Stress vs. strain of CFRP Sheets
Table 3 shows a summary of the mechanical properties of tested CFRP sheets including the tensile strength ($\sigma_u$), modulus of elasticity ($E$), ultimate load ($P_u$) and percent elongation at failure ($\%\delta$).

Table 3: Summary of results for CFRP tensile test.

<table>
<thead>
<tr>
<th></th>
<th>Average Width (mm)</th>
<th>Average thickness (mm)</th>
<th>$\sigma_u$ (Mpa)</th>
<th>$E$ (GPa)</th>
<th>$P_u$ (kN)</th>
<th>$%\delta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>40.0</td>
<td>0.625</td>
<td>753</td>
<td>65.88</td>
<td>19.05</td>
<td>1.42</td>
</tr>
<tr>
<td>2</td>
<td>40.44</td>
<td>0.633</td>
<td>762</td>
<td>65.93</td>
<td>19.07</td>
<td>1.438</td>
</tr>
<tr>
<td>3</td>
<td>39.9</td>
<td>0.617</td>
<td>810</td>
<td>66.07</td>
<td>20.52</td>
<td>1.48</td>
</tr>
<tr>
<td>4</td>
<td>39.95</td>
<td>0.67</td>
<td>672</td>
<td>65.85</td>
<td>18</td>
<td>1.411</td>
</tr>
<tr>
<td>5</td>
<td>40.91</td>
<td>0.617</td>
<td>686</td>
<td>63.09</td>
<td>17.32</td>
<td>1.404</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td></td>
<td>736.6</td>
<td>65.364</td>
<td>18.792</td>
<td>1.4306</td>
</tr>
</tbody>
</table>

3.2.4 SAFSTRIP® plates Material Properties

Safstrip plates [21] were used as the anchorage system in the designated beams. Safstrip is a pultruded composite strip that has high bearing and longitudinal properties. The strip used in the study had a width of 100 mm and a thickness of 3.2 mm. The composite strip is composed of carbon tows sandwiched between layers of fiberglass mats and rovings. This composite plate provides high stiffness and bearing strength. A synthetic surface veil is also incorporated into the composite to improve resistance to corrosion and UV degradation. [21]

Table 4 shows the mechanical material properties of the Safstrip® composite as provided by the manufacturer.

Table 4: Hybrid material properties [21]

<table>
<thead>
<tr>
<th>Mechanical Property</th>
<th>Average Value (MPa)</th>
<th>Design Value (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile Strength</td>
<td>852</td>
<td>540</td>
</tr>
<tr>
<td>Tensile Modulus</td>
<td>62190</td>
<td>62190</td>
</tr>
<tr>
<td>Clamped Bearing Strength</td>
<td>351</td>
<td>279</td>
</tr>
<tr>
<td>Unclamped Bearing Strength</td>
<td>214</td>
<td>180</td>
</tr>
<tr>
<td>Open Hole Strength</td>
<td>652</td>
<td>543</td>
</tr>
</tbody>
</table>
3.2.5 Epoxy

The epoxy adhesive used to attach the CFRP sheet to the damaged bottom flange of the composite beam was Sikadur®-330 [22]. Sikadur®-330 adhesive consists of two parts, a thixotropic epoxy based impregnating adhesive, as described by the manufacturer. This adhesive was tested according to EN 1504-4 and conforms to the requirements of esteemed institutes. The tensile strength, bond strength and tensile modulus are 30 MPa, 4MPa and 4500 MPa, respectively. [22]

3.3 Specimen Preparation

A total of 10 UC203x203x46 beams were prepared for concrete casting. All beams had shear studs of 10mm diameter and 75mm in height. The studs were welded to the top flange of the steel beam. Two shear studs were placed at the top flange of the steel member and spaced 120mm from each other. Figures 7 and 8 show the steel beam geometry including the shear stud arrangement, spacing and dimensions.

![Shear Stud Arrangement](image1)

**Figure 7. Shear Stud Arrangement**

![Shear Studs cross-section](image2)

**Figure 8. Shear Studs cross-section**
Figure 9 illustrates the fabrication process, where shear studs were first temporarily welded to the beams and placed perpendicular to the surface of the upper steel flange. The studs were then fully welded thus losing 5mm of their nominal height.

![Fabricated steel beam with shear studs](image)

**Figure 9. Fabricated steel beam with shear studs**

![Beam holes location](image)

**Figure 10. Beam holes location**

Figure 10 shows the location of the 6mm diameter holes that were made to the tensile bottom flange of the steel beams. The holes were made to mechanically anchor the Safstrip® [21] plate to the bottom flange of the beam. Two holes spaced laterally
at 75mm were imposed on the tensile flange and were spaced along the longitudinal direction with intervals of 135mm as shown in Figure 10.

Nine beams were cut with a 5mm wide U-notch along the width of the flange at the beam’s midspan as shown in Figure 11. The nine beams were divided onto three groups, with different notch depth as follows, to simulate different corrosion levels.

- Group A - 3 beams with U-notch depth of 5mm
- Group B - 3 beams with U-notch depth of 8mm
- Group C - 3 beams with U-notch depth of 11mm

Figure 11. Groove Preparation

Figure 11 shows the process of grooving. A steel cutting machine was used where the tip of the machine that rotates had different colors that allowed the fabricator to know, with high levels of confidence, the depth of the current groove during the operation of the machine. Furthermore, to add and maintain accuracy after each lap of motion, the depth was manually noted.
Figure 12 shows the end product of grooving and Figure 13 illustrates the exact location, dimension and degree of the groove imposed on the steel beams of the three groups.

Figure 12. Groove

Figure 13. Typical notch detail
The webs of the steel beam were laterally supported with 10mm thick stiffeners and placed under loading points and supports as shown in Figure 14. Two stiffeners were placed at each support and loading point with a 50mm space between each stiffener.

![Figure 14. Stiffeners Location](image1)

The final step for the preparation of the beam was casting of the concrete slab on the top flange surface of the steel beams as shown in Figure 15. The width and thickness of the casted concrete slabs were 450mm and 100mm respectively. Figure 15 shows a typical cross section of the prepared steel-concrete composite beams.

