ABSTRACT

WU, ZHENHUA. Prestressed FRP Tubular Deck System (Under the direction of Dr. Amir Mirmiran)

An experimental and analytical study was undertaken to assess the behavior of a new FRP deck system. The deck consists of a series of pultruded FRP tubes, laid side by side on existing stringers, perpendicular to the direction of traffic. The tubes are then post-tensioned at mid-point between the stringers in the direction of traffic. The experimental work consisted of seven FRP tubular specimens crushed on their sides, eleven FRP decks in static bending, and four FRP decks in fatigue bending. The analytical work consisted of modeling of crushing test for a single FRP tube and multiple unbounded FRP tube specimens, modeling of static flexural test for a bonded and an unbounded FRP deck panel, and load rating of the floor system for the bridge with the installed FRP deck. The study showed feasibility of the new deck system for bridges with limited truck traffic and closely spaced stringers, where lack of panel action is not a concern. Failure mode, stiffness and capacity of the deck system are all functions of the FRP material properties and tube size, span length, interface bond and prestress level. In general, longer span decks fail in bending, whereas shorter span decks suffer from local shear failure due to stress concentrations at the corner of the tubes most adjacent to the applied load or the support. The deck system has some redundancy and reserved strength built into it by means of prestressing strand or bar. Short span decks are susceptible to early fatigue failure. The finite element analysis was shown to provide a good simulation of the FRP deck system. The floor system in the bridge was rated for 30 tons and 19 tons at the operating and inventory levels, respectively.
PRESTRESSED FRP TUBULAR DECK SYSTEM

by

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Biography

WU ZHENHUA, was born in Beijing, capital of P.R.China in August 1978. After graduating from high school, he started his undergraduate study in the Civil Engineering Department of the Tsinghua University in Beijing, China from September 1996. He received his Bachelor of Engineering degree in June 2000.

Mr. Wu worked in a structural design company for one year. Subsequently, he came to the USA to further his education. As a graduate student, he first enrolled in the Department of Civil and Environmental Engineering at the University of Cincinnati, Cincinnati, Ohio in September 2001. After one quarter, in January 2002, he transferred to Department of Civil Engineering at the North Carolina State University, Raleigh, North Carolina. He worked at the Constructed Facilities Laboratory of the NC State University as a graduate research assistant with Dr. Amir Mirmiran. In Spring 2003, he successfully passed his oral exam and received his Master of Science degree.

Mr. Wu has been admitted to the doctoral program at the North Carolina State University. He will working on a National Cooperative Highway Research Program project related to ultra-high strength concrete. His research is expected to have significant impact on the bridge design codes in the U.S.
# Table of Contents

List of Tables iv

List of Figures v

1 INTRODUCTION 1

1.1 Statement of the Problem 1

1.2 Research Objectives 5

1.3 Research Approach 5

1.4 Structure of the Thesis 6

2 LITERATURE REVIEW 7

3 EXPERIMENTAL WORK 15

3.1 Test Plan 15

3.2 Crushing Tests of FRP Tubes 15

3.3 Flexural Static Tests 19

3.4 Flexural Fatigue Tests 33

4 FINITE ELEMENT MODELING 85

4.1 Introduction 85

4.2 Modeling of FRP Tubes in Crushing Tests 87

4.3 Modeling of FRP Tubes in Flexure Tests 90

4.4 Load Rating of the Bridge with FRP Deck 92

5 CONCLUSIONS 114

REFERENCES 117
List of Tables

Table 2.1  Some of the FRP Bridge Decks Built in the U.S.  7
Table 3.1  Matrix for Crushing Test Specimens  39
Table 3.2  Mechanical Properties of FRP Tubes  39
Table 3.3  Summary of Crushing Test Results  39
Table 3.4  Matrix for Static Test Specimens  40
Table 3.5  Technical Properties of Electric Resistant Strain Gages  40
Table 3.6  Prestress Levels for Static Test Specimens  41
Table 3.7  Summary of Static Test Results  41
Table 3.8  Matrix for Fatigue Test Specimens  42
Table 4.1  Modifications of the Bridge Geometry  95
Table 4.2  Rating Factor and Bridge Rating of the Floor System for the Tyler Road Bridge  97
# List of Figures

| Figure 1.1 | Tyler Road Bridge | 4 |
| Figure 1.2 | Connections to Floor Beams | 4 |
| Figure 1.3 | Side View of the Bridge | 4 |
| Figure 1.4 | Floor System | 4 |
| Figure 1.5 | Rehabilitated Bridge | 5 |
| Figure 1.6 | FRP Deck and Overlay | 5 |
| Figure 1.7 | Galvanized Steel Truss | 5 |
| Figure 1.8 | FRP Tubes Between Stringers | 5 |
| Figure 2.1 | TYCOR Deck with Fiber Reinforced Foam Core and Stitched Fabric Skin | 12 |
| Figure 2.2 | Schematics of DuraSpan Deck Panel | 12 |
| Figure 2.3 | Transport and Installation of DuraSpan Bridge in Drake County, Ohio | 12 |
| Figure 2.4 | Cross Section of Superdeck System | 13 |
| Figure 2.5 | Bonding and Assembling Superdeck in Xenia, Ohio | 13 |
| Figure 2.6 | Hardcore Honeycomb Core Deck | 13 |
| Figure 2.7 | Kansas Honeycomb Deck System | 14 |
| Figure 2.8 | TechDeck System | 14 |
| Figure 3.1 | FRP Specimen in Crushing Test | 43 |
| Figure 3.2 | Corner Cracks | 43 |
| Figure 3.3 | Flexural Cracks in Web | 44 |
| Figure 3.4 | Oblique Shear Cracks | 44 |
| Figure 3.5 | Web Buckling | 45 |
| Figure 3.6 | Load-Deflection Curves for Crushing Tests of FRP Tubes | 45 |
| Figure 3.7 | Normalized Load-Deflection Curves for Crushing Tests of FRP Tubes | 46 |
| Figure 3.8a | Numbering System of the 3x3 FRP Deck System | 47 |
| Figure 3.8b | Numbering System of the 4x4 FRP Deck System | 48 |
| Figure 3.9 | Tubular Load Cell for Prestressing Rod | 49 |
Figure 3.10  Wire Potentiometers  
Figure 3.11  Stroke Potentiometers  
Figure 3.12  Typical Test Setup for Specimens at the UC  
Figure 3.13  Typical Test Setup for Single Span Deck (Specimen B3S4)  
Figure 3.14  Typical Test Setup for Double Span Deck (Specimen U3D4)  
Figure 3.15  Longitudinal Crack Adjacent to Loading Plate (Specimen U4S2)  
Figure 3.16  Failure Mode at the Support (Specimen U4S2)  
Figure 3.17  Slippage between Adjacent Tubes (Specimen U3S4)  
Figure 3.18  Failure at the Corner and Middle of Top Flange (Specimen U3S4)  
Figure 3.19  Permanent Deformations in Center Tubes (Specimen U3S4)  
Figure 3.20  Failure Mode of Specimen U3S2.5  
Figure 3.21  Curling of Exterior Tubes at the Support (Specimen U3S2.5)  
Figure 3.22  Failure of Specimen B3S4  
Figure 3.23  Failure of Specimen U4D2 at the End Support  
Figure 3.24  Deformation of Center Tubes in Specimen U3D4  
Figure 3.25  Failure at the Bottom Surface of Tubes at Center Support (Specimen U3D4)  
Figure 3.26  Load-Deflection Curves for Specimen U3S4 at NCSU  
Figure 3.27  Load-Deflection Ratios for Specimen U3S4 at NCSU  
Figure 3.28  Load-Strains for Specimen U3S4 at UC  
Figure 3.29  Load-Strain Ratios for Specimen U3S4 at UC  
Figure 3.30  Load-Support Displacement for Specimen U3S4 at NCSU  
Figure 3.31  Load Versus Prestress for Specimen U3S4 at NCSU  
Figure 3.32  Load-Deflection Curves for Specimen U4S2  
Figure 3.33  Load-Strains for Specimen U4S2  
Figure 3.34  Load Load-Strain Ratios for Specimen U4S2  
Figure 3.35  Load-Deflection Curves for Specimen U3S2.5  
Figure 3.36  Load-Strains for Specimen U3S2.5  
Figure 3.37  Load-Strain Ratios for Specimen U3S2.5  
Figure 3.38  Load-Deflection Curves for Specimen U4S5  
Figure 3.39  Load-Strains for Specimen U4S5
| Figure 4.17 | Deflection Contours for Specimen U3S4 | 107 |
| Figure 4.18 | Comparison of Finite Element Results with Experiments for Specimen U3S4 | 107 |
| Figure 4.19 | Comparison of Finite Element Load-Strain Results with Experiments for Specimen U3S4 | 108 |
| Figure 4.20 | Stress Contours at the Top of Specimen U3S4 | 108 |
| Figure 4.21 | Failure of Specimen U3S4 Observed During the Tests | 109 |
| Figure 4.22 | Tyler Road Bridge Before Deck Replacement | 109 |
| Figure 4.23 | Bottom View of the Floor System Before Deck Replacement | 110 |
| Figure 4.24 | Tyler Road Bridge with FRP Deck | 110 |
| Figure 4.25 | Truck Load Testing of Tyler Road Bridge | 111 |
| Figure 4.26 | Details of the Truss in Tyler Road Bridge | 111 |
| Figure 4.27 | Details of the Floor System in Tyler Road Bridge | 112 |
| Figure 4.28 | Finite Element Model of the Tyler Road Bridge | 112 |
| Figure 4.29 | Live Load Cases for Bridge Rating | 113 |
CHAPTER 1 INTRODUCTION

1.1 Statement of the Problem

1.1.1 Background

Highway bridges are plagued with two major problems; premature deterioration and structural deficiency. At the national level, over 40% of the bridges are classified as structurally deficient or functionally obsolete [“The Status” 1993]. Even newer bridges have shown a growing rate of premature decay [Dunker et al. 1987]. Corrosion protection measures such as epoxy coating of steel reinforcement have not been successful over long periods of time and in severe environments.

A major effort is now underway to rebuild the nation's infrastructure. In order to simply maintain the current conditions (without any improvements), an average annual cost of $5.2 billion is needed through the year 2011 [“The Status” 1993]. Hence, it is vital to the U.S. economy and that of each individual state that cost-effective structural systems and materials be investigated in order to extend service life and to improve performance of bridges.

