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PERFORMANCE OF MECHANICAL AND NON-MECHANICAL CONNECTIONS TO GFRP COMPONENTS

by

NNADOZIE N.F DIKE
B.S.C.E. Embry-Riddle Aeronautical University, 2007

A thesis submitted in partial fulfillment of the requirements for the degree of Master of Science in the Department of Civil and Environmental Engineering in the College of Engineering and Computer Science at the University of Central Florida Orlando, Florida

Summer Term
2012

Major Professor: Kevin R. Mackie
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ABSTRACT

There are presently many solutions to dealing with aging or deteriorated structures. Depending on the state of the structure, it may need to be completely over-hauled, demolished and replaced, or only specific components may need rehabilitation. In the case of bridges, rehabilitation and maintenance of the decks are critical needs for infrastructure management. Viable rehabilitation options include replacement of decks with aluminum extrusions, hybrid composite and sandwich systems, precast reinforced concrete systems, or the use of pultruded fiber-reinforced polymer (FRP) shapes. Previous research using pultruded glass fiber-reinforced polymer (GFRP) decks, focused on behaviour under various strength and serviceability loading conditions. Failure modes observed were specific to delamination of the flexural cross sections, local crushing under loading pads, web buckling and lip separation. However certain failure mechanisms observed from in-situ installations differ from these laboratory results, including behaviour of the connectors or system of connection, as well as the effect of cyclic and torsional loads on the connection.

This thesis investigates the role of mechanical and non-mechanical connectors in the composite action and failure mechanisms in a pultruded GFRP deck system. There are many interfaces including top panel to I-beam, deck panel to girder, and panel to panel, but this work focuses on investigating the top panel connection. This is achieved through comparative component level shear, uplift, and flexure testing to characterize failure and determine connector capacity. Additionally, a connection of this GFRP deck system to a concrete girder is investigated during the system-level test. Results show that an epoxy non-mechanical connection may be better than mechanical options in ensuring composite behaviour of the system.
This thesis is dedicated to the greatest mom under the sun, my mother, Boye Gana, without whom this work could not have been possible.
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DISCLAIMER

This work represents the opinions, findings, and conclusions of the author and does not necessarily reflect the views of Zellcomp or Florida Department of Transportation.
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CHAPTER 1

INTRODUCTION

1.1 Introduction

1.1.1 Examples and General In-Situ Applications of GFRP Decks or Materials

On US highway 151 near the city of Waupun, Fond du Lac County, Wisconsin a bridge was constructed using an innovative FRP reinforcing three part system consisting of an FRP stay-in-place (SIP) deck panel, an FRP bi-directional grid, and FRP rebar (Bank, 2005). Another case study is that of Macon county, North Carolina where an old bridge was replaced by a newer one which employed the use of GFRP rebars as reinforcement for the deck instead of epoxy coated steel rebars (Gergely, 2007). Similarly, just completed in July 2003 is the Route 668 Bridge over Gills Creek in Virginia. This Franklin County bridge is designed with the deck of one span reinforced with GFRP bars as the top mat while maintaining steel rebars for the bottom mat (Phillips, 2004). In a very different design unlike the ones already mentioned, a worn and aged reinforced concrete bridge deck of an old 340ft long steel truss bridge was completely replaced by an 8 panel A GFRP deck in Snohomish county, Washington (Brown, 2008). Lastly but of particular relevance to this research is the bridge at Hillsboro Canal in Belle Glade, Florida, where a GFRP deck was installed. This installed pultruded GFRP deck consists of a top plate as well as I-shaped webs that are made of the same material (McCall, Peng, Singh, & Hamilton, 2011)
In recent years decks made of glass-fiber reinforced polymer (GFRP) have been increasingly considered and used as replacement alternatives to traditional steel and concrete decks. In some cases as in Washington, the whole deck may be made of GFRP while in other cases such as in North Carolina and Virginia; GFRP components such as rebars may be used in place of conventional steel reinforcement. Properties that render GFRP the preferred choice include its corrosion resistance, light weight, transparency to radar and radio transmissions, high tensile strength, and the relative ease of assembly when compared to other older construction materials such as concrete or steel. There are many different manufacturing methods for GFRP deck systems. These methods include but are not limited to; vacuum assisted resin transfer moulding (VARTM), pultrusion, and open mould hand lay-up, though about 41 percent of most installed FRP deck systems have been pultruded (O'Connor, 2008). Abulizi et al (2011) also explores a relatively more recent method of glass-fiber reinforced composites manufacturing that is based on a technique of automated fiber placement using in-situ ultra violet curing. However, the deck systems of relevance to this research are manufactured through pultrusion.

1.1.2 Pultrusion

Pultrusion is a manufacturing process whereby continuous fiber rovings or mat (reinforcement) are pulled through an impregnation system or a resin bath (mostly thermoset material), then guided through a preformer to remove excess resin (debulking) before being drawn through a heated die (initiate curing) to create a constant FRP cross section which can be cut to size. A typical industrial pultrusion setup can be seen at the websites of any of the major manufacturers of pultruded decks such as Strongwell, Duraspan or Zellcomp. FRPs are made of
a fiber and resin component. The fiber is the reinforcement and the main function of the fiber is to carry the load, provide stiffness, strength, thermal stability and other desired structural properties (TUAKTA, 2005). The desired structural properties depend on the type of fiber selected, orientation of the fibers and other factors. Three main types of fibers used are; glass, aramid and carbon fibers. The resin or matrix component of FRP, besides many other functions, serves the primary purpose of binding the fibers together through adhesion. Although most resins used by well known companies have properties that are proprietary, they are mostly made up of thermosetting polymers such as vinyl ester, epoxy, polyester and others. In the pultrusion process, resins are also often combined with pigments, fillers and catalysts. The pultruded GFRP materials and deck system evaluated in this research were fabricated using E-glass fibers and isopolyester resin.

1.2 Related Laboratory tests, Field tests and Motivation

1.2.1 Background

Constructed in 1976, a bridge located over the Hillsboro canal in Belle Glade, Florida needed rehabilitation a few years ago. This bridge was initially designed using steel stringers together with a steel grid riding surface. Over time seasonal heavy loading (from October to mid-April) contributed towards damages sustained by the steel grid. The heavy loading came from the frequent passage of sugarcane trucks during the harvesting season. Local damages were initially patched by using steel plates but the need for a better system led to the consideration of a pultruded low profile Zellcomp GFRP deck. This was installed on the bridge in August 2009. Figure 1 (McCall et al., 2011) shows installation of these GFRP deck panels.
Prior to the deck’s installation on the bridge, previous study and laboratory testing of the same Zellcomp deck type was done. Vyas et al (2006) performed failure, fatigue, and skewed tests on the GFRP deck. The fatigue loading for the test was applied at two locations on the deck between equal spans at a load rate of 3 Hz for a total of 2 million cycles and between 2 and 80KN per loading pad (2006). Failure tests were also performed sequentially on each of the two equal spans of the continuous deck wherein each span was loaded until failure in the form of a loud sound accompanied by a load drop of about 13 to 27 percent was observed. In each of the cases, both failure and fatigue, the results reported reflected that the failure mechanisms observed; include web buckling, separation between the lips of two panels, and delamination between the web and bottom plates of the GFRP cross section (found after dissecting the failed specimen near and between the two loading locations). Pictures regarding these reported failure modes are present in the aforementioned report. The report also notes that in the laboratory experiment, no damage was observed at the shear stud connections between the steel beam and
the deck (2009). In addition, tests performed at the University of Washington (briefly mentioned in section 1.1.1) did not investigate failure of the top plate connection or the effect of skew on this connection (see report for more details).

However, field tests conducted over the Hillsboro Canal Bridge involving the same Zellcomp deck presented issues or yielded results which were quite different from laboratory results by Vyas et al. The bridge was constructed with the same GFRP deck placed upon a steel frame superstructure with the bottom GFRP panels attached to the steel girders with grout pockets containing steel studs welded to the girders (McCall et al., 2011).

Upon completion of the bridge in August 2009, two bridge tests, one in October 2009 and the other a year later in October 2010 were conducted to evaluate the relative performance of the bridge after one year of service. There were 3 types of tests; a static, rolling and a 35 MPH test in the 2010 period. Testing also involved the use of thermocouples to evaluate temperatures at select depths of the deck panel (2011).