![Figure 15. Concrete Topping Dimension](image2)

### 3.4 Surface Preparation for Strengthening

Surface preparation is a crucial and necessary procedure to ensure an appropriate adhesion of epoxy to steel. A conventional method of surface preparation was performed which consisted of automatic steel brushing where the steel brush was connected to a drill. The second step was to manually brush the bottom flange surface to ensure that the surface of the beam was brushed uniformly. The final step was to apply acetone on the surface of the beam as shown in Figures 16 and 17.
3.5 Experimental Setup and Instrumentation

Figure 18 shows the four points bending setup of the tested specimen. The supports were placed at a distance of 150mm from the beam’s edge as shown in Figure 18. Two point loads were applied at 150mm away from the centerline of the beam. The CFRP sheets had a maximum length of 1500mm centered laterally and horizontally as shown in Figure 19.
3.5.1 Instrumentation (Strain Gauges Locations)

Figure 20 shows the strain gauge location at the composite beam’s mid-span for the 10 composite beams. Figure 21 shows a typical beam specimen along with the strain gauge instrumentation at mid-span prior to testing.
3.6 Test Matrix

A series of ten steel beams were fabricated and prepared in order to examine the effectiveness of using a hybrid externally bonded-mechanically anchored CFRP composite system to retrofit pre-damaged composite steel girders. One beam acted as a control beam without artificial damage. The other nine beams were divided equally into three main groups, namely Group A, Group B and Group C. Each group consisted of three beams that were fabricated with a compression concrete slab and each group exhibited different damaging levels. The first beam of each damaged group was unstrengthened. The second beam of each damaged group was strengthened with CFRP sheets externally bonded to the surface of the tensile (bottom) flange with epoxy adhesive. The third beam of each damaged group was strengthened with CFRP sheets externally bonded to the surface of the tensile (bottom) flange with epoxy adhesive and mechanically fastened by a pultruded composite strip composed of carbon tows sandwiched between layers of fiberglass mats and rovings. Table 5 shows the beam designation for different damaging levels and the degree of notch induced at the mid-span of the tensile flange.

Table 5: Beam group designation
<table>
<thead>
<tr>
<th>Group</th>
<th>Damage Level Designation</th>
<th>U-Notch Depth (mm)</th>
<th>Reduction in Bottom Flange Thickness (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>D1</td>
<td>5</td>
<td>45.5</td>
</tr>
<tr>
<td>B</td>
<td>D2</td>
<td>8</td>
<td>72.7</td>
</tr>
<tr>
<td>C</td>
<td>D3</td>
<td>11</td>
<td>100</td>
</tr>
</tbody>
</table>

Table 6 shows the designation of the beams of all groups and a brief description of each beam specimen.

Table 6: Beam tabular designation and description

<table>
<thead>
<tr>
<th>Beam Designation</th>
<th>Beam Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>CB</td>
<td>Undamaged and unstrengthened beam (control beam)</td>
</tr>
<tr>
<td><strong>Group A</strong></td>
<td></td>
</tr>
<tr>
<td>BF-T5</td>
<td>Unstrengthened beam with a predamage level of D1</td>
</tr>
<tr>
<td>BF-T5R4</td>
<td>Strengthened beam with a predamage level of D1, with 4 CFRP sheets</td>
</tr>
<tr>
<td>BF-T5R4A</td>
<td>Strengthened beam with a predamage level of D1, with 4 CFRP sheets and mechanically fastened with the composite plate.</td>
</tr>
<tr>
<td><strong>Group B</strong></td>
<td></td>
</tr>
<tr>
<td>BF-T8</td>
<td>Unstrengthened beam with a predamage level of D2</td>
</tr>
<tr>
<td>BF-T8R6</td>
<td>Strengthened beam with a predamage level of D2, with 6 CFRP sheets</td>
</tr>
<tr>
<td>BF-T8R6A</td>
<td>Strengthened beam with a predamage level of D2, with 6 CFRP sheets and mechanically fastened with the composite plate.</td>
</tr>
<tr>
<td><strong>Group C</strong></td>
<td></td>
</tr>
<tr>
<td>BF-T11</td>
<td>Unstrengthened beam with a predamage level of D3</td>
</tr>
<tr>
<td>BF-T11R8</td>
<td>Strengthened beam with a predamage level of D3, with 8 CFRP sheets</td>
</tr>
<tr>
<td>BF-T11R8A</td>
<td>Strengthened beam with a predamage level of D3, with 8 CFRP sheets and mechanically fastened with the composite plate.</td>
</tr>
</tbody>
</table>

3.6.1 Computation of the number of CFRP layers

An approximation index based on a study on the variation of CFRP length on pre-damaged steel [4] was used to come up with the total number of layers needed to restore the damaged beams. Table 7 presents the results based on equation 3.6.1.1 and shows that four, five and eight layers were required to retrofit Group A, B and C’s
specimens, respectively, in order to recover the strength of the control undamaged beam specimen (CB).

\[
\eta = \frac{\sum_{i=1}^{n}[A_i F_{u,i}]}{A_f F_y}
\]

Equation 1

Table 7: CFRP layers

<table>
<thead>
<tr>
<th>Group</th>
<th>Damage Level</th>
<th>(\sum_{i=1}^{n}[A_i F_{u,i}])</th>
<th>(A_f F_y)</th>
<th># of layers required to retrofit</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>D1</td>
<td>315</td>
<td>94</td>
<td>4</td>
</tr>
<tr>
<td>B</td>
<td>D2</td>
<td>505</td>
<td>94</td>
<td>6</td>
</tr>
<tr>
<td>C</td>
<td>D3</td>
<td>693</td>
<td>94</td>
<td>8</td>
</tr>
</tbody>
</table>

\(\eta\) : Number of layers of CFRP

\(A_i\) : Cross-sectional areas of FRP layer

\(F_{u,i}\) : Ultimate tensile strength of FRP layer

\(A_f\) : Cross-sectional areas of steel flange

\(F_y\) : yield strength of steel

3.6.2 Staggering of CFRP Sheets

Table 8 illustrates the CFRP sheet lengths that were staggered for each group of strengthened specimen. The staggering was necessary to have a cost effective solution based on engineering judgment since the maximum bending moment is occurring at the middle of the composite beam. Table 8 shows the length of each layer for the three groups of beams.