In recent years, fiber reinforced polymer (FRP) decks have emerged as a potential solution to the problems associated with the conventional bridge decks [Tarricone 1993]. The main characteristics of FRP decks that make them a competitive alternative include:

- Excellent resistance to electro-chemical corrosion (due to, for example, road salt), and hence a longer useful life with less maintenance cost.
- Light weight and excellent strength-to-weight and stiffness-to-weight characteristics make their construction easy and fast with minimal manpower and
equipment. Furthermore, their low dead weight allows for an increase in the live load carrying capacity of the bridge.

- Versatility of fabrication that could be tailored and optimized for geometry, strength, stiffness, and durability characteristics of a particular bridge.

1.1.2 Description of the Project

An existing steel truss highway bridge in the State of Ohio required a deck replacement. The bridge is on Tyler Road (County Road 175) over Bokes Creek in Delaware County, Ohio. The bridge is 113 ft long, and 19 ft 4 in wide. The superstructure consisted of two steel trusses with hangers spaced at 20 ft 10 in on center. The bottom chord was a channel section 12 in wide and 3 in to 3¾ in deep. Preliminary evaluations by the Ohio Department of Transportation and the Delaware County had indicated that the bottom chord may need strengthening. The floor system consisted of a 4 in wooden deck made of 3 in x 4 in lumber. The deck rested on 12 slightly different steel stringers, with 10 in to 10 1/8 in depths, 4¾ in to 5¼ in flange widths and 5/16 in to 3/8 in flange thicknesses. The stringers were placed on 6 floor beams with 17 in depth, 10½ in flange width and 1 in flange thickness. The creek is generally shallow and is about 11 ft below the top of the deck. Figures 1.1-1.4 show photographs of the bridge before deck replacement. The County decided to dismantle the existing trusses, stringers, and floor beams, and re-assemble the bridge after galvanizing and re-fitting them. The County also opted to use a new pretressed FRP tubular deck system, as will be discussed below.

The new FRP system is an off-spring of a post-tensioned concrete-filled FRP tubular system that was developed for another bridge in Ohio. Due to low traffic on the
Tyler Road Bridge, the manufacturer proposed eliminating the concrete from all but the last few tubes at each end of the bridge. This would allow for a much lighter bridge deck with lower construction time and cost.

The 4 in x 4 in pultruded FRP square tubes were pre-drilled and delivered to the site. The holes were about ¾ in diameter at the mid-point between the two stringers. They were then laid side by side on the stringers, transverse to the direction of traffic. The tubes were attached to (and leveled on) the stringers using a clip angle at every 12 in on center. Using 0.6 in diameter seven-wire strands through the holes in the tubes, the entire deck was post-tensioned. The prestress level was designed at 20 kips in each strand. Figures 1.5-1.8 show the photographs of the bridge after deck replacement. The research project with the University of Cincinnati consisted of several components, as follows:
1. Load testing of the bridge before and after deck replacement;
2. Two-year remote monitoring of the bridge under ambient traffic;
3. Testing of FRP materials for different durability conditions;
4. Load rating of the steel trusses;
5. Load rating of the floor system;
6. Testing of FRP tubes and FRP deck under static and fatigue loading; and
7. Finite element modeling of the FRP deck and the bridge floor system.

The North Carolina State University took on the research components outlined in Items 5-7 above.
1.2 Research Objectives

The objectives of the project at the North Carolina State University were as follows:

1. Evaluate the feasibility, behavior, and effectiveness of the new FRP deck system under static and fatigue loading; and
2. Provide analytical data in support of the load rating of the new floor system with the FRP deck.

1.3 Research Approach

In order to achieve the above objectives, a work plan was developed, as follows:

1. Data gathering included review of literature on existing FRP deck systems and their differences, information on the existing bridge and the changes in the bridge layout, methods of bridge load testing and load rating, and finite element analysis of FRP deck systems and bridge superstructures.

2. Experimental work consisted of the following components:
   a. Crushing tests of FRP tubes to establish the squash load for the tubes laid sideways;
   b. Flexural static tests of FRP decks to assess their flexural behavior and to determine the affecting parameters; and
   c. Flexural fatigue tests of FRP decks to establish their endurance limit under simulated traffic loading.

3. Analytical work consisted of the following components:
a. Finite element modeling of single and multiple tubes in crushing tests to develop appropriate modeling techniques for FRP;
b. Finite element modeling of bonded and unbonded FRP deck system in flexure; and
c. Load rating of the floor system, consisting of the FRP deck, stringers and the floor beams.

1.4 Structure of the Thesis

This thesis consists of a total of five chapters. Chapter 1 (this chapter) highlights the problem statement, research objectives and the research approach. Chapter 2 presents a literature review of the existing FRP bridge decks. Chapter 3 summarizes the experimental work on the crushing tests of FRP tubes, and static and fatigue loading of FRP tubular decks in flexure. Chapter 4 presents the analytical work using the finite element modeling for the FRP tubes and the FRP deck system. The finite element analysis is also used for the load rating of the floor system of the Tyler Road Bridge with the new FRP deck. Finally, Chapter 5 summarizes the research, and provides some conclusions in addition to recommendations for future research on the subject.
CHAPTER 2 LITERATURE REVIEW

The use of FRP bridge decks has been a direct result of technology transfer initiatives taken by the defense industry in late 1980’s and early 1990’s. In fact, many of the manufacturers of FRP bridge decks were directly or indirectly related to the aerospace composites industry. Table 2.1 provides a list of some of the FRP bridge decks that have been installed in the last decade in the U.S. It should be noted that the list is by no means comprehensive.

Table 2.1 Some of the FRP Bridge Decks Built in the U.S.

<table>
<thead>
<tr>
<th>No.</th>
<th>Bridge Location</th>
<th>Span Length</th>
<th>State</th>
<th>Year Built</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Lockheed Martin in California</td>
<td>30 ft</td>
<td>California</td>
<td>1995</td>
</tr>
<tr>
<td>2</td>
<td>No-Name Creek Bridge, Russell County</td>
<td>23 ft</td>
<td>Kansas</td>
<td>1996</td>
</tr>
<tr>
<td>3</td>
<td>University of California, San Diego</td>
<td>15 ft</td>
<td>California</td>
<td>1996</td>
</tr>
<tr>
<td>4</td>
<td>Laurel Lick Bridge, Lewis County</td>
<td>20 ft</td>
<td>West Virginia</td>
<td>1997</td>
</tr>
<tr>
<td>5</td>
<td>Wickwire Run Bridge, Taylor County</td>
<td>30 ft</td>
<td>West Virginia</td>
<td>1997</td>
</tr>
<tr>
<td>6</td>
<td>Idaho Falls</td>
<td>30 ft</td>
<td>Idaho</td>
<td>1997</td>
</tr>
<tr>
<td>7</td>
<td>Tech 21 Bridge, Butler County</td>
<td>33 ft</td>
<td>Ohio</td>
<td>1997</td>
</tr>
<tr>
<td>8</td>
<td>State Route 27, Drake County</td>
<td>50 ft</td>
<td>Ohio</td>
<td>1999</td>
</tr>
<tr>
<td>9</td>
<td>State Route 418 Over Schroon River</td>
<td>160 ft</td>
<td>New York</td>
<td>2000</td>
</tr>
<tr>
<td>10</td>
<td>Kings Stormwater Channel Bridge</td>
<td>33 ft</td>
<td>California</td>
<td>2000</td>
</tr>
<tr>
<td>11</td>
<td>53rd Avenue Bridge, Bettendorf County</td>
<td>47 ft</td>
<td>Iowa</td>
<td>2001</td>
</tr>
<tr>
<td>12</td>
<td>Lewis and Clark Bridge, Clatsop County</td>
<td>124 ft</td>
<td>Oregon</td>
<td>2001</td>
</tr>
<tr>
<td>13</td>
<td>State Route 24, Hartford County</td>
<td>128 ft</td>
<td>Maryland</td>
<td>2001</td>
</tr>
<tr>
<td>14</td>
<td>State Route 655</td>
<td>60 ft</td>
<td>South Carolina</td>
<td>2001</td>
</tr>
<tr>
<td>15</td>
<td>State Route 1627, Union County</td>
<td>42 ft</td>
<td>North Carolina</td>
<td>2001</td>
</tr>
<tr>
<td>16</td>
<td>County Road 46, Lewis County</td>
<td>36 ft</td>
<td>New York</td>
<td>2001</td>
</tr>
<tr>
<td>17</td>
<td>State Route 4012</td>
<td>42 ft</td>
<td>Pennsylvania</td>
<td>2001</td>
</tr>
<tr>
<td>18</td>
<td>South Fayette Street</td>
<td>21 ft</td>
<td>Illinois</td>
<td>2001</td>
</tr>
<tr>
<td>19</td>
<td>Schuyler Heim Lift Bridge, Long Beach</td>
<td>36 ft</td>
<td>California</td>
<td>2002</td>
</tr>
</tbody>
</table>
As obvious from the table, there is an increase in the number of FRP bridge decks in the recent few years, as more design guidelines are being developed for FRP systems, and as more states see the benefits of FRP materials.

In this chapter, a summary of some of the FRP bridge deck systems is presented, so that they could be compared with the new deck system that is the subject of this thesis. Most of the information below is taken from the CD-ROM provided by the Market Development Alliance, an entity of the composites industry \cite{MDA2000}.

3TEX deck system is proposed as an alternative for replacement of corrugated metal decks on secondary bridges with closely spaced stringers no more than 3 ft apart. The deck system is called TYCOR. As shown in Figure 2.1, it is a low-profile sandwich deck with a fiber reinforced foam core. The foam is reinforced in the vertical direction with fiberglass roving, sandwiched between two fiberglass skins. The system has been used on two bridges in Ohio in 2001 and 2002, with girder spacing of 2 ft and 2 ft 8 in, respectively.