Monitoring of the bridge continued from October 2009 to April 2011. Details of the monitoring program can be found in the final report by McCall et al. However certain relevant results and observations from the report include the following:

- Severe weather in December 2010 prompted an emergency partial harvest of sugarcane crop. In addition the lifting of weight limits on agriculture related trucking caused the Hillsboro canal bridge to experience a significant increase in heavy truck traffic.
- Thermal gradients were observed within the top of the bridge deck resulted in the top plate reaching a minimum of 30 °F hotter than the interior of the GFRP deck causing thermal expansion of the top panel relative to the bottom panel.
• Months after the bridge tests, it was observed from further monitoring that there was severe degradation of the wear surface above and severe deterioration of the top plates and web portions of the deck.

• The grout in the grout pocket containing shear studs connecting the deck to the steel girders suffered noticeable degradation in form of cracks. It became loose and resulted in a general decrease in stiffness in lane 1 of the deck.

• A large number of mechanical fasteners came loose from the top plate towards the end of the bridge which the authors attributed might be due to the relative thermal expansion of the unrestrained end of the top plate to the bottom deck panels.

• The combination of the different problems such as the failed fasteners, skew, heavy traffic, and the deterioration of the grout bearing pad caused such an accelerated wear and tear that repair of the bridge was necessary as reported by Hamilton et al at the end of the 18 month monitoring program.

1.2.2 Research Questions and Objectives

Certain important concerns arise after comparing laboratory results as reported by Vyas to field results as reported by McCall et al. It appears previous laboratory work, though detailed, did not entirely capture or successfully predict some of the failure mechanisms observed during the 18 month monitoring program of the bridge deck installation. For instance in the laboratory study by Vyas et al, it was noted that “no damage at the shear stud connections was observed,” whereas the opposite was the case in the field study as mentioned in the previous section. Degradation of the grout pockets containing the steel studs was observed and the grout was loose
and sustained cracks. In addition prior laboratory investigation and reporting did not include the role of connectors in the deck system nor predict their failure mechanism which could have helped better explained why a lot of fasteners came loose during the field evaluation by FDOT.

The objective of this current research is to study and investigate the strength and failure behaviour of the same mechanical connectors used in previous laboratory configuration under different loading conditions such as shear, uplift and bending. In this study another mechanical connector as well as a non-mechanical option (epoxy) were also investigated as viable alternatives. The scope of this research includes component level testing as well as system level testing to better understand the behaviour of the different connection methods. On the system level, this research investigated the effect of cyclic load on the performance of the selected methods of connection. In addition, connection of the GFRP deck system to a concrete girder is investigated during the system-level tests.

1.3 Literature Review

A lot is known about the behaviour of GFRP decks with regards to deflection, ultimate capacity and failure modes. Experiments have been conducted in both laboratory and field settings; a few of which have been mentioned above. However certain other technical challenges and questions related to these GFRP structures remain. Several of these issues are enumerated by Bakis et al.(2002); among them are the need for efficient attachment of decks to stringers, the fatigue behaviour of panels and connections, and the resilience and efficiency of the wear surface.
Many researchers agree that the delamination of the wear surface on GFRP bridge decks is mainly a consequence of the thermal incompatibility between the wear surface and the GFRP deck. Wattanadechachan et al. (2006) proposed a more resilient two-layer hybrid wear surface system for GFRP decks after investigating the thermal compatibility issues between several wear surface materials and GFRP decks.

With regards to grout type connections of FRP bridge decks to steel girders, experimental work by Moon et al.,(2002) showed that an improved connection, containing a larger volume of grout with steel spirals for grout confinement around three shear studs welded to the steel girders, was adequate. Unlike the two earlier tested connections that used a composite sleeve to confine the grout, results of the fatigue testing of this third improved connection, revealed that the connection survived fatigue loading with minimal loss in stiffness.

Based on a study of material tests and the material properties determined from the tests, Hyeong-Yeol Kim et al. (2004), proposed applicable GFRP patterns, cross-sectional dimensions of deck profiles and deck-to-girder connections. The schematic of the proposed deck to existing precast concrete girder is presented in the article. This connection involves a “shear pocket” beneath the FRP deck, the incorporation of new shear studs to adjust the elevation of the FRP deck over the girders while retaining and using previously installed shear studs on the girders.

In a more detailed study, a non-grouted sleeve-type connection for attaching FRP decks to steel girders was investigated by Davalos et al.(2011), for stiffness, strength, degree of composite action and fatigue resistance. This study involved component-level static and fatigue tests on push-out specimens as well as system level tests on a 1:3 scaled bridge model. Results of the study indicated that the shear connection was sufficient for securing the FRP deck while
transferring the interface shear force between the girders and the deck. Additionally, the partial composite action achieved by this connection was deemed to be adequate in ensuring proper response of the bridge under static and fatigue loading while securing the deck against in-plane and uplift forces. Full composite action may not have been desired as a result of potential adverse effects on the bridge system due to incompatible thermal coefficient of expansion between FRP and steel.

A slightly similar but earlier type of connection involving a GFRP deck of a rectangular cross-sectional shape was proposed by Ki-Tae-Park et al.,(2006). The proposed connection of GFRP to bridge decks, concrete or steel, was the result of a finite element analysis where the connection failure was checked by the Tsai-Hill criterion. Details of the connection are presented in the article. The commercial software ABAQUS was used in determining the right bolt diameter, edge distance and stiffening plate design to compensate for stress concentration around bolt holes. Some final recommendations of the study include; the diameter (D) of the anchor bolt should be at least 20 mm, the material for the stiffening plate should be steel and the recommended edge distance of the anchor bolts should be 1.4D-2.0D.

Another experimental research by Correia et al.,(2007) examines for characterization, the flexural behaviour of a hybrid system. A GFRP I-profile connected to concrete through stainless steel bolts is examined in this research. The behaviour of the shear connection to both materials was evaluated through shear connection tests and the results of these tests employed to design simply supported GFRP to concrete hybrid beams. Further bending tests of the designed hybrid beams, yielded results wherein the hybrid beams showed considerable stiffness and strength increase when compared to the behaviour of just the GFRP profile.
Keller et al., (2004) performed laboratory fatigue experiments on adhesively connected pultruded profiles. In this study, the goal was to determine the existence of fatigue limits as well as evaluating measurement methods of detecting damage initiation and progression. Results of the experiment showed fatigue limit of 25% of static failure load at 10 million cycles. However, detection of damage initiation or progression was not achieved given the test set-up. It was also noted that failure was always brittle without warning in the adherents.

In an attempt to verify the degree of composite action of a bolted GFRP bridge deck to steel girder connection, Ki-Tae Park et al., (2006) performed some static tests where bolt fastening intervals were varied. A quantitative estimation of the degree of composite action was achieved by comparing the neutral axis obtained through experimental strain readings to the theoretical neutral axis obtained by calculation. Results showed that for the different fastener intervals, the degree of composite action increased as the fastening intervals of the bolts were shortened.

The aforementioned studies highlight some important research relevant in understanding the behaviour of deck connections in general, and in more specific cases of the type of GFRP deck or components under consideration in this work.

1.4 Research Plan

Many are the advantages and hence the motivations for using FRP and GFRP decks in construction. However, as a relatively new material compared to steel, there are new challenges, some of which have been mentioned earlier. The case study of direct relevance to the research is that of the Hillsboro Canal Bridge. Consequently, the objective of this research is to conduct
several laboratory tests to characterize the role of the deck to top plate connectors in the composite action and failure mechanisms of the deck under different loading conditions. Two mechanical and one non-mechanical connectors were selected for investigation in this research. Connector behaviour was investigated at the component level for shear, uplift, and flexure. Response of multi-span deck segments were investigated under the combined action of cyclic flexure and torsion due to skew. Chapter 2 addresses the component level testing, the results and the analysis of the results. Chapter 3 investigates a system-level testing of a skewed GFRP deck and the accompanying results. Chapter 4 introduces the testing and results of a simple un-skewed deck to concrete girder connection under fatigue loading while chapter 5 presents an overall conclusion and challenges as well as recommendations for future investigation and analysis.
CHAPTER 2

SHEAR, UPLIFT AND SMALL-SCALE BENDING TESTS AND PROTOCOLS

2.1 Shear tests and protocols

This chapter describes the design, test setup, and instrumentation of three different types of component-level tests; shear, uplift and small-scale bending. Each of these three tests was conducted using three connector types; two mechanical and one non-mechanical. Three tests were performed per connector used to observe the level of consistency in particular fastener-FRP behaviour. Altogether twenty seven tests were conducted; three for each of the three sub-categories presented in this chapter. These tests were performed in the Structural laboratory at the University of Central Florida (UCF) using the Instron/SATEC 200 kip Universal Testing Machine (UTM). The GFRP deck system and top plates used were manufactured by Zellcomp. The deck system consisting of a bottom panel with T-sections and a top plate, were cut to small enough sizes to enable testing in the UTM. The following sections 2.1 and 2.2 address shear testing and results respectively. Subsequent section within this chapter address the uplift and small-scale bending experiments and results

2.1.1 Shear Experimental specimens and setup (Mechanical fasteners)

For this type of loading, there are two mechanical stainless steel type epoxy infused fasteners that came with the system; one blue (connector A-of about 65 ksi) and the other silver (connector B – of about 65 ksi) as shown in figure 2. These two are considered under shear force
in lap shear test setup and methods similar to those described in ASTM standards D3163 and D1002. For the test, a T-section was cut to a suitable size, 10in long and a top plate of length 12in and width 10.5in was cut as well for the shear test.