Table 8: CFRP staggering

<table>
<thead>
<tr>
<th>Layer</th>
<th>Group A 4 layers (mm)</th>
<th>Group B 6 layers (mm)</th>
<th>Group C 8 layers (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1550</td>
<td>1550</td>
<td>1600</td>
</tr>
<tr>
<td>2</td>
<td>1500</td>
<td>1500</td>
<td>1550</td>
</tr>
</tbody>
</table>
3.7 Test Matrix Graphical Representation

Tables 9 through 12 show a graphical representation of the tested specimens in this study along with their designation.

Table 9: Control beam

<table>
<thead>
<tr>
<th>Specimen Designation</th>
<th>Specimen Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>CB</td>
<td>Undamaged and unstrengthened beam</td>
</tr>
</tbody>
</table>

Table 10: Group A (Pre-damage level D1 with 5.0 mm depth U-Notch)

<table>
<thead>
<tr>
<th>Specimen Designation</th>
<th>Specimen Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>BF-T5</td>
<td>Unstrengthened beam</td>
</tr>
<tr>
<td>BF-T5R4</td>
<td>Strengthened beam with 4 CFRP sheets</td>
</tr>
</tbody>
</table>
Bf-T5R4A

Strengthened beam with 4 CFRP sheets and mechanically fastened with the composite plate

Table 11: Group B (Pre-damage level D2 with 8.0 mm depth U-Notch)

<table>
<thead>
<tr>
<th>Specimen Designation</th>
<th>Specimen Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>BF-T8</td>
<td>Unstrengthened beam</td>
</tr>
<tr>
<td>BF-T8R6</td>
<td>Strengthened beam with 6 CFRP sheets</td>
</tr>
</tbody>
</table>
Table 12: Group C (Pre-damage level D3 with 11.0 mm depth U-Notch)

<table>
<thead>
<tr>
<th>Specimen Designation</th>
<th>Specimen Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>BF-T11R8A</td>
<td>Strengthened beam with 6 CFRP sheets and mechanically fastened with the composite plate</td>
</tr>
<tr>
<td></td>
<td>BF-T11</td>
</tr>
<tr>
<td></td>
<td>BF-T11R8</td>
</tr>
<tr>
<td></td>
<td>BF-T11R8A</td>
</tr>
</tbody>
</table>
3.8 Accuracy of Specimens

The accuracy of the values presented in this chapter and the following chapters is provided in this section. The accuracy of all testing machines used is +/- 1%. The accuracy of specimen preparation is +/- 2% due to human error.
Chapter 4: Experimental Results

This chapter presents the experimental results of the ten tested composite-beams. The sections will be divided into the three groups discussed in Chapter 3. The failure mode, load versus deflection, concrete-strain response, steel-strain response and CFRP strain response will be reported for each group.

4.1 Group A: Specimens with Damage State D1
Group A consists of composite beams BF-T5, BF-T5R4, and BF-T5R4A. All beams were tested to failure, and the results are reported in the following subsections. The results of the control specimen (CB) are included in all the subsections for the purpose of comparison.

4.1.1 Failure Mode
This section discusses the failure mode of specimens CB, BF-T5, BF-T5R4, and BF-T5R4A observed during the testing stage of the beams. Control specimen CB exhibited a classical flexural mode of failure where failure began by yielding of the steel section at the extreme tension fiber followed by crushing of the concrete at the extreme compression fiber. A longitudinal splitting crack was developed at the onset of failure in the top face of the concrete flange along the beam span. Figure 22 shows a photo of specimen CB after failure.

![Figure 22. Specimen CB at failure](image)

Specimen BF-T5, which was damaged but not retrofitted, failed by sudden fracture of the lower steel flange at the mid-span section. The stress concentration caused by the notch in the lower steel flange resulted in the development and
propagation of a crack stemming from the notch into the web. Yielding of the lower steel flange occurred prior to failure but no crushing of concrete was observed at peak load. Figure 23 shows a photo of specimen BF-T5 at failure.

![Image of specimen BF-T5 at failure](image)

**Figure 23. Failure of specimen BF-T5**

Specimen BF-T5R4, which was damaged then retrofitted with 4 layers of externally-bonded CFRP laminates without mechanical anchors, failed by yielding of the lower steel flange followed by sudden delamination of the CFRP. No crushing of concrete was observed at the onset of failure. Figure 24 shows a photo of specimen BF-T5R4 at failure.

Specimen BF-T5R4A, which was damaged then retrofitted with 4 layers of externally-bonded CFRP laminates in combination with mechanical anchors, failed by crushing of concrete in compression preceded by yielding of the lower steel flange. The presence of the mechanical anchors prevented delamination of the CFRP and allowed the beam to develop full flexural capacity. Figure 25 shows a photo of specimen BF-T5R4 at failure.
4.1.2 Tabulated Results and Load-Deflection Graph

Test results for the specimens of Group A are summarized in Table 13 whereas the load-deflection curves are depicted in Figure 26. Results of the control specimen are included in Table 13 and Figure 26 for the purpose of comparison. The 5mm deep U-notch reduced the load capacity by approximately 11%. It also reduced the beam ductility significantly by approximately 77%. The CFRP retrofitting system fully restored the flexural capacity of the damaged beams. The ultimate loads of retrofitted specimens BF-T5R4 and BF-T5R4A were even higher than that of control specimen CB by approximately 3 and 6%, respectively. The ductility indices of
retrofitted specimens BF-T5R4 and BF-T5R4A were higher than that of specimen BF-T5 but still lower than that of control specimen CB by 45 and 43%, respectively. The stiffness of specimens CB, BF-T5, and BF-T5R4 was insignificantly different. The stiffness of specimen BF-T5R4A was, however, 24% higher than that of the control specimen.