Martin Marietta Composites has installed a number of bridges across the country (Nos. 6-19 in Table 2.1). Their deck, called DuraSpan, as shown in Figure 2.2, consists of different pultruded fiberglass elements bonded together into panels. The elements are made of E-glass tows stitched into multiple structural fabrics combined with isopolyester resin. The pultrusion process involves wetting the fibers with the resin, and then pulling them at a controlled temperature and speed through a heated metal die with the desired cross section. The elements are assembled into panels with a polyurethane adhesive. Figure 2.3 shows the transport and installation of DuraSpan bridge deck in Drake County, Ohio (No. 8 in Table 2.1).
Superdeck by Creative Pultrusions, which weighs only 20% of a concrete deck, consists of a double trapezoid element and a hexagonal element, both pultruded individually and then bonded together to form the bridge deck (Figure 2.4). Superdeck complies with the HS25 loading [AASHTO 1999] with girder spacing as far apart as 6 ft. It also has a high fatigue resistance and good environmental durability. Figure 2.5 shows the bonding and assembling of the deck on site for the Shawnee Creek Bridge in Ohio. Superdeck has been installed on five bridges, two in West Virginia (No.4 and 5 in Table 2.1), two in Ohio, and one in Pennsylvania.

Hardcore Composites uses a different fabrication technique to manufacture its FRP deck. The method is called vacuum assisted resin transfer molding (VARTM), which involves molding the deck, placing the fabric, and using the vacuum to transfer the resin to the mold. The deck consists of a honeycomb core sandwiched between top and bottom skins (Figure 2.6). Hardcore has installed its decks on nine bridges, four in Delaware, three in New York, one in Maryland, and one in Ohio. The deck is capable of carrying HS25 loading with deflections less than 1/800 of the span length [AASHTO 1999].

Kansas Structural Composites has introduced another type of honeycomb deck (No. 2 in Table 2.1) with a sandwich cross section, as shown in Figure 2.7. The deck is capable of supporting the HS25 load [AASHTO 1999]. The deck is fabricated through hand lay-up techniques. The core is fabricated on molds, while the faces are laid up and the panels are assembled on a flat platen. The honeycomb core laminates consist of multiple layers of chopped strand mat and 40% polyester resin by weight, producing a core web thickness of approximately 0.090 in.
Fiber Reinforced Systems (FRS) of Columbus, Ohio introduced another deck system, called TechDeck, which utilizes pultruded FRP tubes in conjunction with concrete. The system consists of a series of 6 in x 9 in x 5/16 in pultruded fiberglass tubes filled with concrete and then prestressed to form a bridge deck. Figure 2.8 shows the tubes being installed on the bridge. The tubes not only act as pour form for concrete, but also prevent cracking and deterioration of concrete, and lend added stiffness to the bridge deck. Once the individual tubes are lifted into place, they are post-tensioned together to become a single unit, thus eliminating all field joints. The tubes rest on epoxy-coated slab bolsters on top of the existing stringers. Epoxy-coated, low-relaxation steel strands are used for post-tensioning the deck. One such bridge was installed in 2000 in Columbus, Ohio for the Ohio Department of Transportation. As described in Chapter 1, the manufacturer of TechDeck decided to use the post-tensioned FRP tubes, but without the concrete in-fill, for the Tyler Road Bridge in Delaware County, Ohio.
Figure 2.1 TYCOR Deck with Fiber Reinforced Foam Core and Stitched Fabric Skin [MDA 2000]

Figure 2.2 Schematics of DuraSpan Deck Panel [MDA 2000]
Figure 2.3 Transport and Installation of DuraSpan Bridge in Drake County, Ohio \textit{[MDA 2000]}

Figure 2.4 Cross Section of Superdeck System \textit{[MDA 2000]}
Figure 2.5 Bonding and Assembling Superdeck in Xenia, Ohio [MDA 2000]

Figure 2.6 Hardcore Honeycomb Core Deck [MDA 2000]
Figure 2.7 Kansas Honeycomb Deck System
[MDA 2000]

Figure 2.8 TechDeck System[FRS 2000]
CHAPTER 3 EXPERIMENTAL WORK

3.1 Test Plan

A detailed experimental program was carried out with the following three components to evaluate the behavior of tubular FRP bridge deck systems:

1. Crushing tests of FRP tubes: Seven specimens made of single or multiple FRP tubes of two different sizes, and with or without interface bonding were prepared and subjected to pure compression on their sides. The objective of these tests was to establish the squash load for the tubes laid sideways.

2. Flexural static tests of FRP decks: Eleven FRP deck panels with two different tube sizes, with or without interface bonding, and different levels of post-tensioning were tested in static flexural loading with single or dual spans of different lengths. The objective of these tests was to assess the flexural behavior of the FRP decks and to determine the affecting parameters.

3. Flexural fatigue tests of FRP decks: Four dual span FRP decks with two different tube sizes and three different span lengths were subjected to high cycle fatigue loading. The objective of these tests was to establish the endurance limit of the FRP decks under simulated traffic loading.

3.2 Crushing Tests of FRP Tubes

3.2.1 Specimen Preparation

Seven FRP tubular specimens were prepared for crushing test on their sides. Test parameters included: tube size, interface bond, and number of tubes under load. Table 3.1
shows the specimen test matrix for the crushing tests. The specimen name reflects the tube size (3x3 or 4x4), number of tubes, and whether the tubes are bonded or simply laid side by side. The table also shows tube size, number of tubes in each specimen, interface bond or lack of it, and rate of loading. Rate of loading and its effects will be discussed in more detail in Section 3.2.2.

The FRP tubes were identified as standard structural shapes of EXTERN 525 Series manufactured by STRONGWELL. The tubes were fabricated in a pultrusion process by which the impregnated fibers were pulled through a die to form the section. Table 3.2 shows the mechanical properties of the tubes, as reported by the manufacturer. The lengthwise and crosswise directions mean parallel and transverse to the direction of the pultrusion, respectively. Also reported by the manufacturer, are the shear modulus and the Poisson’s ratio of the FRP tubes as 425 ksi and 0.33, respectively.

The specimens with single tube or multiple unbonded tubes were cut at 20 in lengths from 10 ft and 20 ft sections of the FRP tubes using an electric saw. The bonded specimens were cut at 18 in lengths from the un-cracked portion of the bonded FRP deck specimens. The bonding procedure and the bonding agent are described in Section 3.3.

3.2.2 Test Setup, Instrumentation and Test Procedure

Test specimens were placed between two 10 in x 20 in steel plates with 1 in thickness. The dimensions of the plates were selected as the footprint of the wheel load identified by AASHTO [AASHTO. 1999]. Testing was carried out in the 220-kip FORNEY compression machine (Figure 3.1). The load was monitored using a 200 kip or a 50 kip load cell, depending on the type of specimen and the amount of load applied.
Both load cells were calibrated within one year from the test date. Three linear variable differential transformers (LVDT) were used to measure the deflections of the top and bottom steel plates. Data was recorded using a computer controlled high speed Megadac data acquisition system. Tests were carried out in displacement mode, however, with manual control. The rate of loading for each specimen was reported in Table 3.1, as the average rate determined later from the recorded data. Generally, variation of the loading rate did not appear to have an adverse effect on the performance of the FRP specimens.

3.2.3 Test Observations and Failure Modes

Cracking of the single tube specimens could be heard at about 50% of their capacity. However, these cracks were not visible. Visible cracks were initially formed at the corners of the tubes due to high stress concentration. The cracks were often followed by web buckling. As the load dropped, cracks continued to propagate along the entire length of the tube. The tube deformations would also continue to increase, until a secondary crack developed in the web and the specimen failed as a mechanism.

Three modes of failure were observed. Figure 3.2 shows a corner crack at the along the tube length. As the web deforms laterally, two types of cracks may develop: a flexural crack due to excessive bending and buckling at mid-height (Figure 3.3) and an oblique shear crack near the flanges (Figure 3.4). The 3x3 and 4x4 tubes behaved similarly. However, web buckling was more apparent in the 4x4 tubes (Figure 3.5).

Specimens with multiple bonded or unbonded tubes generally performed better than the single tubes, because the webs of the interior tubes were confined against lateral buckling. Failure in multiple unbonded tubes also initiated at the corner of one of the
interior tubes. Web buckling typically occurred after the peak load was reached. Due to
their fabrication process, FRP tubes are not homogeneous. Therefore, they fail at their
weakest location, which may be different in each tube. As the web of an interior tube
begins to buckle outward, it forces the exterior tube away. This further reduces the share
of the load the exterior tubes bear; hence they survive longer than the interior tubes. The
friction between the steel plates and the tubes would eventually arrest the movement of
the exterior tubes, and helped confine the web buckling of the interior tube. In these cases,
often an oblique shear crack would develop in the web of an interior tube.

Interface bond between the sides of the bonded specimens greatly enhanced their
performance. Failure in these specimens was due to the corner cracks, which would
eventually propagate along the length of the tubes. The web typically failed in oblique
shear rather than flexural cracking. For the bonded specimen with three 3x3 tubes, the
outside webs of the exterior tubes also buckled. In all specimens with multiple tubes,
particularly the bonded ones, redundancy allowed a rather large post-peak response.

3.2.4 Test Results and Discussions

Table 3.3 summarizes test results for the crushing specimens. Figure 3.6 shows
the load-deflection curves for all specimens. The initial loading is linear elastic, with a
gradual descending branch after the peak load, and the load stabilizing at some level. For
example, the load stabilizes at about 70% of the peak load in the single 3x3 tube. The
deflection corresponding to the peak load is about the same for all specimens, since it is
controlled by the web buckling. The 3x3 tubes show slightly larger deflection at the peak
load, as compared to the 4x4 tubes. This may be attributed to the smaller buckling length
of the web in the 3x3 tubes. The same argument can be made regarding bonded specimens in comparison with the unbonded ones, as the web of the interior tubes are joined together. In general, specimens with multiple tubes, whether bonded or not, show a larger stiffness in comparison with the single tubes. Rate of loading does not appear to have an effect on the performance of the tubes.

In order to better compare the behavior of the crushed specimens, their responses are normalized by dividing the load by the number of tubes. Figure 3.7 shows that the average strength drops down, as more bonded or unbonded tubes are laid side by side. This may be attributed to the fact that the exterior webs are not fully compressed, and therefore, do not carry the same level of load. The three 3x3 bonded tubes was an exception, since they were all under the loading plates.

3.3 Flexural Static Tests

3.3.1 Specimen Preparation

A total of eleven specimens were tested to study the behavior of FRP tubular deck system under static load. Test parameters included tube size, interface bond, span length, number of spans, and rate of loading. Table 3.4 shows the test matrix for the deck specimens. Specimen name consists of four identifying characters. The first character is either $U$ or $B$ for unbonded or bonded specimens, respectively. The second character denotes the tube size, which is either 3 for 3x3 tubes or 4 for 4x4 tubes. The third character is either $S$ for single span or $D$ for double span configurations. Finally, the last character indicates the span length in ft for each span. For example, $B3S2.5$ is a bonded 3x3 single span with 2.5 ft span length. Two of the single span specimens ($B3S4, B3S2.5$)
were bonded with epoxy, and then post-tensioned prior to the test. All other single span specimens and all double span specimens were only post-tensioned.