![Figure 2: Two different types of mechanical connectors](image)

A total of six tests were performed for the mechanical connectors, three tests per connector. After cutting the GFRP components to size the following steps were taken towards ensuring successful preparation and testing;

- For each test/specimen corresponding holes (1/4 in and 1/3 in), one per test for the connection were drilled pneumatically through the top and center (web) of the GFRP T-section. Similarly, holes were equally drilled through the top plate.
- The GFRP top plates were then attached to the GFRP T-sections using either mechanical connector A (blue) or connector B (Silver) as shown in figure 3. An L-bracket was also
attached as a washer to the connector. Another L-bracket was also attached only to the top plate using a G-clamp.

- The vertical end of the top plates was trimmed to fit the self adjusting grip of the load cell.

![Figure 3: Configuration of assembled specimen](image)

- The final assembled specimen for each test was setup like a lap shear installation in the UTM. Steel plates were bolted down to the lower crosshead of the Instron universal test machine and the T-section was placed upon this steel plate while another high stiffness reaction plate was placed and secured by bolts right above the T-section while allowing just enough space for the unrestrained top plate.
• The trimmed end of the unrestrained top plate was placed in the grips of the testing machine such that the applied load coincided with the long axis of the specimen.

• Two Linear variable differential transformers (LVDT) were attached to the final setup, one bearing directly on the L-bracket attached to the top plate and the other installed bearing on the L-bracket around the connector. These are to measure the deformation of the connector during the shear test and the movement of the plate. Figure 4 shows the final setup.

Figure 4: Setup for shear test
2.1.2 Experimental specimens and setup (Non-Mechanical adhesive connection)

The test setup for the selected non-mechanical fastener remained the same as that for the mechanical connectors. The principal difference lay in the preparation of the specimen before testing as well as the difference in connection. For this option, the adhesive selected was CarbonBond™ 200P, a two part epoxy structural adhesive. In comparing this form of connection to the mechanical, the chosen bond area of the top of the T-section to the top plate was 8 by 4 inches (see appendix A). This is more of an estimation based upon the calculated (see appendix) equivalent connector spacing in comparison to previous research by Vyas. Prior to mixing and applying the epoxy adhesive, the glossy surface of the T-section was prepared in accordance with similar methods as described in ASTM D2093. After sanding of the surface, cleaning, application of adhesive and adhering the top plate, some constant weight of about 25 pounds was applied to the specimen to allow time for the epoxy to cure. Figure 5 shows the bond area where the epoxy was applied and the weights applied to the specimen. Testing was not commenced until at least five days after adhering both parts together.

Figure 5: Preparation of specimen for shear test (epoxy)
Figure 6 shows the setup for the epoxy shear test that is also the same setup used for the mechanical connectors.

2.1.3 Shear test and procedure (Mechanical and Non-Mechanical)

Shear tests and loading for both the mechanical and non mechanical connectors were conducted in the same fashion. The tests were done in an almost similar manner as described in ASTM standard D3163. The trimmed end of the top plate in each case was placed into the grips of the load cell such that loading direction coincided with the long axis of the test specimen. The outer jaws of the grips were set to engage the outer 38mm (1.5 in.) of the trimmed end of the specimen. This length of the specimen in the grip was maintained in all the tests. The specimen was then loaded to failure in each case. The load rate was 0.05 in. /min.
2.2 Shear Test Results

2.2.1 Test results for Connector A

For the first connector, the blue connector, the specimen was loaded until failure occurred. Failure was defined as the first sudden load drop. The load drop in some cases was accompanied with a little sound that turned out to be the connector splitting in half in the case of test 1 and test 3. Test 2 was the only case for connector A wherein the fastener was pulled out through the deck.

![Figure 7: Load vs. displacement for connector A in Shear](image)

Figure 7 shows the plot of the load versus displacement (connector deformation) for each test for connector A. As can be observed from the figure, test 1 and test 3 exhibit a load displacement
behaviour characterized by a reduced rate of load increase right before failure, which indicates it is the steel that is failing in this case. For test 2 the middle curve, there is more of a sudden load drop. In this case, there was more of a noticeable bearing failure around the drilled hole in the GFRP T-section and top plate which allowed the connector to be pulled through the widened hole as shown in figures 8 and 9 for the GFRP T-section. For test 1 and 3, it appears a crack in the connector initiates somewhat earlier before the load plateau occurs. In both cases as well, there is some observable stability in the specimen after reaching maximum load since the test is performed in displacement control.

Figure 8: Widened hole in top plate and failed Connector A

Figure 8 shows one kind of failure. In one of the tests, one connector is bent and eventually pulled through the T-section while the other two connectors are sheared in half. In figure 8, the fractured connector is shown along with the damage around the FRP top plate.
Figure 9: Broken and bent connectors extracted from T-section

The results of the tests for connector A can be seen in table 1. Connector pull out may have happened if there may not have been a perfectly horizontal shear force on the connector. This could have happened if an angle of load application may have been formed during loading of the specimen. Again, the GFRP has properties in different directions (and proprietary) and this may explain why the FRP around the connector failed before the connector and thereby permitted pull out. Based on the capacity of connector A (65 ksi) and approximate cross-sectional area, it is expected to fail between 1562 and 1687 pounds.
Table 1: Summary of Shear test results for Connector A

<table>
<thead>
<tr>
<th></th>
<th>Shear test failure load Connector A (lbs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test 1</td>
<td>1594</td>
</tr>
<tr>
<td>Test 2</td>
<td>1591</td>
</tr>
<tr>
<td>Test 3</td>
<td>1511</td>
</tr>
</tbody>
</table>

2.2.2 Test results for Connector B

For the case of the second mechanical connector, it was observed that none of the connectors were split in half. However, all three showed some deformation in the form of bending but all final failure occurred in the FRP substrate as the connectors were pulled out. Connector B does not fail because it is thicker than connector A, hence more shear force is required to completely shear this connector. In this case, pull out failure from the FRP controls as this is reached before the capacity of this connector is reached. Figure 10 illustrates the load versus displacement curve for the movement of the fastener under shear force. Connector B has a strength of about (65 ksi), hence based on its cross-sectional area, it is expected to fail in shear between 3000 to 3300 pounds. This explains why connector B failed only through pullout and not through fracturing.
Figure 10: Load vs. displacement for Connector B in Shear

Figure 11 shows the effect of the bearing failure on the specimen after testing. The failure is within the substrate in this case and this allows the connector to be pulled out unlike the case with connector A. Besides the capacity, connector B has better-defined threading than connector A. This could also contribute to the higher peak load required to fail the connection. There is also the contribution of the higher surface area of connector B. However in this case, the failure of the substrate controls the connection failure. Without knowing the orientation of the fibers and resin attributes which are proprietary, it becomes difficult to evaluate the limits of the interlaminar strength of the GFRP material at the interface between the material and a given connector but it is evident that after a certain stress level a bearing failure would occur. This may be avoided by keeping the stress levels below the capacity of connector A at the system level.
The summary of the failure peak loads for connector B can be seen in table 2. The peak load for the second test, for connector B is lower when compared to test 1 and test 3. This is because the connector was not completely pneumatically driven into the specimen. One challenge encountered working with connector B was the level of difficulty faced in completely driving this connector through the specimen. In the case of the second test it was not possible to completely drive the screw into the specimen. This could be attributed to be due to the bigger thread on this connector or not enough torque.