Table 13: Results of Group A

<table>
<thead>
<tr>
<th></th>
<th>Load Py</th>
<th>ΔY</th>
<th>k = Py/Δy</th>
<th>k/k_CB</th>
<th>Load Pu</th>
<th>ΔU</th>
<th>μ1 = Δu/Δy</th>
<th>Strength reduction (%)</th>
<th>Strength gain with respect to that of CB (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CB</td>
<td>485</td>
<td>8.1</td>
<td>59.88</td>
<td>1.00</td>
<td>567</td>
<td>54.5</td>
<td>6.73</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>BF-T5</td>
<td>375</td>
<td>5.8</td>
<td>64.66</td>
<td>1.08</td>
<td>505</td>
<td>9</td>
<td>1.55</td>
<td>11</td>
<td>-</td>
</tr>
<tr>
<td>BF-T5R4</td>
<td>462</td>
<td>7</td>
<td>66.00</td>
<td>1.10</td>
<td>583</td>
<td>25.8</td>
<td>3.69</td>
<td>-</td>
<td>103</td>
</tr>
<tr>
<td>BF-T5R4A</td>
<td>460</td>
<td>6.2</td>
<td>74.19</td>
<td>1.24</td>
<td>603</td>
<td>23.7</td>
<td>3.82</td>
<td>-</td>
<td>106</td>
</tr>
</tbody>
</table>

- ΔY → Deflection at yielding load (mm)
- K → Stiffness of section (KN/mm)
- ΔU → Deflection at ultimate load (mm)
- μ1 → Ductility index
- P_u → Ultimate load (KN)
- P_y → Yield load (KN)

4.1.3 Concrete Strain Response

Figure 27 shows the concrete strain response of specimens of Group A along with those of control specimen CB. The concrete strain was measured at the extreme compression fiber of the concrete flange. The concrete strain in control specimen CB reached the crushing value (0.003 mm/mm) at the onset of failure. Specimen BF-T5 reached its peak load at a concrete strain of approximately 2000 microstrains whereas specimen BF-T5R4 recorded a maximum concrete strain of approximately 585 microstrains only. Concrete crushing took place in specimen BF-T5R4A as shown in Figure 25 but the concrete strain gauge was damaged prior to failure.
Figure 26. Load vs midspan deflection response for Group A

Figure 27. Concrete strain response for Group A

4.1.4 Steel Strain Response

Figure 28 shows the load versus the steel strain measured in the mid-span section at the extreme tension fiber of the steel web. The figure shows that the steel yielded in all specimens because the strain at peak load exceeded the steel yield strain (0.0016 mm/mm). Initially, specimen BF-T5 exhibited a steel strain response similar to that of control specimen CB until it reached a load value of approximately 150 kN. After that, the steel strain of specimen BF-T5 increased at a rate higher than that of
the control specimen. The rapid increase in the steel strain of specimen BF-T5 reduced the yield load and hence the flexural capacity relative to those of the control specimen. From this figure, it can also be seen that the steel strain of the retrofitted specimens in the pre-yield stage increased at a rate similar to that of control specimen CB. Following yielding, the control specimen exhibited a plastic steel strain response whereas the steel strain in the retrofitted specimens continued to increase but at a rate higher than that recorded in the pre-yield stage.

![Steel strain response for Group A](image)

**Figure 28. Steel strain response for Group A**

### 4.1.5 CFRP Strain Response

Figure 29 shows the load versus the CFRP strain measured in the mid-span section at the extreme tension fiber of the CFRP laminates. The rupture strain of the CFRP as calculated in Chapter 3 was (0.0143mm/mm). Figure 29 shows that both BF-T5R4 and BF-T5R4A did not utilize the full capacity of the CFRP layers because specimen BF-T5R4 failed prematurely by CFRP delamination and specimen BF-T5R4A failed by concrete crushing without rupture of CFRP. Specimen BF-T5R4A showed better stiffness than BF-T5R4 since it had a lower slope value. A maximum CFRP strain value of approximately 1870 microstrains was recorded at failure of specimen BF-T5R4 whereas, for specimen BF-T5R4A, the CFRP strain recorded at failure was approximately 3300 microstrains.
4.2 Group B: Specimens with Damage State D2

Group B consists of composite beams of BF-T8, BF-T8R6, and BF-T8R6A. All beams were tested to failure and the following subsections will report the results. Results of control specimen CB are included in all subsections for the purpose of comparison.

4.2.1 Failure mode

Specimen BF-T8, which was pre-damaged but not retrofitted, failed by sudden fracture of the lower steel flange at the mid-span section in addition to development and propagation of a crack stemming from the notch into the web. Yielding of the bottom part of the web took place prior to failure but no concrete crushing was observed. Figure 30 shows a photo of specimen BF-T8 at failure.

Similarly, specimen BF-T8R6 failed by sudden fracture of the lower steel flange at the mid-span section in combination with development and propagation of a crack stemming from the lower steel flange into the web. Yielding of the bottom part of the web occurred almost at the same time of failure by development of the mid-
span crack. No concrete crushing was observed at failure. Figure 31 shows a photo of specimen BF-T8R6 at failure.

For specimen BF-T8R6A, the lower steel flange exhibited sudden fracture at the onset of failure in the mid-span section. Yielding of the bottom part of the web occurred shortly prior to failure. Neither delamination of CFRP nor concrete crushing was observed at failure. Figure 32 shows a photo of specimen BF-T8R6A at failure.

Figure 30. BF-T8 at failure

Figure 31. BF-T8R6 at failure
4.2.2 Tabulated results and load-deflection graph

Test results for specimens of Group B are summarized in Table 14 whereas the load-deflection curves are depicted in Figure 33. Results of the control specimen are included in Table 14 and Figure 33 for the purpose of comparison. The 8mm deep U-notch reduced the load capacity by approximately 23%. It also reduced the beam stiffness and ductility by approximately 15 and 76%, respectively. The CFRP retrofitting system did not increase the load capacity and ductility of the pre-damaged beams but slightly improved the stiffness. This occurred because of the rapid propagation of the crack stemming from the lower steel flange into the web, which resulted in a premature failure and concealed the effect of retrofitting with CFRP.