All specimens were 3 ft wide, hence consisting of either twelve 3x3 or nine 4x4 tubes. The first four specimens were tested at the University of Cincinnati (UC), whereas all others were tested at the NC State University (NCSU). Two of the tests at UC (U3S4 and U4S2) were repeated at the NCSU on identical specimens and under the same conditions, in order to ensure consistent data and for calibration purposes.

All specimens were cut 6 in longer than the span length to allow for 3 in overhang on either side. At the mid-height of each tube, ¾ in and 1 in holes were drilled along the support lines and the middle of each span, respectively. This was to accommodate ½ in and 5/8 in threaded rods along the support lines and the middle of each span, respectively. Tolerance for the hole sizes was selected to avoid the rod to be in direct contact with the tube, so that no shear transfer could occur. Test observations and results, however, indicate that shear transfer was indeed present in some of the specimens at the later stages of loading. The rods were ASTM 193 Grade B-7 high strength alloy steel with coarse threads. The prestress level at the middle of each span was at least twice that of the prestress along the support line. Along the support lines, 3x3 or 4x4 steel plates with ½ in thickness were used to transfer the prestressing force to the end tubes. At the mid-span, a 17 in long section of a 4x4 tube filled with concrete was used to distribute the prestressing force and help avoid local failure of the webs in the end tubes.

In addition to prestressing, two of the deck specimens were bonded with epoxy to help improve the panel action of the deck. The tubes were first cleaned and aligned with the threaded rods that were wrapped with wax paper. A two-part structural epoxy paste
adhesive (KEMKO 040 DOWEL) from Chemco Systems was used to bond the tubes. Mixing was done with a volume ratio of 1:1 or a weight ratio of 1:1.4, using an electric mixer at a low speed of 350-370 rpm. Since the resin and the hardener were of two different colors, black and white, mixing was carried out for 2-3 minutes until a consistency of bluish gray color was achieved. The paste was then applied onto the FRP tubes with brush, within the 30 minute pot life of the epoxy. After all tubes were bonded, a small amount of prestressing was applied to hold them together. This resulted in the excess epoxy to flow out from in between the tubes. The specimen cured at a temperature of 70°F for at least 7 days prior to loading.

3.3.2 Instrumentation

The primary interests in the instrumentation of the tubular deck system for static flexural tests were twofold: (a) stiffness of the system, and (b) panel action. Therefore, a detailed instrumentation plan was adopted, as outlined below.

Electrical resistance strain gages with 1¼ in gage length (TML PFL-30-3L by Tokyo Sokki Kenkyujo Co.) were used to measure surface strains in the tubes. The gages were bonded using Devcon 5-minute epoxy gel, after surface preparation of the tubes with sandpaper, degreaser, acid, and neutralizer. The tubes were numbered from 1 to 12 for the 3x3 tubes, and from 1 to 9 for the 4x4 tubes. Strain gages were bonded to the top and bottom surfaces of tubes No. 1, 4, and 6 for the 3x3 tubes, and tubes No. 1, 3, and 5 for the 4x4 tubes. Figure 3.8a&b show the locations of strain gages.

Two 4 in long, 1.2 in diameter, 0.25 in thick hollow steel tubes were used as load cells to measure the prestress force (Figure 3.9). Each load cell was instrumented with
four 1/2 in long strain gages (CEA-06-250UW-120 by Measurements Group), connected as a full Wheatstone bridge. The load cells were calibrated in the FORNEY compression testing machine for up to 20 kip axial load. Since only two load cells were made, the number of turns of the nut was used for those rods without the load cell. The load cells, however, were specifically placed at the mid-spans to ensure the level of prestress is accurately known at these locations. Technical properties of the strain gages for the surface gages and the gages on the load cell are presented in Table 3.5.

Two types of instruments were used to measure mid-span or support displacements: cable-extension transducers (wire potentiometers) with a 15 in stroke (PT-100 Series by Celesco Transducer Products, Inc.) (Figure 3.10), and potentiometers with a 2 in maximum stroke (BEI Duncan Electronics Division Sensors & Systems company) (Figure 3.11).

Load and stroke were directly measured from the output of the controller of the actuator. Those readings along with the readings from the transducers and the strain gages were recorded using a computer controlled high speed Megadac data acquisition system.

3.3.3 Test Setup and Procedure

Test setup consisted of an actuator, test frame, support system, and loading system. The actuator at the UC was a 350 kip Sheffer actuator with an MTS Testar controller system. Two MTS actuators with 250 kip and 450 kip capacity were used for the static flexural tests of the decks at the NCSU. Test frame at the UC was a self-reacting frame (Figure 3.12), whereas the frame at the NCSU was tied to a 2½ ft strong
floor with a 100 kip pretension force at each of the supporting columns (Figures 3.13 and 3.14).

A 10 in x 20 in x ½ in steel plate resting on a high density neoprene pad was used as the footprint of the load at the middle of each span. The longer side of the plate was in the direction of the tubes, simulating the width of the truck wheels or tiers in the transverse direction of the bridge. A spreader beam was used for the double span decks. One or two 10½ in x 10½ in x ½ in thick hollow rectangular steel blocks were used to transfer the load from the actuator or the spreader beam to the steel plates.

Two different bearing conditions were used in this study. The support system at the UC consisted of 2 in steel rollers in between two channel sections. Neoprene pads were used at the NCSU for support bearings. The pads were placed on 3½ ft long 6 in x 6 in square tubes with 0.5 in wall thickness. The steel tubes were supported laterally by two steel angles to restrain their outward movement. Results showed no significant difference in the behavior of the system under the two different bearing conditions.

The FRP tubes were aligned, instrumented and prestressed, before applying transverse loads. The first specimen at the UC, Specimen U3S2.5, was prestressed only along the support lines. Premature local failure of the web of the end tube, and lack of any panel action, indicated the need to prestress at the middle of each span. Therefore, all other decks were prestressed at the middle of each span as well as along the support lines. The level of prestress at the mid-span was intentionally greater than that along the support lines. Table 3.6 shows the level of prestress for each of the deck specimens. All tests were carried out in a displacement control at the rate shown in Table 3.4. The loading rate did not seem to adversely affect the performance of the FRP specimens.
3.3.5 Test Observations and Failure Modes

The FRP tubular deck system consists of a number of bonded or unbonded tubes connected with prestressing rods. The system has considerable redundancy built into it. The friction that develops between the tubes as a result of prestressing or the interface bond may not be sufficient at high load levels. Initially, the threaded rods are not in contact with the tubes, because of the larger hole sizes. However, with the loss of bond between the adjacent tubes, at later stage of loading the rods may come in direct contact with the tubes, and bear some of the applied loads in flexure and direct shear. Failure of a system is often distinguished by its catastrophic collapse or a significant load drop. Park and Paulay [Park R. and Paulay T., 1973], for example, suggest ultimate failure of concrete structures after 15% load drop. However, what constituted “failure” for each of the deck specimens was qualified as the end of their useful function. The tests were stopped after significant cracking of the tubes, when the load path was transferred to the rods, bending them considerably, since this was not an intended load path for the system.

Failure of a single span unbonded deck is somewhat similar to that observed in the crushing tests. Longitudinal cracks typically initiated at the corner of the tube most adjacent to the edge of the loading plate (Figure 3.15), often tube No. 5 or 8 for the 3x3 tubes, and tube No. 4 or 6 for the 4x4 tubes. While deformations were largest at the mid-span, the end tubes curled up at the supports. Upon formation of initial cracks, the decks failed in several different modes. In Specimen U4S2 with the shortest span of 2 ft, shear cracks developed at the bottom corners on the supports. This was followed by the buckling of bottom flanges. Meanwhile, the first longitudinal crack at the top propagated towards the supports, without much deflection. With the buckling of the bottom flange at
the support, the load began to fluctuate, and the test was stopped (Figure 3.16). After the test no crack was found at the bottom face of the tubes in the mid-span, or in the webs of the tubes.

Specimen U3S4, with a much longer span, behaved quite differently. As the load increased, considerable slippage developed between the center tubes and their adjacent tubes (Figure 3.17). After the first corner cracks, top flange of one of the tubes also buckled and failed in the middle (Figure 3.18). Test was stopped when deflections became large, and the tube began bearing on threaded rods. At the support, a failure mode similar to Specimen U4S2 was observed at tube No. 6 tube, but with less severity. Deformations of the center tubes were present even after removing the load (Figure 3.19). Additional cracks were noted at the bottom corners of the center tubes at the mid-span.

Specimen U3S2.5, with no pre-stress at mid-span, showed no panel action, as the loading plate punched through top the deck (Figure 3.20). No distress was observed at the supports, except some noticeable curling of the exterior tubes (Figure 3.21).

Both bonded specimens failed at much lower loads. Cracking sound could be heard even while prestressing the threaded rods. This indicated some fracture in the epoxy bond. The tubes, however, deformed together indicating a good panel action. The panel action was also evident from curling up of the end tubes. The system had significant elastic energy capacity, with a rather sudden and potentially violent rupture. In Specimen B3S4, the test was stopped shortly after load dropped due to the longitudinal cracking and rupture at the top corner of the tube No. 5 (Figure 3.22). Failure of Specimen B3S2.5 was slightly different, in that the epoxy-bonded interface between tubes No. 4 and 5 ruptured with a huge sound. Relative displacement was observed at the
sliding surfaces. The test was stopped when similar longitudinal crack was observed in tube No. 8.

The general behavior of the three double span specimens was similar to their single span counterparts. In longer span decks, slippage between the center tubes and their adjacent tubes was more apparent. On the other hand, curling of the end tubes was more significant in shorter span decks. In Specimen U4D2, much the same as Specimen U4S2, longitudinal crack at the corner of tube No. 4 propagated towards the exterior supports, where the bottom flange of the tube eventually buckled and failed (Figure 3.23). Specimens U3D4 and U4D5 both failed with considerable deformation of center tubes (Figure 3.24). However no significant crack was observed at the end supports. Load was stopped, as it became evident that the tubes were bearing on the threaded rods. Further examination revealed additional cracks at the bottom of tube No. 5 at the center support, where maximum compressive stresses would occur (Figure 3.25).