**Table 2: Shear test results for connection B**

<table>
<thead>
<tr>
<th></th>
<th>Shear test failure load Connector B (lbs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test 1</td>
<td>1839</td>
</tr>
<tr>
<td>Test 2</td>
<td>1632</td>
</tr>
<tr>
<td>Test 3</td>
<td>1860</td>
</tr>
</tbody>
</table>
2.2.3 Test results for Epoxy test

Unlike the same type test performed with the other connectors, the shear test with epoxy failed at a much higher load. The load-displacement curves for all three epoxy shows the epoxy connection is brittle. Prior to failure, certain sounds were heard. A very loud sound was heard right at failure. Loading, as shown in figure 12 for all three specimens progressed gradually and all the way to a sudden failure.

![Figure 12: Load vs. displacement for Epoxy in Shear](image)

With epoxy, preparation of the adherends before bonding is the key to a stronger or weaker bond. Without an exactly identical preparation and application process of the epoxy, a premature adhesive failure could occur. The mode of failure in each of the tests is fairly consistent. This implies that the surface preparation in all three cases did not matter as the failure occurs in the
substrate. This kind of failure depends on the interlaminar strength of the GFRP. Hence there may not be concern for connection failure of the top plate to bottom panels in this GFRP material if epoxy is used, provided the surface of the adherend is adequately prepared. A good understanding of the interlaminar shear strength of the GFRP material should be gained in order to stay below this level. Failure patterns for tests 1, 2 and 3 can be seen in figures 13, 14 and 15 respectively. A substrate failure is observed in all three tests. GFRP layers are peeled off and there is no observed cohesive failure within the epoxy and very minimal patches of adhesive failure at the interface between the cured epoxy and the GFRP surface.

Figure 13: Substrate failure in T-section for Test 1
Figure 14: Substrate failure in top plate for test 2

Figure 15: Substrate failure in top plate for test 3
The peak loads as well as the corresponding displacements for each of the three tests are summarized in table 3. In table 4, the relative performance of all fasteners is presented.

**Table 3: Shear test results for epoxy connection**

<table>
<thead>
<tr>
<th>Test</th>
<th>Shear test failure load (lbs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>17,130</td>
</tr>
<tr>
<td>2</td>
<td>16,710</td>
</tr>
<tr>
<td>3</td>
<td>17,760</td>
</tr>
</tbody>
</table>

**Table 4: Relative performance between connectors in Shear test**

<table>
<thead>
<tr>
<th>Shear</th>
<th>Connector A (lbs)</th>
<th>Connector B (lbs)</th>
<th>Epoxy (lbs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test 1</td>
<td>1594</td>
<td>1839</td>
<td>17,130</td>
</tr>
<tr>
<td>Test 2</td>
<td>1591</td>
<td>1632</td>
<td>16,710</td>
</tr>
<tr>
<td>Test 3</td>
<td>1511</td>
<td>1860</td>
<td>17,760</td>
</tr>
</tbody>
</table>

Given these test results, connector A could either be pulled out or fractured at an average shear flow of 174 lbs per inch (based on 9 inch spacing). Connector B performs better but is more difficult to drive through the GFRP material because of the bigger thread. Connector B is not fractured during testing but always pulled out from the GFRP material. Epoxy does outperform all the other connector connectors. The bonded area in all the epoxy tests was about 8 in by 4 in. Hence the shear stress resisted by epoxy before the failure in the substrate comes to an average of 33.17 kips per square inch.
2.3 Uplift tests and protocols

The objective in this test was to fail each of the fasteners by a normal force acting on the connection between the top plate attached to the T-section. This would provide an insight into the capacity of each connector as well as what kind of results to expect in the field giving this same kind of loading criteria. As in the previous section, the relative performance of each connector, given this loading criteria, is observed. Care is taken to prevent eccentricity in this experiment through the use of a self-leveling plate attached to the upper crosshead of the UTM. For this test, the UTM loading rate was 0.05in/min with loading on displacement control as in the shear tests.

2.3.1 Experimental specimens and setup (Mechanical fasteners and Epoxy)

Uplift tests for all connectors were conducted in the same manner. The test was performed in the UTM. For the mechanical connection option using either connector A or B, a top plate (12 by 4in) was attached perpendicular to the T-section using a mechanical fastener. For the adhesive connection using epoxy, the entire portion of the top plate in contact with, and perpendicular to the T-section, was bonded to it. The bond area of the top plate to the T-section was a rectangular 4in by 2in. The surface of the T-section was prepared in accordance with ASTM standard D2093 before applying the two-part adhesive. The assembled specimen was then allowed to cure for at least five days before testing. For both connection types, the assembled specimen as shown in figure 16 was then flipped upside down to yield the front view configuration in figure 17. Two aluminum boxes of equal height were then placed upon the top plate. A self leveling plate, attached to the upper crosshead of the UTM, bore directly upon the
two aluminum boxes. The aluminum boxes transfer the uplift force to the steel fastener or epoxy attaching the top plate to the inverted T-section.

Figure 16 Assembled specimen

Figure 17: Configuration of assembled specimen
A total of three tests for each connector were performed. The setup for the tests involved placing the inverted T-section between two supports. A hole was drilled through the top plate. This hole was made for the insertion of an LVDT to read the relative displacement of the Top of the GFRP.
T-section. Two other LVDTs were positioned under the assembled specimen, such that one bore directly on the mechanical connector (measuring pull-out displacement of the connector with increasing load), while the other bore directly upon the top plate towards the edge (used to check for eccentricity). In the case of the epoxy connection, it made sense to use only two LVDTs during testing, with one bearing upon the GFRP top plate while the other one upon the inverted T-section through the hole drilled in the top plate.

2.3.2 Uplift test and procedure (Mechanical and Non-Mechanical)

With the same loading rate for all cases, each specimen was loaded until failure was observed in the form of a load drop. In this test no sound accompanied failure. All LVDT readings for each test were compared against each other as a means of verifying if the load applied was normal. The loading was normal hence bending was not induced in the connector based upon the test setup and loading procedure.

2.4 Uplift Test Results

2.4.1 Test results for Connector A, and B

For this loading criteria, connection failure behaviour is the same for both mechanical connectors. None of the connectors fail by fracturing which can be verified by calculation knowing the properties of the connectors as previously stated. Shear failure at the interface between the connectors and the GFRP materials is the mode of failure. Performance of both connectors is almost identical however connector B performs better than connector A on average. This may be due to the bigger threads on connectors B. The load-displacement curves,
which can be seen in the following figures 19 and 20 for each of these tests, show the deformation of the GFRP material as loading is gradually increased.

Figure 19: Load vs. displacement for Connector A in Uplift
Figure 20: Load vs. displacement for Connector B in Uplift

Figure 21: Failed Connection (pullout)
Figure 21 shows a typical fail pattern for the mechanical connectors. For the uplift loading criteria, all the connections fail the same way which involves a pullout from the GFRP material.

**Table 5: Summary of uplift test results for Connector A**

<table>
<thead>
<tr>
<th>Shear test failure load Connector A (lbs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test 1</td>
</tr>
<tr>
<td>Test 2</td>
</tr>
<tr>
<td>Test 3</td>
</tr>
</tbody>
</table>

**Table 6: Summary of uplift test results for Connector B**

<table>
<thead>
<tr>
<th>Shear test failure load Connector B (lbs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test 1</td>
</tr>
<tr>
<td>Test 2</td>
</tr>
<tr>
<td>Test 3</td>
</tr>
</tbody>
</table>

There was some evidence of a little eccentricity by comparing LVDT readings however this was not much.
2.4.2 Uplift test results for epoxy

The load displacement curve (figure 22). The first test using epoxy yielded the highest peak load before failure when compared to the other fasteners. There is eccentricity in the epoxy specimens and more testing is recommended to more accurately determine the stress gradient and hence predict the pull-off load if uplift force is ever a major concern for design. However in all cases, the failure mode is controlled by the interlaminar strength of the GFRP material where since failure occurs in the substrate. For the epoxy connection, an additional test was performed in which case the bond area was doubled in an attempt to assess if the peak failure load would equally be doubled but the test result shows that this does not happen. The failure patterns observed are shown in the following figures 23 and 24.