Table 14: Results of Group B

<table>
<thead>
<tr>
<th></th>
<th>Load $P_y$</th>
<th>$\Delta Y$</th>
<th>$k = P_y/\Delta Y$</th>
<th>$k/k_{un}$</th>
<th>Load $P_u$</th>
<th>$\Delta U$</th>
<th>$\mu = \Delta u/\Delta y$</th>
<th>Strength reduction (%)</th>
<th>Strength gain with respect to that of CB (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CB</td>
<td>485</td>
<td>8.1</td>
<td>59.88</td>
<td>1.00</td>
<td>567</td>
<td>54.5</td>
<td>6.73</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>BF-T8</td>
<td>290</td>
<td>5.7</td>
<td>50.88</td>
<td>0.85</td>
<td>438</td>
<td>9.2</td>
<td>1.61</td>
<td>23</td>
<td>-</td>
</tr>
<tr>
<td>BF-T8R6</td>
<td>440</td>
<td>6.6</td>
<td>66.7</td>
<td>1.11</td>
<td>440</td>
<td>7.2</td>
<td>1.1</td>
<td>-</td>
<td>78</td>
</tr>
<tr>
<td>BF-T8R6A</td>
<td>424</td>
<td>7</td>
<td>60.6</td>
<td>1.02</td>
<td>432</td>
<td>9.5</td>
<td>1.35</td>
<td>-</td>
<td>76</td>
</tr>
</tbody>
</table>

- $\Delta Y$ → Deflection at yielding load (mm)
- K → Stiffness of section (KN/mm)
- ΔU → Deflection at ultimate load (mm)
- μ1 → Ductility index
- P_u → Ultimate load (KN)
- P_y → Yield load (KN)

4.2.3 Concrete Strain Response

Figure 34 shows the concrete strain response of specimens of Group B along with that of control specimen CB. The concrete strain was measured at the extreme compression fiber of the concrete flange. The concrete strain in control specimen CB reached the crushing value (0.003 mm/mm) at the onset of failure. Specimen BF-T8 reached its peak load at a concrete strain of approximately 2000 microstrains whereas specimens BF-T8R6 and BF-T8R6A recorded maximum concrete strain values of 1330 and 1100 microstrains, respectively. This confirms that no concrete crushing occurred in any specimen of this group.

Figure 33. Load vs midspan deflection response for Group B
4.2.4 Steel Strain Response

Figure 35 shows the load versus the steel strain measured in the mid-span section at the extreme tension fiber of the steel web. Specimen BF-T8 exhibited higher steel strains than those exhibited by other specimens of this group. The steel strain for specimens BF-T8R6 and BF-T8R6A increased at a rate similar to that of control specimen CB. For specimen BFT8, the steel yielded prior to failure since the strain at failure exceeded the steel yield strain (0.0016 mm/mm). Specimen BF-T8R6 failed by fracture of steel that occurred almost simultaneously at the onset of yielding whereas specimen BF-T8R6A failed shortly after the yielding of steel.
4.2.5 CFRP Strain Response

Figure 36 shows the load versus the CFRP strain measured in the mid-span section at the extreme tension fiber of the CFRP laminates. The rupture strain of the CFRP as calculated in Chapter 3 was (0.0143mm/mm). Figure 36 shows that both BF-T8R6 and BF-T8R6A did not utilize the full capacity of the CFRP layers because the specimens failed prematurely by fracture of steel in the lower steel flange at the mid-span section. Specimen BF-T8R6A showed better stiffness than BF-T8R6 since it has a lower slope value. A maximum CFRP strain value of approximately 1000 microstrains was recorded at failure of specimen BF-T8R6 whereas for specimen BF-T8R6A the CFRP strain recorded at failure was approximately 2100 microstrains. This indicates that the use of mechanical anchors results in better utilization of CFRP.

Figure 36. CFRP strain response for Group B

4.3 Group C: Specimens with damage state D3

Group C consists of composite beams of BF-T11, BF-T11R8, BF-T11R8A. All beams were tested to failure and the following subsections will report the results. Results of control specimen CB are included in all subsections for the purpose of comparison.

4.3.1 Failure mode
Specimen BF-T11 failed by yielding of the bottom part of the steel web followed by development of a crack at the mid-span section. No concrete crushing was observed. Figure 37 shows a photo of specimen BF-T11 at failure.

Specimen BF-T11R8 failed by development of a crack in the bottom part of the steel web at the mid-span section accompanied by CFRP end delamination. Failure was preceded by yielding of the bottom part of the web but no concrete crushing was observed. Figure 38 shows a photo of specimen BF-T11R8 at failure.

Specimen BF-T11R8A failed by yielding of the bottom part of the steel web followed by development of a crack in the extreme tension fiber of the web at the mid-span section. The presence of mechanical anchors prevented delamination of the CFRP sheets but no concrete crushing was observed.

Figure 37. BF-T11 at failure

Figure 38. BF-T11R8 at failure
4.3.2 Tabulated results and load-deflection graph

Test results for specimens of Group C are summarized in Table 15 whereas the load-deflection curves are depicted in Figure 39. Results of the control specimen are included in Table 15 and Figure 39 for the purpose of comparison. Cutting the full depth of the lower steel flange (11mm deep U-notch) significantly reduced the load capacity by approximately 50%. It also reduced the beam stiffness and ductility by approximately 22 and 75%, respectively. The CFRP retrofitting system effectively increased the load capacity of the pre-damaged beams but could not restore the full original flexural strength. The CFRP retrofitting with and without anchors restored 81 and 73% of the original flexural strength, respectively. The stiffness of specimens BF-T11, BF-T11R8, and BF-T11R8A was insignificantly different. The ductility index of specimen BF-T11R8 was lower than that of specimen BF-T11 whereas specimen BF-T11R8A had a similar ductility index as that of specimen BF-T11.