3.3.4 Test Results and Discussions

Table 3.7 summarizes the results from all deck panels. In the table, the strain ratio is defined as the ratio of strain at any point on the deck to its corresponding maximum strain at the mid-span of the deck. The term “equivalent number of tubes” is defined to represent the number of tubes that can carry the same load, if they were all fully effective and directly under the load. For example, for a deck made up of twelve 3x3 FRP tubes with strain ratios 10%, 20%, 30%, 40%, 80%, and 100%, for tubes No. 1-6, respectively, the total strain ratio is 560%. In other words, the twelve tubes in the deck are actually
equivalent to 5.6 tubes directly loaded and fully effective. Therefore, the larger the “equivalent number of tubes,” the greater panel action is present in the deck.

Specimen U3S4 was tested at the UC. However, another replicate of the same specimen was tested at the NCSU for calibration and comparison. Figure 3.26 shows the load-displacement curves for the specimen at the NCSU. The response is generally linear, with some slope changes at different stage of loading. The displacements measured at the bottom of tubes No. 1, 2, 3 and 4, which are not directly under the loading plate, are much lower than those of tubes No. 5 and 6. The difference, which becomes more significant at higher loads, is attributed to lack of panel action and some slippage at the interface of tubes No. 4 and 5. The same point can be made more clear by examining the displacement ratios in Figure 3.27. Figures 3.28 and 3.29 show the load-strains and the strain ratios of the specimen tested at the UC, respectively. The strain ratios correspond fairly well with the displacement ratios shown in Figure 3.27. While the strain in tube No. 3 is about 60%-70% of that in the center tube (No. 5), the strain in the end tube (No. 1) is only 10%-20% of that in the center tube. At about 50% of the ultimate load, longitudinal cracks are formed and cause the strain in the center tubes to increase dramatically. As such, the strain ratios drop significantly. Figure 3.30 shows the support displacements, where the center tubes (No. 5 and 6) cause much more deformation in the neoprene pad, as compared to the exterior tubes. Also, exterior tubes show some indications of curling up at the corner. Figure 3.31 shows the prestress as a function of the applied loads. Note that the scale in the vertical direction is enlarged, and the total loss is less than 20%. A
comparison of Figures 3.31 and 3.27 shows that the entire loading process can be divided into several stages, as follows:

1. Initially, the deck acts similar to a two way slab, as if the tubes are “bonded.” Therefore, displacements at various locations of the deck are close to each other, and the pre-stress loss is just due to the seating of the system.

2. At about 20% of the capacity, the friction provided by the threaded rods is no longer sufficient to transfer the shear force from the center tubes to their adjacent tubes (No. 5 to No. 4). Therefore, the load share of the center tubes increase and the relative displacements also increase. The pre-stress loss at this stage accelerates as the center tubes begin to bear on the rod and cause slippage in the end anchorage.

3. At about 50% of the capacity, longitudinal cracks appear. The stiffness of the center tubes decrease and the threaded rod begins to recover some of its prestress losses. This is mainly due to the fact that as the tubes crack, the web has a tendency to expand outward, stretching the bar, thereby increasing its stress level.

4. At about 70% of the capacity, the displacement ratios stabilize, as the threaded rod actively participates in the bending of the deck. This increases the panel action by pulling down all of the tubes together. It also develops a pre-stress moment in the lateral direction, and leads to curling of the end deck tubes at the corners.

Specimen U4S2 with a much shorter span, behaved quite differently from the longer span decks. The displacement at the bottom of the deck at its mid-span is only 0.08 in, which is less than 10% of that in Specimen U3S4. The displacement at the top of
the deck is measured at about twice the bottom deflection. Some of the difference is because of the deformation of the neoprene pad under the loading plate. However, the primary cause is the flexure of the top flange of the tubes, which was more evident in 4x4 tubes. Figure 3.32 shows the load-displacements for the specimen. The response is generally linear without any noticeable stiffness degradation. The test was stopped after a significant longitudinal crack at the mid-span extended to the support. Figure 3.33 shows the load-strains for the same specimen and with the same general linear response. The maximum strain captured was at the support in the bottom of tube No. 4, where severe longitudinal cracking was observed. This may be attributed to the stress concentrations at the support. As shown in Figure 3.34, the strain ratios are relatively stable, indicating no severe damage in the center tubes and no significant slippage between the center tubes and their adjacent tubes.

Specimen U3S2.5 is the first specimen tested at the UC, and the only deck without any pre-stress at mid-span. Figures 3.35 and 3.36 show the load-displacements and load-strains for the specimen, respectively. Displacements result from local buckling of top flange as well as overall bending of the deck. However, the bending component is not as significant in the shorter span specimens. Strain ratios are depicted in Figure 3.37. The contribution from the end tubes is almost negligible, since there is no prestressing at mid-span.

Specimen U4S5 had much larger displacements (Figure 3.38), primarily due to its longer span. Since the friction afforded by the prestress was not adequate to transfer the shear, considerable slippage was observed between the center tubes and their adjacent tubes. The cracking of the center tubes and the slippage between the tubes were
accompanied by distinct load drops. Figures 3.39 and 3.40 show the load-strains and the strain ratios, respectively. The increase in strain ratios at the later stages of loading may be attributed to the effect of threaded rods in transferring the shear by direct bearing.

Specimen B3S4 showed greater stiffness than its unbonded counterpart, U3S4. Figures 3.41-3.43 show the load-displacements, load-strains, and strain ratios, respectively. Prior to cracking of the deck at about 15 kips, the strain ratios for tubes No. 1 and 4 are 35% and 70%, respectively, as compared to 15% and 65% in U3S4. Therefore, it appears that the interface bond helps transfer the shear to the end tubes, and improves the panel action. However, in both decks, the first longitudinal crack emerges at about 15 kips due to stress concentration at the corner of center tubes. The test was stopped early to avoid catastrophic failure of the test setup. Otherwise, it was expected that the bonded specimen would fail at a higher load than Specimen U3S4.

Figures 3.44-3.46 show the load-displacements, load-strains, and strain ratios, respectively, for Specimen B3S2.5. The response is generally linear. The interface bond between tubes No. 4 and 5 was broken at a load of about 25 kips. Subsequently, tube No. 4 releases most of its load back to the center tubes. This is evident from the sudden drop of the strain ratio as well as the displacements for tube No. 4. Prior to cracking, the strain ratio for tube No. 4 is about 5 times that of the same tube in the unbonded Specimen U3S2.5. However, the interface bond does not provide much improvement to the end tubes (No. 1 and 12). The displacements too are not much different between the bonded and unbonded specimens. Failure in both cases is dominated by local fracture of the tubes, as overall flexural deformations tend to be less significant in short span decks.
Figures 3.47-3.49 show the load-displacements, load-strains, and strain ratios, respectively, for Specimen U4D2. The response is quite similar to the single span deck specimens, in that it is linear with abrupt changes due to cracking and fracture of the tubes. The maximum load per span of Specimen U4D2 is 10% higher than that of its single span counterpart, Specimen U4S2. However, the maximum strains at failure are not much different. There is significant stress concentration in the bottom of the deck at the center support. Theoretically, the reaction at the center support is four times that of the end supports. Therefore, the specimen develops its first major cracking at the center support, and subsequently, the deck acts very closely to its single span counterpart. Examination after the test showed much damage to the neoprene pad at the center support.

Figures 3.50-3.52 show the load-displacements, load-strains, and strain ratios, respectively, for Specimen U3D4. During the test, the load was stopped to ensure stability of the setup, and the specimen was then reloaded. The stiffness is increased when the tubes bear on the threaded rods. In comparison to its single span counterpart, the maximum load per span is 10% higher. However, in terms of displacements and strains, there is not much difference between the two specimens.

Finally, Figures 3.53-3.55 show the load-displacements, load-strains, and strain ratios, respectively, for Specimen U4D5. The general behavior is the same as other deck specimens. In comparison to its single span counterpart, the maximum load per span is 25% higher. The continuity increases the stiffness of the tube, as the displacements are about 25% less than the single span counterpart.
Figure 3.56 compares the comparison of load-displacement curves for all deck specimens. From this figure and the discussions in this section, the following conclusions could be made:

- Span length influences the behavior of the deck the most. It also affects the mode of failure. In short span decks, overall bending of the deck is not significant, as compared to local bucking of the flanges. In long span decks, on the other hand, bending component is more significant.

- Interface bond is improved with the epoxy paste. The effects, however, are not as significant in short span decks, where bending is not the primary source of displacements.

- Prestressing generally improves the panel action. However, the extent of panel action seems to be quite limited in a long bridge.

- Regardless of the hole sizes in the sides of the tubes, the threaded rods seem to always be directly loaded by the tubes at later stages of loading. This may not be desired in a field application, although it may provide additional redundancy for the system.

3.4 Flexural Fatigue Tests

3.4.1 Specimen Preparation

Three unbonded double span specimens were tested in flexure under high cycle fatigue loading. Test parameters included tube size and span length. Table 3.8 shows the specimen test matrix for the fatigue tests. The specimen name follows the same
convention as that of static tests. Two replicates of Specimen U4D2 were tested to verify its short fatigue life.

The maximum load represents service live load that would be applied on the deck. The live load moment ($LLM$) is calculated from AASHTO [AASHTO 1999], as given by

$$LLM = \frac{P(S + 2)}{32}$$

where $LLM$ is the live load moment as ft-lb per linear ft width of the deck, $P$ is equal to 16 kips as the weight of the rear axle of an HS20 truck (Figure 3.57), and $S$ is the span length of the deck in the direction perpendicular to the traffic. The impact factor for the transverse span of the deck is always the maximum prescribed limit of 30%. The AASHTO [AASHTO 1999] allows a reduction factor 80% for continuity. For example, for Specimen U4D2, the maximum live load moment for the entire 3 ft width of the deck is given by:

$$\text{ftkipMoment} = 24.63 \times 80.0 \times 30.1 \times 32 = 2216$$

(3.2)

On the other hand, the maximum moment for a two span continuous beam occurs at the center support, and is equal to $\frac{3QS}{16}$, where $Q$ is the concentrated load at the middle of each span. Therefore, two concentrated loads of 16.64 kips for a total of 33.3 kips will provide the same maximum service load moment for the deck.