The failure is in the substrate in each of these pictures with the exception of test 3 which shows a bit of an adhesive failure of about 6 percent. A closer examination of the failed specimen in each of these pictures reveals the different fibers with different orientations pulled off from the surface of the T-section while still attached to the epoxy. It was also not possible to detect the direction in which failure began or progressed during testing. The variation in epoxy performance could also be due to small but significant difference in application of the adhesive to the adherend or differences in the amount of time for which the adhesive was exposed before joining the two surfaces to be bonded.
Figure 22: Load vs. displacement for Epoxy in Uplift

Figure 23: Failure pattern for test 1 (left) and test 2 (right)
Figure 24: Failure pattern of Test 3 (left) and Test 4 (right)

Table 7: Relative performance between connectors in uplift test

<table>
<thead>
<tr>
<th>Uplift</th>
<th>Connector A (lbs)</th>
<th>Connector B (lbs)</th>
<th>Epoxy (lbs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test 1</td>
<td>663</td>
<td>875</td>
<td>905</td>
</tr>
<tr>
<td>Test 2</td>
<td>827</td>
<td>891</td>
<td>791</td>
</tr>
<tr>
<td>Test 3</td>
<td>803</td>
<td>817</td>
<td>642</td>
</tr>
<tr>
<td>Test 4</td>
<td>N/A</td>
<td>N/A</td>
<td>1107</td>
</tr>
</tbody>
</table>

Generally, connector B performed best for the uplift tests. This can be attributed to the thread pattern, thickness and larger surface area of this connector in contact with the GFRP material.
2.5 Simple Bending Tests and Protocol

A simple bending test was performed using each of the connectors to see the effect of the loading criteria on the connection of the top plate to T-section. Although failure of the connection was not expected in this static test, each of the assembled specimens was loaded to failure. With the exception of size of the specimen and connection of deck to steel stringers, this static test is almost a scaled model of the static experiment performed by Vyas. In his test, there were eight webs, so that eight T-sections were connected to steel stringers at the bottom (through grout pockets) and to a top plate at the top with a polymer concrete wear surface above the top plate.

For this test the peak failure load of the deck section with just one T-section and top plate was predicted to be within the vicinity of 37kips, based upon the failure load obtained from previous research by Vyas (2006). The calculation involved in reaching this estimation of the failure load can be seen in the appendix. For the tests, a total of 6 tests, two experiments for each connector were performed.

2.5.1 Experimental specimens and setup (mechanical and non-mechanical fastener)

For all connectors, T-sections of length 558.8 mm (22in) were cut and top plates of corresponding length were cut and attached to the T-section using each connector type. A connector spacing of 9 inches was adopted. Top plates were attached as well using epoxy after the preparation of the adherends for bonding. The bond area for the epoxy was selected to be the same as the tributary area for the mechanical fasteners. The area bonded was about 457.2mm by 101.6mm (18in by 4in). In all cases, an LVDT was fixed to measure the middle displacement of
the T-section. Two LVDTs were also placed at the supports. The final setup can be seen in figure 25.

2.5.2 Simple bending test and procedures (mechanical and epoxy)

All test specimen were loaded the same way at a displacement control load rate of 0.05in/min in the UTM using two symmetrically placed load points. The specimens were loaded until failure was observed. The final test setup can be seen in figure 26. Failure was observed when a very loud noise was heard during loading after which load drop was also observed for all the sample beams.
2.6 Simple bending Test Results

2.6.1 Test results for Connector A, B and Epoxy

No connection failure as expected was observed for any of the elected connector options. This result is in agreement with the result from previous laboratory test by Vyas. In this case, the failure observed was mostly in the load bearing parts of the GFRP material. The load-displacement curves for all the six tests are highlighted in the following figure 27 and table 8.
Figure 27: Load displacement graph of small GFRP beams in bending

Table 8: Failure load of simply supported beam

<table>
<thead>
<tr>
<th></th>
<th>Connector A (lbs)</th>
<th>Connector B (lbs)</th>
<th>Epoxy (lbs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test 1</td>
<td>39,960</td>
<td>36,090</td>
<td>39,090</td>
</tr>
<tr>
<td>Test 2</td>
<td>39,460</td>
<td>33,430</td>
<td>39,370</td>
</tr>
</tbody>
</table>

Failure modes observed in all six tests include web buckling, delamination in the web and flange as well as separation in the upper and lower lip. These failure modes are consistent with the
failure modes reported by Vyas from previous research. The failure modes from all six tests are shown in the following remaining figures of this chapter.

Figure 28: Failed specimen

Figure 29: Failure modes (a) web buckling and (b) lower flange delamination
Figure 30: Failure upper flange delamination/separation

(a) Web buckling  
(b) Upper flange separation/delamination  
(c) Upper flange separation/delamination
CHAPTER 3

DECK TO CONCRETE GIRDER CONNECTION

3.1 Motivation

Researchers have often considered possible GFRP deck connections to steel girders, but seldom have they considered an FRP deck connection to already existing precast AASHTO (American Association of State Highway and Transportation Officials) bulb tee concrete girders. One of the potential advantages of this deck is that its pultruded cross-sectional shape/profile, allow this deck to be recommendable for consideration as a viable deck system for this purpose. Additionally, a principal goal of the system-level testing in this chapter, is to observe the relative performance of epoxy and connector A, (in connecting top plate to T-section) and to employ the understanding gained from the results of component level testing in the previous chapter, in characterizing connection failure where and if it is observed.

Two specimens are used; one employing connector A to connect the top plate to the T-section while the other is with epoxy. This experimental investigation is primarily motivated by the need to better understand the cause of the loosening of the screws connecting top plate to T-section in the Hillsboro Canal Bridge as discussed in chapter 1. However, due to casting, this test does not contain skew, and therefore will not have torque plus stiffness differential that likely affected the in-situ deck. The objective is to apply cyclic loading to the system in an attempt to mimic the effect of the cyclic fatigue loading caused by traffic over time. In the process of achieving this primary goal, the performance of the proposed deck to concrete girder connection is equally studied to determine the viability of the suggested connection to concrete girder.
3.2 Specimen details, Preparation and Instrumentation

Two specimens were tested in this chapter. Each specimen consisted of a deck with two T-sections. The only difference between both specimens is that the top plate in deck 1 was connected to the T-section via connector A while the top plate in deck 2 was attached to the T-section using the CarbonBond™ 200P epoxy. The span and width of the deck for each test specimen were kept constant at 44 inches (span) and 16 inches (width) respectively. The following additional steps were taken towards and in assembling the final specimen to be tested;

- Six forms (20in Long and 8in high by 4in wide) were made and concrete mix of strength 5000psi, were poured into the forms. A no.3 dowel (J-bar) with a 180 degree hook was also installed into the grout with the depth of embedment as 5in. The grout was then allowed to set and gain optimum strength for 28 days.

- After 28 days, a hole was drilled at three locations, one at each support end and at the center of the span (22in). These holes were drilled through bottom of the deck, in-between the two T-sections.

- Since the deck has a low profile (a depth of only 5in), the dowels on the concrete blocks had to be modified. They were straightened, and cut to a height of 5 in above the concrete blocks. They were then inserted through the holes at the bottom of the deck before being bent to form an angle of 60 degrees with the horizontal bottom of the deck. The cross section of the final setup can be seen in figure 31.
Figure 31: Cross-section of proposed deck to concrete connection

- An area for a grout pocket of about 5in by 7.5in was then defined around the bent rebar and a temporary grout containing form, built around this marked area.
- A Rapid set concrete mix™ was then prepared and poured into the grout pocket (7.5in by 5in by 3.75in). The properties of the grout are such that it reaches 3000psi strength in one hour and a strength of 5500psi after 7 days. The specimen was then left undisturbed for seven days (see figure 33) for the concrete mix to reach the desired strength.

Figure 32: Grout pockets
• After 7 days a top plate of similar dimensions (44in by 16in) to the deck being tested, was cut and attached to deck 1 by means of connector A at a spacing of 9in (228.6mm). A total of 8 metal fasteners (connector A) are used on deck 1 and this can be seen in figure 33.