Table 15: Results of Group C

<table>
<thead>
<tr>
<th></th>
<th>Load P_y</th>
<th>ΔY</th>
<th>k = P_y/Δy</th>
<th>k/k_Cb</th>
<th>Load P_u</th>
<th>ΔU</th>
<th>μ₁ = Δu/Δy</th>
<th>% reduction in strength</th>
<th>% Restoration with respect to CB</th>
</tr>
</thead>
<tbody>
<tr>
<td>CB</td>
<td>485</td>
<td>8.1</td>
<td>59.88</td>
<td>1.00</td>
<td>567</td>
<td>54.5</td>
<td>6.73</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>BF-T11</td>
<td>237</td>
<td>5.1</td>
<td>46.47</td>
<td>0.776</td>
<td>284</td>
<td>8.4</td>
<td>1.65</td>
<td>50</td>
<td>-</td>
</tr>
<tr>
<td>BF-T11R8</td>
<td>330</td>
<td>5.9</td>
<td>55.93</td>
<td>0.934</td>
<td>414</td>
<td>7.9</td>
<td>1.34</td>
<td>-</td>
<td>73</td>
</tr>
<tr>
<td>BF-T11R8A</td>
<td>342</td>
<td>6.1</td>
<td>56.07</td>
<td>0.936</td>
<td>462</td>
<td>10.2</td>
<td>1.67</td>
<td>-</td>
<td>81</td>
</tr>
</tbody>
</table>

- ΔY → Deflection at yielding load (mm)
- K → Stiffness of section (KN/mm)
- ΔU → Deflection at ultimate load (mm)
- μ₁ → Ductility index
- P_u → Ultimate load (KN)
- P_y → Yield load (KN)

4.3.3 Concrete strain response

Figure 40 shows the concrete strain response of specimens of Group C along with that of control specimen CB. The concrete strain was measured at the extreme compression fiber of the concrete flange. The concrete strain in control specimen CB reached the crushing value (0.003 mm/mm) at the onset of failure. Specimen BF-T11 failed at a concrete strain of approximately 900 microstrains only, whereas specimens
BF-T11R8 and BF-T11R8A recorded maximum concrete strain values of 2000 and 1100 microstrains, respectively. This confirms that no concrete crushing occurred in any specimen of this group.

Figure 39. Load vs midspan deflection response for Group C

Figure 40. Concrete strain response for Group C
4.3.4 Steel strain response

Figure 41 shows the load versus steel strain measured in the mid-span section at the extreme tension fiber of the steel web. Specimen BF-T11 exhibited higher steel strains than those exhibited by other specimens. The steel strain of specimens BF-T11R8 and BF-T11R8A increased at a rate similar to that of control specimen CB. For specimen BFT11, the steel has yielded prior to failure since the strain at the peak load exceeded the steel yield strain (0.0016 mm/mm). Specimens BF-T11R8 and BF-T11R8A featured yielding of the bottom part of the web just prior to failure. The steel strain gauge of specimen BF-T11R8 was damaged prior to failure.

4.3.5 CFRP Strain Response

Figure 42 shows the load versus the CFRP strain measured in the mid-span section at the extreme tension fiber of the CFRP laminates. The rupture strain of the CFRP as calculated in Chapter 3 was (0.0143 mm/mm). Figure 42 shows that both specimens BF-T11R8 and BF-T11R8A did not utilize the full capacity of the CFRP layers because the specimens failed prematurely by developing a crack in the bottom part of the web at the mid-span section. Specimen BF-T11R8A showed better stiffness than BF-T11R8. A maximum CFRP strain value of approximately 780 microstrains was recorded at failure of specimen BF-T11R8. For specimen BF-T11R8A, the CFRP strain recorded at failure was approximately 2500 microstrains. This demonstrates that the use of mechanical anchors results in better utilization of CFRP.

![Figure 41. Steel strain response for Group C](image-url)
Figure 42. CFRP strain response for Group
Chapter 5: Finite Element Modeling

5.1. Background

The aim of this chapter is to develop finite element (FE) models that can predict the behavior and load-carrying capacity of the following three selected beam specimens: “CB”, “BF-T11”, and BF-T11R8A”. The geometrical and material properties of these specimens are presented in Chapter 3, and the measured experimental data is given in Chapter 4 of this thesis. The development of 3D FE quarter models using the ANSYS 14.0 [23] software in terms of the used element types, constitutive material properties, geometry, loading and boundary conditions will be presented in this chapter.

Quarter models are developed due to symmetry in the geometry and loading of the investigated specimens. The models have the same geometric and mechanical properties of the tested specimens and incorporate nonlinear material properties for the steel and concrete materials. The CFRP composite sheets are modeled using orthotropic material properties. The measured and predicted load-midspan deflection response results along with the beams’ load-carrying capacity are compared to validate the accuracy of the developed models. The results have shown close correlation between the experimentally obtained and predicted FE results. The developed FE models could be thus expanded in a future research investigation to validate the response of the other tested specimens presented in this study. The developed models could be also used in design-oriented parametric studies to analytically investigate the behavior of strengthened pre-damaged composite steel beams with externally bonded FRP sheets and laminates.

5.2. Properties of the Developed FE Models

Figures 43-45 show the meshed 3D geometry of the quarter “CB”, “BF-T11”, and “BF-T11R8A” models, respectively. The total number of elements and nodes for each developed model is given in Table 16. The steel beam I-section, loading supports, and epoxy adhesive materials were simulated using three-dimensional (3D) SOLID45 [23] brick elements. The concrete slab was simulated using 3D concrete SOLID65 [30] brick elements. Both the SOLID45 and SOLID65 elements are defined by eight nodes. Each node of the brick element has 3 degrees of freedom in the translational x, y, and z directions, respectively. The SOLID45 and SOLID65 elements are capable of simulating the material’s nonlinearities, plasticity, and large
deflections. The concrete SOLID65 element has an additional capability of cracking and crushing in tension and compression, respectively. The CFRP sheets are simulated as an orthotropic material using SHELL63 [23] elements that have bending and large deflection capabilities. The SHELL63 element is defined by four nodes with six degrees of freedom (translational and rotational in the x, y, and z directions), thickness, and orthotropic material properties. A perfect bond between the steel and concrete, steel and epoxy, and epoxy and CFRP surfaces is assumed in the developed FE models. This is simulated by enforcing strain compatibility of coincident nodes between the different materials in all directions. Thus, the elements of different materials share the same nodes along their adjacent surface.
Figure 43. Developed quarter FE model of the tested “CB” beam specimen
Figure 44. Developed quarter FE model of the tested “BF-T11” beam specimen