The minimum load was selected to represent the superimposed dead loads such as the wearing surface but primarily for the stability of the test setup. The minimum load represented a stress ratio between 15% and 22% of the maximum load for the different specimens, as shown in Table 3.8. Specimen preparation was carried out the same as that described in the static tests.
3.4.2 Test setup, Instrumentation, and Test Procedure

Figure 3.58 shows the typical setup for fatigue tests. All fatigue tests were carried out at the NCSU, using a 220 kip MTS actuator and the steel frame described earlier. The hydraulic system could pump oil at a rate of 100 gallons per minute with a pressure of 3000 psi. The loading system was the same as the static tests.

The instrumentation plan was also similar to that of the static tests. The primary concerns with the fatigue tests, however, were twofold; (a) stiffness degradation, and (b) degradation of the panel action. Readings from the transducers and the strain gages were recorded using a computer controlled high speed Megadac data acquisition system.

The specimens were tested in load control with a sine wave between the minimum and maximum load levels. A safety threshold was set to force the hydraulic system to shut down, if the stroke exceeded 1¼ in. Prior to automatic fatigue loading, 200-300 cycles were run to monitor the specimen and the test setup. During the continuous fatigue loading, data was collected at regular intervals for about 10 seconds at a sample frequency of 400 scans per second. At each interval, the specimen was visually examined for any sign of distress or cracking. The contact instruments such as potentiometers were also checked to make sure they had not moved. Moreover, the loading frequency and the maximum and minimum loads were verified. Safety checks were carried out at the same time to ensure that vibration of the test frame was not excessive, the hydraulic hoses were not rubbing against each other, and that the oil in the hydraulic system was not overheated. If the specimen did not fail within 1,000,000 cycles of fatigue loading, it was then subjected to a static reserved strength test.
3.4.3 Test Observations and Failure Modes

Specimen U4D2 failed at much lower number of cycles than the other two specimens. In order to verify its fatigue susceptibility, two replicates of the same specimen were tested with the first failing at only 3300 cycles and the second one failing at about 7200 cycles. The center tubes vibrated significantly under the load. However, no clear slippage between the center tubes and their adjacent tubes was observed. The first longitudinal crack emerged at about 1500 cycles at the corner of tube No. 6 on its top surface most adjacent to the loading plate. This too was consistent with the static tests. At about 2500 cycles, a minor crack was noted at both end supports in the bottom corner of tube No. 5. At about 3000 cycles, the crack extended through the corner at one end. At the other end, a new longitudinal crack was developed and the bottom flange buckled. Similar crack was noted in tubes No. 4 and 6 as the test progressed. The test was terminated at about 7200 cycles, when the mid-span crack opened up and extended to the end support. After the test, it was observed that the center tubes (No. 4, 5, and 6) had actually crushed at the center support. Figures 3.59-3.62 show the various cracks and fractures in Specimen U4D2.

Specimen U4D5 lasted much longer under fatigue loading than the first specimen. Figure 3.63 shows the test setup. The test was carried out at a frequency of 2 Hz, and data was initially collected every hour (i.e., 7200 cycles). The deck behaved much the same as its static counterpart in that the center tubes deformed significantly more than the end tubes. A minor crack was observed at the bottom corner of tube No. 5 at the end support just before 10,000 cycles (Figure 3.64). Another minor crack developed in the top corner
of tube No. 4 at the mid-span at about the same time. At about 16,000 cycles, the crack at the end support extended longitudinally towards the mid-span. At about 36,000 cycles, the cracks at the mid-span appeared in both tubes No. 4 and 6. At about 45,000 cycles, a severe cracking was noticed in the bottom of tube No. 6 at the center support (Figure 3.65). However, the cracks at the mid-span and the end supports hardly developed any further. The loading frequency was increased to 3 Hz, while data was still collected every hour (i.e., 10,000 cycles). The data collection interval was increased to 4½ hours (i.e., 50,000 cycles) after 200,000 cycles. At about 580,000 cycles, one of the pre-stressed threaded rods at the mid-span ruptured with a huge sound (Figure 3.66). The test continued for a couple of minutes before the failure was located. The rupture of the pre-stressed rod was believed to be a secondary failure for the deck system, and the test was stopped. An examination of the deck after the test showed no other noticeable crack anywhere in the deck. Afterwards, the specimen was refitted with another threaded rod, and tested in static loading to failure. The test did not show any noticeable degradation in the reserved strength of the deck.

Due to shortage of tubes, and since only the center tubes participate in carrying the majority of the loads, Specimen U3D4 was made with 10 rather than 12 tubes (Figure 3.67). The specimen, however, showed a better panel action than the previous specimens with little or no slippage between the center tubes and their adjacent tubes. The first visible crack was noticed after about 350,000 cycles in the bottom of the center tube at the center support. The first longitudinal crack was noticed at the corner of tube No. 4 at the mid-span after 450,000 cycles (Figure 3.68). No other crack was observed during the test, and the specimen survived 1,000,000 cycles of loading. After the fatigue test, a static
test was carried out in the same setup to determine the reserved strength of the specimen. No significant degradation in the strength was noted. The failure mode too was not any different from that of the static tests of virgin specimens, except for the more significant buckling of tube No. 4 at the mid-span. Figures 3.69-3.71 show the various fractures observed in the reserve strength test of Specimen U3D4.

3.4.4 Test Results and Discussions

Figure 3.72 shows the load versus mid-span deflection of Specimen U4D2 at different cycles of fatigue loading. The response clearly remains linear elastic, without much stiffness degradation. However, it does shift almost 0.17 in after only 7000 load cycles. This shift is 4 times the amount of deformation that the deck goes through in one cycle of loading. The main source for the shift in the displacements is the relative slippage between the center tubes and their adjacent tubes, as the center tubes are pushed down at the maximum load in each cycle. As the load is released from its maximum level, the threaded rods straighten out and force the tubes together. The friction that is provided by the prestressed rods would prevent the center tubes from going back to their original positions. Therefore, the slippage tends to be irreversible. The shift in the load-displacement curve may also be attributed to the creep effect in the deck system, since the deck can be assumed to be under an average load at all times. The shift tends to increase at higher load cycles. There is some stiffness degradation in the order of about 25% after 7000 cycles. Majority of the stiffness degradation occurs near the end. Figure 3.73 shows the strain ratio for the mid-span readings at the bottom of tube No. 3 to that of tube No. 5.
The strain ratio does not seem to be affected by the fatigue loading, as it stays close to 30%, and only near the end of the test drops down to about 25%.

Figures 3.74 and 3.75 show the load-deflections and the strain ratios, respectively, for Specimen U4D5 at different cycles of fatigue loading. The load-deflection curves show a ¼ in shift in mid-span deflection of the center tube and a 10% stiffness degradation after about 500,000 cycles. The displacement shift occurs early in the fatigue loading. Some of the displacement shift may have been caused by creep deformations when the test was paused for the weekend, and the actuator weight rested on the specimen. The graphs of Figure 3.75 represent ratios of mid-span bottom strains in tube No. 3 to those of tube No. 5 at different cycles of fatigue loading. The strain ratios vary between 0.45 and 0.55, however, with no clear trend of change in the panel action.

Figures 3.76 and 3.77 show the load-deflections and the strain ratios, respectively, for Specimen U3D4 at different cycles of fatigue loading. The load-deflection curves show only a 1/16 in shift in mid-span deflection of the center tube and a 15% stiffness degradation after 1,000,000 cycles. Most of the displacement shift still occurs early in the fatigue loading. The overall displacement shift is about ¼ of that observed in Specimen U4D5. The panel action was also more apparent in Specimen U3D4, as hardly any relative slippage was observed. This may be attributed to the shorter width of the specimen, which in turn results in the more concentrated prestressed load at the midspan. The strain ratios, as shown in Figure 3.77, vary between 0.56 and 0.62, however, again with no clear trend of change in the panel action.

Figures 3.78 and 3.79 show the load-deflections and the strain ratios, respectively, for Specimen U3D4 under static reserved strength test. No strength degradation nor any
stiffness degradation was observed in comparison with the virgin specimen. Noting that
the reserved strength specimen contained only 10 tubes, one may conclude that end tubes
hardly participate in carrying the loads. Some nonlinearity in the load-displacements is
observed at about 65% of the capacity. The nonlinearity is attributed to the bending of the
threaded rods. The strain ratios remain quite stable up to about 40%-60% of the capacity
for different tubes. Slight increase in the strain ratios near the ultimate load indicates the
effect of threaded rods in distributing the shear across the deck, effectively forcing a
better panel action.

Table 3.1 Matrix for Crushing Test Specimens

<table>
<thead>
<tr>
<th>No.</th>
<th>Specimen Number</th>
<th>Tube Size (in)</th>
<th>Number of Tubes</th>
<th>Tube Length (in)</th>
<th>Interface Bond</th>
<th>Rate of Loading (in/min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3”x1a</td>
<td>3 x 3 x ¼</td>
<td>1</td>
<td>18</td>
<td>No</td>
<td>0.0002</td>
</tr>
<tr>
<td>2</td>
<td>3”x1b</td>
<td>3 x 3 x ¼</td>
<td>1</td>
<td>20</td>
<td>No</td>
<td>0.0001</td>
</tr>
<tr>
<td>3</td>
<td>4”x1</td>
<td>4 x 4 x ¼</td>
<td>1</td>
<td>20</td>
<td>No</td>
<td>0.0003</td>
</tr>
<tr>
<td>4</td>
<td>4”x3</td>
<td>4 x 4 x ¼</td>
<td>3</td>
<td>20</td>
<td>No</td>
<td>0.00007</td>
</tr>
<tr>
<td>5</td>
<td>3”x3bond</td>
<td>3 x 3 x ¼</td>
<td>3</td>
<td>18</td>
<td>Yes</td>
<td>0.0005</td>
</tr>
<tr>
<td>6</td>
<td>3”x4</td>
<td>3 x 3 x ¼</td>
<td>4</td>
<td>20</td>
<td>No</td>
<td>0.00008</td>
</tr>
<tr>
<td>7</td>
<td>3”x4bond</td>
<td>3 x 3 x ¼</td>
<td>4</td>
<td>20</td>
<td>Yes</td>
<td>0.00009</td>
</tr>
</tbody>
</table>