Figure 33: Assembled deck 1

• For deck 2, a top plate (44in by 16in) was attached to the deck but only after epoxy was applied to the tributary area (18in by 4in), equivalent to the 9in connector spacing on each web. Prior to epoxy application, the bond area was prepared in accordance with ASTM D2093. The bond area can be seen in figure 34.
Both tests in this section were performed using an MTS servo-controlled actuator. The hydraulic actuator had a capacity of 100 kips. Labview data acquisition system was used in recording all data. A combination of linear variable differential transducers (LVDT) and strain gauges were used in gathering measurements. Four LVDTs were installed to get displacement measurements at both mid-span and the two support locations. Two strain gauges were also installed at both mid-spans and all measuring devices (LVDT and strain gauges) were calibrated before testing. Three 100 ton hydraulic jacks were placed beneath the bottom end of the actuator of the MTS and equally spaced apart as the support locations on the deck. Steel plates were then placed upon the hydraulic jacks before placing the deck with each of the concrete blocks directly upon the steel plates. Before being changed, neoprene pads were initially used for the load to be applied by the spreader beam. The final setup can be seen in figure 35.
3.3 Loading procedure

The two specimens Deck 1 and Deck 2 were subjected to one million cycles of repeated high amplitude loading at a frequency of 2Hz. The load range was from 3.6 kip to 36 Kip. This loading was applied using the MTS. Both Deck 1 and Deck 2 were loaded at two symmetrically spaced points, midway between each span as shown in figure 35. Due to the high amplitude loading 2 layers GFRP strips (3in wide by 18in) were used to replace the two steel HSS sections in transmitting the load from the spreader beam to the deck. Initially, two neoprene pads were used before the steel HSS section but because they were not stiff enough the actuator could not cycle between the desired high and low load range. Before beginning the cyclic loading, the decks was first loaded monotonically to about 37 kips to ensure there were no instabilities in the
setup. After this was confirmed in each case, the cyclic test was then started with soft start option to prevent a sudden impulse load on the deck. Deck 1 did not fail after the first million cycles so testing was continued on it at even higher amplitude (5.4 to 54kips) after the completion of the initial one million cycles.

3.4 Test results and analysis

3.4.1 Deck 1 Results

Monotonic testing on Deck 1 showed the setup to be stable and there were no observed cracks in the grout pockets except for a surface crack on the concrete block. Cyclic tests were commenced on Deck 1 shortly after loading monotonically to a little over 36 kips. After about ten thousand cycles, additional inspection of the specimen revealed a long crack in the grout pocket on each end of the deck as shown in the following figure 36.

![Figure 36: Crack in Deck 1 observed after about 10000 cycles](image-url)
Additional observation of the specimen during the first round of testing revealed some sounds which was as a result of the spreader beam rubbing on the HSS section which was initially used for this first test with Deck 1. The top plate also seemed to bend independently of the bottom panels (T-section). Testing progressed until the completion of one million cycles. Since no failure was observed upon completion of one million cycles, the decision was made to continue the test on the same Deck 1 for another one million cycles but on higher amplitude. The new load range was from 5.4 kips to 54 kips at a frequency of 2 Hz. Deck 1 did not make it all the way to another million cycles. A crushing failure of the north side steel HSS section (used in transferring load from the spreader beam to the deck) caused most of the load to be carried by only one T-section on the north side. This caused the web crushing or buckling on this side of the deck. This failure was observed after about 138,000 cycles and this caused the test to be stopped. An autopsy of Deck 1 revealed that all the connectors (A) joining the top plate to the T-section had either loosened of were completely fractured. There was noticeable degradation of the grout pocket linking the GFRP deck to the concrete blocks. Additionally, the top plate was noted to have suffered some significant damage. The wear and tear on Deck 1 was caused by the premature failure of the steel loading pad and consequently the resulting high amplitude loading of the North part of the deck. Figure 37 shows all the damages; fractured connectors, degraded grout pockets, and partially crushed T-section. However, there was no observed failure between the dowels embedded in the concrete block. If the problem of the grout pocket cracking under cyclic loading can be addressed, this would make this a viable connection system of a deeper profile of this type of deck to an already existing concrete girder.
Figure 37: Failed Deck 1

Figure 38: Fractured connectors (A)
3.4.2 Deck 2 Results

Like the first specimen, results of the monotonic test showed the setup to be stable. Cyclic test began shortly after monotonic testing. Observation of the test specimen after several thousand cycles showed no signs of the top plate bending out of plane or independently of the T-section. An inspection of the specimen after about 200,000 cycles only revealed a crack in the south grout pocket.

![Crack in grout pocket (Deck 2)](image)

**Figure 39: Crack in grout pocket (Deck 2)**

Besides the initial observed crack in one of the grout pockets, no further deterioration of the deck was observed. Upon completion of one million cycles, there was no observed major damage of this deck besides some minor wear on the top plate around the loading points. It was not possible to repeat a higher amplitude cyclic testing on deck 2 or any other decks as was the case with deck 1. However, a failure test was attempted on deck 2, in which case the MTS reached a load
of 100 kips (50kip per load point). At this point testing had to be stopped due to concerns about the MTS capacity being reached. There was no observed failure in deck 2.

An examination of the results of displacement, strains, monotonic failure testing of deck 2, and stiffness on each of decks 1 and 2 on a logarithmic scale reveals the following graphs;

![Graph](image)

**Figure 40: Maximum Displacement vs. Cycles**
Figure 41: Max Strain vs. Cycles

Figure 42: Stiffness vs. Cycles
Figure 43: Displacement vs. cycles for higher peak load (Deck 1)

Figure 44: Stiffness vs. Cycles for higher peak load (Deck 2)
CHAPTER 4

EFFECT OF SKEW

4.1 Motivation

One of the challenges of pultrusion is that shapes made from this manufacturing process typically end up with orthotropic properties. In the case of GFRP bridge decks, subject to flexure, deck orientation is important and in most cases, a deck placed with its strong direction perpendicular to supporting girders takes full advantages of the strength properties of the deck. However, sometimes it is only possible to install a skewed deck. This is the situation involving the bridge over the Hillsboro Canal where there is a 28 degree skew between the deck and the supporting girders.

In this chapter, the objective of the performed experiments, are to study the combined effects of skew and cyclic load on the relative performance of the suggested mechanical and non-mechanical connectors of the top plate to the T-sections. As in the previous chapter, connector A and epoxy were used. The top plate in Deck 3 is connected to the T-sections through connector A, while top plate in Deck 4 is connected to T-sections using epoxy. Both decks are skewed 28 degrees respective to load application while being subjected to cyclic loading in order to observe the effect of fatigue as well as the generated shear and torsion on the chosen type of connection. One distinction between this lab experiment and a possible field installation is that boundary conditions in this test are not fixed while a field installation would be most definitely be fixed at the boundaries. However, walking of the specimen during testing is not of much concern due to the constant load application.
4.2 Specimen details, Preparation and Instrumentation

Two specimens, Deck 3 and Deck 4 were tested in this chapter. As in the previous chapter, each specimen deck was made up of two T-sections and a top plate. The only difference between both specimens is that the top plate in deck 3 was connected to the T-sections via connector A while the top plate in deck 4 was attached to its corresponding T-section using the CarbonBond™ 200P epoxy. The length and width of the deck for each test specimen were the same, the dimensions being 44 inches for the span and 16 inches for the width of the deck. The final specimen to be tested were prepared as follows;

- Deck 3 was assembled by attaching a top plate of dimensions (44in by 16in) to the T-sections via connector A at a spacing of 9in (228.6mm). A total of 8 metal fasteners are used as can be seen in figure 45.

![Figure 45: Connector spacing on assembled Deck 3](image-url)
• For Deck 4, a top plate (44in by 16in) was attached to the deck after epoxy was applied to the tributary area (18in by 4in), equivalent to the selected 9in connector spacing on each T-section for the Deck 3. Before applying the epoxy, the bond area was prepared in accordance with ASTM D2093. The bond area can be seen in figure 46.

![Figure 46: Epoxy bond area on T-section of Deck 4](image)

Both decks were loaded using the MTS as described in the previous chapter. LVDTs were used to measure deflection of the beam at the midpoint of each span. Strain gauges also located at the midpoint of each span were used in measuring strains. The primary difference in this chapter is that both decks 3 and 4 were skewed 28 degrees relative to the loading points. This was achieved by skewing the deck 28 degrees relative to the spreader beam. The deck was supported at both ends and in the middle using GFRP strips placed upon steel plates, which were upon the three hydraulic jacks shown in figure 47.
4.3 Loading procedure

Deck 3 and Deck 4, were subjected to one million cycles of high amplitude loading at a frequency of 2Hz with the load ranging from 3.6 kips to 36 kips as was the case in chapter 3 with the MTS. Both decks were loaded at two symmetrically spaced points, midway between each span as shown in the figure above. The high amplitude loading, ensured that neither the supports nor the GFRP loading strips were walking during the test. As was the case in chapter 3, before beginning the cyclic loading, the decks was first loaded monotonically to about 37 kips to ensure there were no instabilities with the skewed setup. After this was confirmed in each case of Deck 3 and 4, the cyclic test was then commenced with a soft start. The test was also monitored at frequent intervals to ensure no problems developed such as walking of the specimen during the test. Compared to previous research, the difference here is that the ratio of the mean stress relative to the failure load (as shown later to be 64.5kips in failure test) is much higher than the ratio of the mean stress to failure load in the previous research.
4.4 Test results and analysis

4.4.1 Deck 3 Results

Deck 3 has its top plate connected to the bottom panels via connector A. Cyclic tests were commenced on this deck shortly after loading monotonically to about 37 kips. Observation of the specimen during the first 100,000 cycles showed some considerable torsion on the deck. A little walking of the specimen was equally noticed after several hundred thousand cycles. In this specimen, the top plate seemed to flex independently of the bottom panels and after about 100,000 cycles, the two end metal connectors along a diagonal came pultruded almost have an inch due to the apparent torsion on the GFRP deck. The picture in the following figure shows the loosened fasteners

![Figure 48: Loose screw after 10000 cycles](image-url)
The testing progressed on this deck for several more thousand cycles. Wear on the top plate was visible after about half a million cycles. Deck 3 did not make it to a million cycles as testing was stopped a little over 900,000 cycles because one of the webs had failed in crushing. This can be seen in figure 49 and figure 50.