(c) Vicinity of the Notch Region

Figure 45. Developed quarter FE model of the tested “BF-T11R8A” beam specimen
Table 16: Total number of elements and nodes in the developed FE models

<table>
<thead>
<tr>
<th>Beam Model</th>
<th>Number of Elements</th>
<th>Number of Nodes</th>
</tr>
</thead>
<tbody>
<tr>
<td>CB</td>
<td>40,956</td>
<td>50,526</td>
</tr>
<tr>
<td>BF-T11</td>
<td>40,920</td>
<td>50,477</td>
</tr>
<tr>
<td>BF-T11R8A</td>
<td>43,872</td>
<td>52,658</td>
</tr>
</tbody>
</table>

Linear and nonlinear material properties were assigned for the steel beam, concrete, epoxy, and rigid supports. The material properties for the CFRP unidirectional sheets were defined as linear and orthotropic. The steel beam was modeled as an elastic-fully plastic material property. The elastic modulus, yield strength, and Poisson’s ratio for the steel beam were taken as 200 GPa, 318.75 MPa, and 0.3, respectively. The concrete was simulated as a nonlinear material based on the constitutive model of William and Warnkee [23, 24] with an elastic modulus, compressive strength, tensile strength, and Poisson’s ratio of 21.7 GPa, 20.1 MPa, 2.78 MPa, and 0.2, respectively. The stress-strain curve of the concrete material in compression was simulated as shown in Figure 46 using the Hognestad parabola [25]. The epoxy adhesive was simulated as a brittle material with an elastic modulus, tensile strength, and Poisson’s ratio of 4500 MPa, 30 MPa, and 0.21, respectively. The thickness of each layer of the composite CFRP sheets (CFRP fibers and resin) was taken as 0.666 mm and was simulated with the orthotropic material properties listed in Table 17.

![Figure 46. Compressive stress-strain curve of the concrete material](image-url)
The developed models were loaded by applying a downward incremental displacement to the loading support till failure. This will capture the ductility response of the tested specimens. Nonlinear-static analysis was performed and failure was defined when the reaction of the modeled beam specimen was dropped by 15%. The modeled beams were simply supported and had two planes of symmetry. Symmetrical boundary conditions were applied by restraining the movement perpendicular to the planes of symmetry by rollers as shown in Figure 43. The ANSYS [23] Newton-Raphson technique was employed to run the nonlinear analysis by dividing the applied loads into a series of load steps and sub-steps using the automated time stepping option.

5.3. Results and Discussion

5.3.1. Validation of FE Models

The data obtained from the experimental results for the load-midspan deflection response results of beam specimens “CB”, “BF-T11”, and “BF-T11R8A” are compared with predicted FE results as shown in Figures 47-49. Table 13 also provides a comparison between the experimental and numerical results for the load-carrying capacity and deflection at failure. It is clear from Figures 47-49 and Table 18 that the predicted FE results are very close to the measured experimental data with a maximum deviation less than 10%. The percent difference for the predicted over the measured ultimate load and deflection at failure ranged from -0.48 to 6.47% and from -4.43 to 5.98%, respectively. Thus, it can be concluded that the developed numerical

<table>
<thead>
<tr>
<th>Mechanical Property</th>
<th>Value &amp; Unit</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_x$</td>
<td>65.36 GPa</td>
<td>Elastic modulus along the fibers direction</td>
</tr>
<tr>
<td>$E_y$</td>
<td>4.57 GPa</td>
<td>Elastic modulus transverse to the direction of the fibers</td>
</tr>
<tr>
<td>$E_z$</td>
<td>4.57 GPa</td>
<td>Elastic modulus transverse to the direction of the fibers</td>
</tr>
<tr>
<td>$G_{xy}$</td>
<td>1786 MPa</td>
<td>Shear modulus in the $xy$ plane</td>
</tr>
<tr>
<td>$G_{xz}$</td>
<td>1786 MPa</td>
<td>Shear modulus in the $xz$ plane</td>
</tr>
<tr>
<td>$G_{yz}$</td>
<td>1610 MPa</td>
<td>Shear modulus in the $yz$ plane</td>
</tr>
<tr>
<td>$\nu_{xy}$</td>
<td>0.28</td>
<td>Poisson’s ratio in the $xy$ plane</td>
</tr>
<tr>
<td>$\nu_{xz}$</td>
<td>0.28</td>
<td>Poisson’s ratio in the $xz$ plane</td>
</tr>
<tr>
<td>$\nu_{yz}$</td>
<td>0.42</td>
<td>Poisson’s ratio in the $yz$ plane</td>
</tr>
</tbody>
</table>
models can serve as a valid and economical tool to capture the response and predict the load-carrying capacity of the tested composite beam specimens with a good level of accuracy.