Table 3.2 Mechanical Properties of FRP Tubes

<table>
<thead>
<tr>
<th>Type of Test</th>
<th>Ultimate Strength (ksi)</th>
<th>Elastic Modulus (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Lengthwise</td>
<td>Crosswise</td>
</tr>
<tr>
<td>Tension</td>
<td>30</td>
<td>7</td>
</tr>
<tr>
<td>Compression</td>
<td>30</td>
<td>15</td>
</tr>
</tbody>
</table>
### Table 3.3 Summary of Crushing Test Results

<table>
<thead>
<tr>
<th>Specimen Name</th>
<th>Total Ultimate Load (kip)</th>
<th>Ultimate Load per Tube (kip)</th>
<th>Stiffness (kip/in)</th>
<th>Displacement (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3&quot;x1a</td>
<td>37.0</td>
<td>37.0</td>
<td>617</td>
<td>0.060</td>
</tr>
<tr>
<td>3&quot;x1b</td>
<td>38.2</td>
<td>38.2</td>
<td>637</td>
<td>0.060</td>
</tr>
<tr>
<td>4&quot;x1</td>
<td>48.2</td>
<td>48.2</td>
<td>1,303</td>
<td>0.038</td>
</tr>
<tr>
<td>4&quot;x3</td>
<td>117</td>
<td>39.0</td>
<td>2,600</td>
<td>0.045</td>
</tr>
<tr>
<td>3&quot;x3bond</td>
<td>152</td>
<td>51.0</td>
<td>2,533</td>
<td>0.060</td>
</tr>
<tr>
<td>3&quot;x4</td>
<td>133</td>
<td>33.2</td>
<td>2,956</td>
<td>0.045</td>
</tr>
<tr>
<td>3&quot;x4bond</td>
<td>155</td>
<td>38.8</td>
<td>2,123</td>
<td>0.073</td>
</tr>
</tbody>
</table>

### Table 3.4 Matrix for Static Test Specimens

<table>
<thead>
<tr>
<th>No.</th>
<th>Specimen Name</th>
<th>Test Location*</th>
<th>Tube Size (in)</th>
<th>Span Type</th>
<th>Span Length (ft)</th>
<th>Interface Bond</th>
<th>Rate of Loading (in/min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>U3S2.5</td>
<td>UC</td>
<td>3 x 3</td>
<td>Single</td>
<td>2.5</td>
<td>No</td>
<td>0.5</td>
</tr>
<tr>
<td>2</td>
<td>U3S4</td>
<td>UC</td>
<td>3 x 3</td>
<td>Single</td>
<td>4</td>
<td>No</td>
<td>0.5</td>
</tr>
<tr>
<td>3</td>
<td>U4S5</td>
<td>UC</td>
<td>4 x 4</td>
<td>Single</td>
<td>5</td>
<td>No</td>
<td>0.5</td>
</tr>
<tr>
<td>4</td>
<td>U4S2</td>
<td>UC</td>
<td>4 x 4</td>
<td>Single</td>
<td>2</td>
<td>No</td>
<td>0.5</td>
</tr>
<tr>
<td>5</td>
<td>B3S2.5</td>
<td>NCSU</td>
<td>3 x 3</td>
<td>Single</td>
<td>2.5</td>
<td>Yes</td>
<td>0.1</td>
</tr>
<tr>
<td>6</td>
<td>B3S4</td>
<td>NCSU</td>
<td>3 x 3</td>
<td>Single</td>
<td>4</td>
<td>Yes</td>
<td>0.25</td>
</tr>
<tr>
<td>7</td>
<td>U3D4</td>
<td>NCSU</td>
<td>3 x 3</td>
<td>Double</td>
<td>4</td>
<td>No</td>
<td>0.1</td>
</tr>
<tr>
<td>8</td>
<td>U4D2</td>
<td>NCSU</td>
<td>4 x 4</td>
<td>Double</td>
<td>2</td>
<td>No</td>
<td>0.1</td>
</tr>
<tr>
<td>9</td>
<td>U4D5</td>
<td>NCSU</td>
<td>4 x 4</td>
<td>Double</td>
<td>5</td>
<td>No</td>
<td>0.1</td>
</tr>
<tr>
<td>10</td>
<td>U3S4**</td>
<td>NCSU</td>
<td>3 x 3</td>
<td>Single</td>
<td>4</td>
<td>No</td>
<td>0.5</td>
</tr>
<tr>
<td>11</td>
<td>U4S2**</td>
<td>NCSU</td>
<td>4 x 4</td>
<td>Single</td>
<td>2</td>
<td>No</td>
<td>0.5</td>
</tr>
</tbody>
</table>

* UC = University of Cincinnati, NCSU = NC State University.
** These specimens were duplicates of the ones tested at the University of Cincinnati.

### Table 3.5 Technical Properties of Electric Resistant Strain Gages

<table>
<thead>
<tr>
<th>Parameters</th>
<th>FRP Tube</th>
<th>Load Cell</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gage Type</td>
<td>PFL-30-11</td>
<td>CEA-06-250UW-120</td>
</tr>
<tr>
<td>Manufacturer</td>
<td>Tokyo Sokki Kenkyujo Co.</td>
<td>Measurements Group, Inc.</td>
</tr>
<tr>
<td>Length</td>
<td>1 ⅛ mm</td>
<td>½ mm</td>
</tr>
<tr>
<td>Electric Resistance</td>
<td>120.4±0.5 Ω</td>
<td>120.0±3.6 Ω</td>
</tr>
<tr>
<td>Gage Factor</td>
<td>2.13±1%</td>
<td>2.065±0.5%</td>
</tr>
<tr>
<td>Temperature Compensation</td>
<td>11E-6/°C</td>
<td>0</td>
</tr>
<tr>
<td>Transverse Sensitivity</td>
<td>-0.5%</td>
<td>(0.4±0.2)%</td>
</tr>
</tbody>
</table>
Table 3.6 Prestress Levels for FRP Deck Specimens

<table>
<thead>
<tr>
<th>No.</th>
<th>Specimen Name</th>
<th>Test Location</th>
<th>Prestress Level at Mid-Span (kips)</th>
<th>Prestress Level at Supports (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>U3S2.5</td>
<td>UC</td>
<td>0</td>
<td>8</td>
</tr>
<tr>
<td>2</td>
<td>U3S4</td>
<td>UC</td>
<td>20</td>
<td>8</td>
</tr>
<tr>
<td>3</td>
<td>U4S5</td>
<td>UC</td>
<td>20</td>
<td>8</td>
</tr>
<tr>
<td>4</td>
<td>U4S2</td>
<td>UC</td>
<td>20</td>
<td>8</td>
</tr>
<tr>
<td>5</td>
<td>B3S2.5</td>
<td>NCSU</td>
<td>8.5</td>
<td>4</td>
</tr>
<tr>
<td>6</td>
<td>B3S4</td>
<td>NCSU</td>
<td>5.5</td>
<td>4</td>
</tr>
<tr>
<td>7</td>
<td>U3D4</td>
<td>NCSU</td>
<td>8</td>
<td>4</td>
</tr>
<tr>
<td>8</td>
<td>U4D2</td>
<td>NCSU</td>
<td>6.6</td>
<td>4</td>
</tr>
<tr>
<td>9</td>
<td>U4D5</td>
<td>NCSU</td>
<td>9.5</td>
<td>4</td>
</tr>
<tr>
<td>10</td>
<td>U3S4*</td>
<td>NCSU</td>
<td>10</td>
<td>4</td>
</tr>
<tr>
<td>11</td>
<td>U4S2*</td>
<td>NCSU</td>
<td>12.5</td>
<td>4</td>
</tr>
</tbody>
</table>

Duplicate specimens.

Table 3.7 Summary of Static Test Results

<table>
<thead>
<tr>
<th>Specimen Name</th>
<th>Ultimate Total Load</th>
<th>Ultimate Moment</th>
<th>Stiffness</th>
<th>Equivalent Number of Tubes*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>kips</td>
<td>Support kip-in</td>
<td>Mid-Span kip/in</td>
<td></td>
</tr>
<tr>
<td>U3S2.5</td>
<td>31.2</td>
<td>156.0</td>
<td>132</td>
<td>4.4</td>
</tr>
<tr>
<td>U3S4</td>
<td>33.9</td>
<td>321.9</td>
<td>59</td>
<td>5.5</td>
</tr>
<tr>
<td>U4S5</td>
<td>20.8</td>
<td>260.0</td>
<td>42</td>
<td>4.4</td>
</tr>
<tr>
<td>U4S2</td>
<td>30.2</td>
<td>105.8</td>
<td>297</td>
<td>3.8</td>
</tr>
<tr>
<td>B3S4</td>
<td>22.3</td>
<td>212.2</td>
<td>38</td>
<td>7.2</td>
</tr>
<tr>
<td>B3S2.5</td>
<td>32.2</td>
<td>161.0</td>
<td>46</td>
<td>5.7</td>
</tr>
<tr>
<td>U4D2</td>
<td>67.3</td>
<td>66.5</td>
<td>54</td>
<td>3.7</td>
</tr>
<tr>
<td>U3D4</td>
<td>71.7</td>
<td>199.8</td>
<td>33</td>
<td>5.8</td>
</tr>
<tr>
<td>U4D5</td>
<td>49.8</td>
<td>184.5</td>
<td>27</td>
<td>4.1</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Specimen Name</th>
<th>Deflection (in)</th>
<th>L /δ</th>
<th>Strain Ratio**</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(in)</td>
<td>L /δ</td>
<td>Top***</td>
</tr>
<tr>
<td>U3S2.5</td>
<td>0.62</td>
<td>48.4</td>
<td>No.B/No.A 0.11</td>
</tr>
<tr>
<td>U3S4</td>
<td>1.05</td>
<td>45.7</td>
<td>No.B/No.A 0.30</td>
</tr>
<tr>
<td>U4S5</td>
<td>1.2</td>
<td>50.0</td>
<td>No.B/No.A 0.41</td>
</tr>
<tr>
<td>U4S2</td>
<td>0.15</td>
<td>160</td>
<td>No.B/No.A 0.33</td>
</tr>
<tr>
<td>B3S4</td>
<td>0.64</td>
<td>75.0</td>
<td>No.B/No.A 0.62</td>
</tr>
<tr>
<td>B3S2.5</td>
<td>0.54</td>
<td>55.6</td>
<td>No.B/No.A 0.38</td>
</tr>
<tr>
<td>U4D2</td>
<td>0.36</td>
<td>66.7</td>
<td>No.B/No.A 0.31</td>
</tr>
<tr>
<td>U3D4</td>
<td>0.97</td>
<td>49.5</td>
<td>No.B/No.A 0.43</td>
</tr>
<tr>
<td>U4D5</td>
<td>1.08</td>
<td>55.6</td>
<td>No.B/No.A 0.34</td>
</tr>
</tbody>
</table>