Figure 49: Deck 3-damaged top plate and crushed web.
Initially it seemed only two connectors came loose from test Deck 3 but an autopsy of the deck after unloading revealed that not just two but all the connectors had either been loosened or had been completely fractured. This failure is due to the test setup and not failure of the specimen. This explains why the top plate seemed to be bend in some dissimilar way from the bottom panels as it deflects more than the bottom panels (if not fully restrained by connectors) during testing. Due to technical difficulties with data acquisition and LVDT, displacement plots and other plots could not be made for the cyclic testing of this deck 3 and deck 4.

**Figure 50: Damaged Deck 3**
4.4.2 Deck 4 Results

Deck 4 has its top plate attached to the T-sections through the use of epoxy. Testing of this deck was performed as was the case for deck 3. Monitoring within the first thousand cycles revealed the top plate to be behaving compositely with the bottom panels. Top plate moved together with the deck as though it were laminated to the GFRP T-section hence there were no signs of sliding. This makes sense as results of component level tests in chapter 2, showed epoxy to have good shear strength capacity. Deck 4 was monitored at several intervals before reaching and exceeding a million cycles. Further examination of Deck 4 after testing revealed that the epoxy connection of the GFRP top plate to the T-section did not fail as the two parts were still connected such that two people could carry deck 4 people by lifting both ends of the top plate. Figure 51 shows Deck 4 after testing completed, with no serious damage to top plate except for minor wear around load points.

Figure 51: Deck 4 after 1000000 cycles of testing
As deck 4 and the epoxy connection of the top plate to the lower panel were undamaged after cyclic testing, a monotonic test was equally performed following cyclic testing. Unlike deck 2, there was a failure in this case. A web buckling failure was observed in deck 4 at 64.5 kips.

**Figure 52: Failure of deck 4**

**Figure 53: Load-displacement plot of Deck 4**
Although deck 4 failed in monotonic testing, inspection of the specimen after testing did not show failure of the epoxy connection of the top plate to the bottom panel. Based on this and the previous result from testing deck 2, failure of the connection of the top plate to the T-sections is not a major concern when epoxy is used.

### 4.4.3 Implication of component-level tests on system-level

Results of the component-level tests in chapter 2 indicate possible failure modes observed under individual load cases. In the case of shear tests performed on different types of mechanical connectors, load-displacement plots identify the maximum load resisted by connectors A, B, and epoxy before failure.

![Figure 54: Single shear force on connector](image)

Figure 54 shows the free-body diagram of the load on the mechanical connectors for the shear tests in chapter 2. The failure modes observed include fracturing of the connectors (in the case of connector A) when the average shearing stress capacity of connector A was exceeded. Another failure mode is the bearing failure of the FRP parts immediately surrounding the fastener (for the case of connector B), which leads to the widening of the hole such that the connector (B in this
case) is easily pulled out at an angle. An appropriate mechanical connector with a capacity greater than the system-level demands (adjusted for number of cycles) can be selected.

The allowable shearing stress on the connector $f_a$ is; $f_a = \frac{P}{A} = \frac{P}{\pi d^2/4}$, where $P$ is the loading on an individual fastener (from load-displacement curve in component-level testing) and $A$ is the cross-sectional area of the unthreaded part of the fastener. An appropriate factor of safety (F.S.) can then be enforced by changing the allowable shear stress accordingly or vice-versa since; $f_a = \frac{f_{fail}}{F.S.}$ and adjusting for the cyclic demand on the connector. This capacity must then satisfy the demands of the system. The shear demands on a connector at the system level can be determined from the shear diagram and the shear flow given the load configuration in the larger-scale tests.

Figure 55: Shear diagram of system-level
In the case of this system the maximum shear to be resisted based on the loading would be 12.375 kips. For a system where the maximum shear to be resisted is an arbitrary \( V \), then the shear flow, \( q \), (force per unit length along the beam) would be: \( q = \frac{VQ}{I} \), where \( Q \) is the moment of the areas about the neutral axis and \( I \) is the moment of inertia of the cross section. Hence in order to meet the demands of the system in selecting a proper mechanical fastener at an appropriate spacing, the allowable force on such a connector divided by the spacing must be equal to the shear flow. That is; \( \frac{f_{allow} \times A_f}{s} = q_{system} \), where \( A_f \) is also the cross-sectional area of the unthreaded part of the fastener and \( s \) is the fastener spacing. S-N curves from AASHTO can be used to estimate the reduced allowable strength.

For the epoxy system, failure occurred in the substrate as shown in the component-level tests. Hence the epoxy used in this study is much stronger, and the mode of failure is controlled by the interlaminar shear strength of the GFRP material. In order to design for a different system of epoxy which would satisfy the system demands, the allowable shear strength of the epoxy \( (\tau_{epoxy}) \), should be selected such that \( \tau_{epoxy} \times t = q_{system} \), where, \( t \) is the width of application of the epoxy. The allowable shear stress of the epoxy could also be selected such that it is lower than the interlaminar shear stress of the GFRP material in order to prevent this type of failure from occurring. An epoxy system can be selected between the interlaminar shear strength of the GFRP material (upper bound) and the system demands (lower bound).

This same method of allowable stress design can also be applied for the uplift (not significant in this case but may be relevant where wind loading is present) case using the cross sectional area of the fastener and knowing the peak load from load-displacement graphs however, for this system uplift is not a concern as much as shear and can be neglected provided
heavy flexural loading which causes large deformations are avoided. No uplift failure of fastener was observed in cyclic testing.

Using S-N relationship a suitable connector could be selected when the minimum or expected fatigue life of the deck is known. A mechanical fastener with a matching or higher fatigue life can be selected to satisfy the shear demands of the system. The shear resistance, $Z_r$ in kips, for a single connector under fatigue loading is given in AASHTO LRFD Section 6.10.10.2:

$$Z_r = \alpha d^2 \geq \frac{5.5d^2}{2} \quad (1)$$

Such that;

$$\alpha = 34.5 - 4.28 \log N \quad (2)$$

Where;

$d =$ the diameter of the stud (in), and

$N =$ the approximate number of cycles for the design life

The reduced shear strength of connector A can be determined based on one million cycles of testing using equations 1 and 2 and dividing the shear resistance, $Z$, by the cross-sectional area of the connector. Alternatively, an S-N curve from AASHTO which already incorporates both equations can be used. For a brief comparison between component-level and system-level both the S-N curve and equation could be used.

From chapter 2, the average shear force resisted by connector A (diameter of 0.174 in) in component-level test is 1.565 kips and with a cross-sectional area of 0.023779 square inches, the static capacity is 65.83 ksi. However by the dividing the shear force $Z_r$ (obtained from equations
1 and 2 at 1 million cycles) by the cross-sectional area the reduced shear stress capacity of the
fastener becomes 47 ksi. Alternatively using the S-N curve, one million cycles corresponds to a
shear stress of approximately 50ksi. Both approaches yield almost the same result. The lower of
the two stress capacities can then be used to select an appropriate spacing for the connector to
ensure it does not fail at the end of a million cycles. The maximum shear the connector could
resist at this reduced capacity is; $47 \text{ ksi} \times 0.02378 \text{ in}^2 = 1.12 \text{kip}$.

Hence a recommended maximum spacing to meet fatigue demands can be determined using
equation 3.

$$\frac{N(112\text{kip})}{\text{spacing}} = \frac{V \times Q}{l}$$  \hspace{1cm} (3)

Where;

$V = 12.375 \text{kip}$, from shear diagram of the system and $N$ is the number of connectors per web

Taking the reduced capacity of the connector into consideration and based on initial spacing, the
new recommended spacing for the connector is 6 inches.