Figure 47. Experimental and predicted load-deflection results for “CB” specimen

Figure 48. Experimental and predicted load-deflection results for “BF-T11” specimen
Figure 49. Experimental and predicted load-deflection results for “BF-T11R8A” specimen

Table 18: Comparison between the experimental and FE results

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Ultimate Load (kN)</th>
<th>% Difference (FE/Exp.)</th>
<th>Deflection at Failure (mm)</th>
<th>% Difference (FE/Exp.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Exp.</td>
<td>FE</td>
<td>Exp.</td>
<td>FE</td>
</tr>
<tr>
<td>CB</td>
<td>567.0</td>
<td>583.04</td>
<td>2.82</td>
<td>-7.35</td>
</tr>
<tr>
<td>BF-T11</td>
<td>284.1</td>
<td>282.77</td>
<td>-0.48</td>
<td>3.93</td>
</tr>
<tr>
<td>BF-T11R8A</td>
<td>465.97</td>
<td>496.14</td>
<td>6.47</td>
<td>-4.43</td>
</tr>
</tbody>
</table>

5.3.2. FE results

The FE results of the developed models can provide deformation and stress results in all directions throughout the model. Figure 50 shows a nodal contour for the vertical displacement (y direction) for the “CB”, “BF-T11”, and “BF-T11R8A” specimens, respectively, at failure.
(a) “CB” specimen

(b) “BF-T11” specimen
Figure 50. Vertical nodal displacement

Figure 51 shows the axial stress ($S_x$) distribution at failure for the “CB” beam specimen. It is clear from Figure 51 that the beam failed by yielding of the steel I-beam and concrete crushing in the mid-span region.
Figure 51: Axial stress distribution (Sx) for the “CB” specimen at failure

Figure 52 shows the axial stress distribution for the pre-damaged “BF-T11” beam specimen at failure. It is clearly indicated from Figure 52 that failure was initiated as expected due to the high stress concentration at the tip of the notch. It should be also noted that the stresses in the top concrete flange shown in Figure 52(b) of the damaged specimen are lower than those of the “CB” beam specimen shown in Figure 51(b).
Figure 52. Axial stress distribution (Sx) for the “BF-T11” beam specimen at failure

Figure 53 shows the axial stress distribution for the pre-damaged and repaired “BF-T11” beam specimen at failure. It is clearly observed from Figure 53 (b) that the stress concentration of the “BF-T11R8A” specimen in the vicinity of the notch is significantly lower than that of specimen “BF-T11” at failure. Figure 53 (c) indicates that failure of the “BF-T11R8A” specimen was initiated at mid-span in the epoxy adhesive when the epoxy
reached its tensile strength of 30 MPa. It should be also noticed from Figure 53 (d) that the maximum tensile stress achieved in the CFRP composite sheets at failure was 511.4 MPa, which is lower than its average tensile strength of 736.6 MPa. Thus, the full tensile strength of the CFRP sheets was not utilized. It can be concluded that failure of the repaired beam with CFRP composite sheets was controlled by the tensile strength of the epoxy adhesives. Thus, the load-carrying capacity of the repaired beam would be increased by using a higher strength epoxy adhesive.
Figure 53. Axial stress distribution (Sx) for the “BF-T11R8” specimen at failure
Chapter 6: Summary and Conclusions

In this study, pre-damaged steel-concrete composite beams retrofitted with CFRP sheets and mechanically anchored with pultruded composite plates with high stiffness and bearing strength were investigated. A total of 10 composite steel-concrete beams were prepared and tested in two-point loading till failure. One beam was left undamaged to serve as a control specimen, while the remaining beams were divided into three groups, each consisting of three specimens, and artificially damaged by cutting different notch depths of 5 mm, 8 mm and 11 mm in the bottom flange representing damage levels of 45%, 73% and 100%, respectively. In each group, the first beam was tested without strengthening, the second specimen was externally retrofitted in flexure with CFRP sheets bonded with epoxy adhesives, and the third beam was strengthened in a similar fashion to the second specimen and mechanically fastened using the proposed composite pultruded plate system. A finite element (FE) model was also developed to predict the performance of selected beam specimens that accurately simulated the response of the control beam and some selected strengthened specimens.

The following conclusions and observations were drawn from this study:

- The test results showed that the load-carrying capacity of the deteriorated specimens with different notch depths of 5 mm, 8 mm and 11 mm was reduced by 10.57, 21.52, and 49.1%, respectively.
- The strength of the repaired beam specimens with CFRP sheets ranged from 74.19 to 104.48% of the undamaged (control) beam, and ranged from 77.42 to 108.06% for the beams repaired with the proposed mechanically anchorage system.
- CFRP sheets bonded to the bottom of the tensile flange will fully restore the strength of damaged steel-concrete beams if the notch size at midspan is less than 50% of the bottom flange thickness.
- A mechanically fastened CFRP system with the proposed pultruded composite plates will fully restore the strength of damaged steel-concrete beams if the notch size at midspan is less than 50% of the bottom flange thickness.
• The strength and ductility of the beams repaired with the proposed mechanically fastened system outperformed those strengthened with bonded CFRP sheets only. This is due to the fact that the mechanically fastened system delayed debonding of the CFRP sheets from the bottom steel flange surface.
• The developed FE model accurately simulated the response of the control beam and some selected strengthened specimens.
• The FE model generated for the CB, BF-T11 and BF-T11R8A specimens had a load-capacity difference of 2.82, 0.48 and 6.5%, respectively from the obtained experimental data.
• The FE model generated for the CB, BF-T11 and BF-T11R8A specimens had a failure midspan deflection difference of 6, 4 and 4.5%, respectively, from the obtained experimental data.

Future studies should investigate the proposed retrofitting scheme with different notch sizes and shapes along the flange or beam’s web. In addition, more FE models should be developed to simulate the debonding of FRP sheets from steel surfaces and grid refinement. Such models could be expanded to numerically evaluate the effect of several variables on the performance of pre-damaged steel-concrete beams repaired with the proposed mechanically anchored system with flexible pultruded composite plates.
References


Vita

Ehab Clovis Karam was born on December 31, 1988, in Dubai, U.A.E. He studied and received his higher school certificate from the Internation School of Choueifat in Sharjah, U.A.E. He topped in O’leves and A’levels, and therefore he got the scholarship from American University Of Sharjah. He was awarded the Bachelor of Science in Civil Engineering from American University of Sharjah in summer of 2010. Ehab gained his design experience from econstruct by working on the design of Dubai Pearl project in Dubai. In September 2011, he joined the Masters of Science in Civil Engineering program with a concentration in structures in American University of Sharjah during which he worked as a lab assistant and grader for various courses. He also worked as a graduate research assistant for one semester and supervised undergraduate senior design projects. He also gained experience in post-installed connections by working with Hilti and currently working as a structural engineer on site for WME.