For the epoxy system, the average area of the region where GFRP substrate failure is
observed was 16 square inches. Hence an estimate of the interlaminar shear capacity of the
GFRP based on the average peak failure load (17.7 kips) is 1.1 ksi. The maximum shear stress on
the system based on the load is given by;

$$\tau_{max} = \frac{VQ}{lt} = \frac{12.375\text{kip} \times 2.23 \text{in} \times 16 \text{in} \times 0.5 \text{in}}{69.52 \text{in}^2 \times 4 \text{in}} = 0.77 \text{ksi}$$  \hspace{1cm} (4)
Based on equation 4, the system design can be optimised by selecting an epoxy that has a shear capacity greater than 0.77ksi but less than 1.1 ksi
CHAPTER 5
CONCLUSIONS AND RECOMMENDATIONS

In this thesis, the behavior of pultruded GFRP deck sections is investigated. Specifically, the role of mechanical and non-mechanical connectors between the web and top plates on composite action and failure modes are investigated. Two mechanical and a non-mechanical connectors are considered for shear, and uplift loading on component-level testing. One mechanical and a non-mechanical connector are selected for system-level flexural tests. Results show that connector A failed in shear in component-level test. Other failure modes observed include substrate failure in the GFRP when the non mechanical (epoxy) was used and pull-out through the GFRP material. Based upon the results presented in earlier chapters several conclusions and recommendations are drawn;

- The epoxy connector appeared to perform better. The shear flow achieved by the epoxy was high and during the full-scale tests, the mechanical connector appear to be susceptible to degradation due to the high amplitude cyclic loading, particularly with the additional combined action coming from the skew of the deck panels.
- If many mechanical connectors are used to achieve the same level of composite action gained with epoxy, there would be additional concerns of stress concentrations and the possibility of strength reduction of the deck due to excessive drilling. The use of epoxy eliminates this concern. However, the use of epoxy on a big structure such as a bridge deck needs to be further investigated due to durability concerns for epoxy when exposed to adverse ambient conditions such as elevated temperature.
• Another issue for this type of bridge deck would be how to apply constant pressure on the top plate till the epoxy cures and gains full strength. To address this concern, the author would recommend a combined system of epoxy and mechanical connection with the mechanical connection serving the additional purpose of maintaining the constant pressure necessary to allow the epoxy curing time.

• It is possible to connect this FRP deck to a concrete girder in a fairly simple way using grout pockets. Possible degradation of grout pockets may be prevented by grout confining reinforcement as recommended by other researchers (mentioned in literature review).

• Cyclic loading on a skewed configuration yields the most adverse effects on the GFRP deck. From all system level tests, the skewed decks with top plates (Decks 3 and 4) showed more overall degradation upon completion of one million cycles of testing.

• A fully pultruded option where the top plate and bottom panels are fabricated as one element may be the best alternative for certain installations especially if a grout pocket or mechanical connection to the girder is not needed.

• A complete understanding of the connector behavior is necessary and should be designed for as shown in this research. It is not enough to select an arbitrary spacing without evaluating the needs of the system especially in shear.

• While uplift is less likely to occur in the field, the skewed installation of this deck system not only causes reduced strength in both failure and fatigue testing but the torsion on the deck also contributes to connection failure as shown in the result of deck 3.
As shown at the end of chapter 4, once the strength of a connector is known, it can be designed for, with an understanding of the capacity and the failure mode around the top plate. Hence if the loads on the system are kept underneath these limits, then flexural and serviceability requirements would equally be acceptable. However, exceeding these limits would show the type of connector failure and movement relative to the top plate as was the case in some of the experiments.
APPENDIX: PEAK LOAD DETERMINATION

AND CONNECTOR SPACING
PEAK LOAD ESTIMATION

Previous testing involved a longer continuous deck spanning 48 inches on each of the two spans. Failure loading was achieved by applying a concentrated load to one of the center spans. Additionally, previous deck tested had a bigger cross-section with about 8 webs (4 effective), whereas the deck used in this research only used 2 webs (T-sections). The length of the deck used in this research for the system level cyclic tests was 22 inches on each span of the continuous beam (approximately half of the span of the previous test).

Figure 56: Previous cross section (1st case)

Figure 57: Cross section of deck tested in cyclic loading (2nd case)
Figure 58: Cross section of small-scale bending test (3rd case)

The peak failure load in case 1 which was not performed in this research was 370 kilonewtons or about 83.17 kips (2006). Since the material is the same an assumption is that the maximum bending stresses \( \sigma_{\text{max}} \) for a 8-web (case 1) cross-section of GFRP at the point of failure should be equivalent to the \( \sigma_{\text{max}} \) for a 1-web (case 3) or 2-web (case 2) cross section.

\[
\sigma_{\text{max, 8-web}} = \frac{M_8 y_8}{I_8}
\]

Where \( M_8 = \frac{13pl}{64} \). \( M_8 \) is the maximum bending moment for the cross-section in case 1. \( y_8 \) is the distance to the neutral axis in case 1 which is calculated to be 0.073m given the dimensions of the figure. \( I_8 \) is the moment of inertia of the cross-section in case 1, which is calculated to be \( 6.89 \times 10^{-5} m^4 \).

So, \( \sigma_{\text{max, 8-web}} = \frac{M_8 y_8}{I_8} = \frac{13}{64} \times 370 \times 1.22 \times \frac{0.073}{0.0000689} = 91.815 \text{ KN/m}^2 \)

Hence, 91.815 \text{ KN/m}^2 = (\sigma_{\text{max, 1-web}}) = (\sigma_{\text{max, 2-web}}). With the loading configuration for both case 2 (continuous beam and concentrated load on the center of both spans) and case 3 (simple bending-four point loading) and the cross-section properties; \( I_1 = 1.297 \times 10^{-5} m^4 \), \( I_2 = 2.894 \times 10^{-5} m^4 \), \( Y_1 = 0.069 m \), \( Y_2 = 0.073 m \) from the dimensions, The Peak load for case 3 is estimated to be 80 kilonewtons (18 kips) on each of the two load points for the loading
configuration. This estimate is consistent with the average peak failure load obtained from component-level tests. Similarly, since \(91.815 \text{ KN/m}^2 = (\sigma_{max})_{2\text{web}}\), the peak failure load (actuator load) for the continuous beam configuration for case 2 is also estimated to be 736.04 kn (165.5 kips). Based on this, the peak cyclic load of 36 kips (18 kips per loading pad) was selected as about 22 percent of the estimated failure load.

**CONNECTOR SPACING**

The spacing of connectors in the previous research was 0.3m. The connector spacing in this test had to be estimated and scaled to the suit the span used in this research. Hence suppose the connection was adequate for the cross-section in case 1 (continuous beam with two-point loading), as failure was not reported, then the shear flow (q) multiplied by the connector spacing for each respective span could be assumed equal. In other words, since the connectors (connector A) used are the same, then the load resisted by a connector given the 8-T cross-sections (with 4-effective webs), divided by the shear flow (q) for the 2-T cross-section would give the required maximum spacing. That is;

\[
\text{Spacing}(2\text{ webs}) = \frac{\text{load resisted by a connector (8 – web)}}{\text{Shear flow (2 – web)}} = \frac{\text{spacing}_{8\text{-web}} \left(\frac{V \times Q}{I}\right)_8}{\left(\frac{V \times Q}{I}\right)_{2\text{-web}}}
\]

Where, \(V\) is the shear force from the shear diagram of the loading configuration. The loading is the same in both cases hence \(V = \frac{11P}{16}\). where P is the peak load.

Q is the first moment of the cross-section areas about the neutral axis, and I is the moment of inertia.
Based on the cross section dimensions; $Q_8 = 667,012 \text{mm}^3$, $Q_2 = 309,451 \text{mm}^3$

Also the moments of inertia for both cases are;

$I_8 = 68,903,529.077 \text{mm}^4$, $I_2 = 28,936,353.13 \text{mm}^4$. Hence, using the earlier estimated failure load;

\[
Spacing_{2-web} = \frac{(300 \text{mm} \times 667,012 \text{mm}^3 \times 28,936,353.13 \text{mm}^4 \times 370.5 \text{kn})^{\frac{1}{2}}}{(736.04 \text{kn}/2) \times 309,451 \text{mm}^3 \times 68,903,529.077 \text{mm}^4}
\]

\[= 136.625 \text{mm} = 5.4 \text{ inches}\]

Based on this result (spacing of 5.4 inches), an increased connector spacing of 9 inches was adopted.
REFERENCES