* Equivalent tubes out of 12 for 3x3 tubular decks; out of 9 for 4x4 tubular decks
** Value at maximum load
*** A=6, B=4 and C=1 for 3x3 tubular decks and A=5 B=3 and C=1 for 4x4 tubular decks
Table 3.8 Matrix for Fatigue Test Specimens

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>U3D4</th>
<th>U4D2</th>
<th>U4D5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tube Size</td>
<td>3 x 3</td>
<td>4 x 4</td>
<td>4 x 4</td>
</tr>
<tr>
<td>Span Length</td>
<td>4 ft</td>
<td>2 ft</td>
<td>5 ft</td>
</tr>
<tr>
<td>Deck Width</td>
<td>2.5 ft</td>
<td>3 ft</td>
<td>3 ft</td>
</tr>
<tr>
<td>Number of Tubes</td>
<td>10</td>
<td>9</td>
<td>9</td>
</tr>
<tr>
<td>Maximum Load*</td>
<td>33.3 kips</td>
<td>20.8 kips</td>
<td>23.3 kips</td>
</tr>
<tr>
<td>Minimum Load*</td>
<td>5 kips</td>
<td>4.5 kips</td>
<td>4.5 kips</td>
</tr>
<tr>
<td>Stress Ratio</td>
<td>15%</td>
<td>22%</td>
<td>19%</td>
</tr>
<tr>
<td>Prestress at Support</td>
<td>4.0 kips</td>
<td>4.0 kips</td>
<td>4.0 kips</td>
</tr>
<tr>
<td>Prestress at Mid-Span</td>
<td>10.1 kips</td>
<td>12.1 kips</td>
<td>9.5 kips</td>
</tr>
<tr>
<td>Loading Frequency</td>
<td>3 Hz</td>
<td>3 Hz</td>
<td>2 - 3Hz</td>
</tr>
</tbody>
</table>

*Total load from the actuator.
Figure 3.1 FRP Specimen in Crushing Test

Figure 3.2 Corner Cracks
Figure 3.3 Flexural Cracks in Web

Figure 3.4 Oblique Shear Cracks
Figure 3.5 Web Buckling

Figure 3.6 Load-Deflection Curves for Crushing Tests of FRP Tubes
Figure 3.7 Normalized Load-Deflection Curves for Crushing Tests of FRP Tubes

<table>
<thead>
<tr>
<th>Load per Tube (lb)</th>
<th>Deflection (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>60,000</td>
<td>0.00</td>
</tr>
<tr>
<td>50,000</td>
<td>0.05</td>
</tr>
<tr>
<td>40,000</td>
<td>0.10</td>
</tr>
<tr>
<td>30,000</td>
<td>0.15</td>
</tr>
<tr>
<td>20,000</td>
<td>0.20</td>
</tr>
<tr>
<td>10,000</td>
<td>0.25</td>
</tr>
<tr>
<td>0</td>
<td>0.30</td>
</tr>
</tbody>
</table>

- 1 --- 3''x1a
- 2 --- 3''x1b
- 3 --- 4''x1
- 4 --- 4''x3
- 5 --- 3''x3bond
- 6 --- 3''x4
- 7 --- 3''x4bond
Figure 3.8a Numbering System of the 3x3 FRP Deck System
Fig 3.8b Numbering System of the 4x4 FRP Deck System
Figure 3.9 Tubular Load Cell for Prestressing Rod

Figure 3.10 Wire Potentiometers
Figure 3.11 Stroke Potentiometers

Figure 3.12 Typical Test Setup for Specimens at the UC
Figure 3.13 Typical Test Setup for Single Span Deck (Specimen B3S4)

Figure 3.14 Typical Test Setup for Double Span Deck (Specimen U3D4)
Figure 3.15 Longitudinal Crack Adjacent to Loading Plate (Specimen U4S2)

Figure 3.16 Failure Mode at the Support (Specimen U2S4)
Figure 3.17 Slippage between Adjacent Tubes (Specimen U3S4)

Figure 3.18 Failure at the Corner and Middle of Top Flange (Specimen U3S4)
Figure 3.19 Permanent Deformations in Center Tubes (Specimen U3S4)

Figure 3.20 Failure Mode of Specimen U3S2.5
Figure 3.21 Curling of Exterior Tubes at the Support (Specimen U3S2.5)

Figure 3.22 Failure Mode of Specimen B3S4
Figure 3.23 Failure of Specimen U4D2 at the End Support

Figure 3.24 Deformation of Center Tubes in Specimen U3D4
Figure 3.25 Failure at the Bottom Surface of Tubes at Center Support (Specimen U3D4)

Figure 3.26 Load-Deflection Curves for Specimen U3S4 at NCSU
Figure 3.27 Load-Deflection Ratios for Specimen U3S4 at NCSU

Figure 3.28 Load-Strains for Specimen U3S4 at UC
Figure 3.29 Load-Strain Ratios for Specimen U3S4 at UC

Figure 3.30 Load-Support Displacement for Specimen U3S4 at NCSU
Figure 3.31 Load Versus Prestress for Specimen U3S4 at NCSU

Figure 3.32 Load-Deflection Curves for Specimen U4S2
Figure 3.33 Load-Strains for Specimen U4S2

Figure 3.34 Load-Strain Ratios for Specimen U4S2
Figure 3.35 Load-Deflection Curves for Specimen U3S2.5

Figure 3.36 Load-Strains for Specimen U3S2.5
Figure 3.37 Load-Strain Ratios for Specimen U3S2.5

Figure 3.38 Load-Deflection Curves for Specimen U4S5
Figure 3.39 Load-Strains for Specimen U4S5

Figure 3.40 Load-Strain Ratios for Specimen U4S5
Figure 3.41 Load-Deflection Curve for Specimen B3S4

Figure 3.42 Load-Strains for Specimen B3S4
Figure 3.43 Load-Strain Ratios for Specimen B3S4

Figure 3.44 Load-Deflection Curve for Specimen B3S2.5
Figure 3.45 Load-Strains for Specimen B3S2.5

Figure 3.46 Load-Strain Ratios for Specimen B3S2.5
Figure 3.47 Load-Deflection Curve for Specimen U4D2.

Figure 3.48 Load-Strains for Specimen U4D2
Figure 3.49 Load-Strain Ratios U4D2 for Specimen

Figure 3.50 Load-Deflection Curves for Specimen U3D4
Figure 3.51 Load-Strains for Specimen U3D4

Figure 3.52 Load-Strain Ratios for Specimen U3D4
Figure 3.53 Load-Deflection Curves for Specimen U4D5

Figure 3.54 Load-Strains for Specimen U4D5
Figure 3.55 Load-Strain Ratios for Specimen U4D5

Figure 3.56 Comparison of Load-Deflection Curves for All FRP Deck Specimen
Figure 3.57 Configuration of the HS20-44 Truck [AASHTO 1999]

$W =$ Combined weight on the first two axles, which is the same as for the corresponding H truck.
$V =$ Variable spacing —14 ft to 30 ft, inclusive. Spacing to be used is that which produces maximum stresses.
Figure 3.58 Fatigue Test Setup (Specimen U4D2)

Figure 3.59 Local Failure at the End Support of Specimen U4D2
Figure 3.60 Longitudinal Crack on the Top Surface of Specimen U4D2

Figure 3.61 Ultimate Failure Mode at the End Support of Specimen U4D2
Figure 3.62 Failure on the Bottom Surface at the Center Support of Specimen U4D2

Figure 3.63 Test Setup for Specimen U4D5
Figure 3.64 Cracking at the Support in Specimen U4D5

Figure 3.65 Minor Crack at the Mid-Span of Specimen U4D5
Figure 3.66 Ruptured Prestressed Rod in Specimen U4D5

Figure 3.67 Test Setup for Specimen U3D4
Figure 3.68 Longitudinal Crack at the Top of Specimen U3D4

Figure 3.69 Failure of Tube No.4 in Reserve Strength Test of Specimen U3D4
Figure 3.70 Fractures in the Bottom at the Center Support of Specimen U3D4

Figure 3.71 Cracking in the Web at the Mid-Span of Specimen U3D4
Figure 3.72 Load-Deflection Curves for Specimen U4D2

Figure 3.73 Load-Strain Ratios for Specimen U4D2 Tube
No.3/5 Midspan Bottom
Figure 3.74 Load-Deflection Curves for Specimen U4D5

Figure 3.75 Load-Strain Ratios for Specimen U4D5 Tube No.3/5 Midspan Bottom
Figure 3.76 Load Deflection Curves for Specimen U3D4

Figure 3.77 Load-Strain Ratios for Specimen U3D4 Tube
No.3/5 Midspan Bottom
Figure 3.78 Load-Deflection Curves for Specimen U3D4 in Reserve Strength Test

Figure 3.79 Load-Strain Ratios for Specimen U3D4 in Reserve Strength Test
CHAPTER 4 FINITE ELEMENT MODELING

4.1 Introduction

In this chapter, the results of analytical work using finite element modeling for the prestressed FRP tubular deck system are presented. ANSYS Program Release 6.1 [ANSYS 2002] was used in this study. Four types of elements were selected for different components of the deck system, as follows:

- SHELL63 was used to model the flanges and webs of the FRP tube, since the thickness ratio is only 1/12 for the 3x3 and 1/16 for 4x4 the tubes. The element is a four-noded elastic shell with both bending and membrane capabilities (Figure 4.1). The shell can be loaded with both in-plane and normal loads. The element has six degrees of freedom at each node; three translations and three rotations. It also has stress stiffening and large deflection options.

- SOLID45 was used to model the 10 in x 20 in x 1 in steel plate. It is an eight-noded solid element with three translational degrees of freedom at each node (Figure 4.2). The element also has plasticity, creep, swelling, stress stiffening, large deflection, and large strain options.

- TARGE170 together with CONTA174 were used to model the contact between the steel plate and the FRP tube. The target surface is discretized by a set of TARGE170 elements (Figure 4.3), paired with CONTA174 elements (Figure 4.4) through a shared set of real constants. CONTA174 elements overlay the solid elements describing the boundary of a deformable body, potentially in contact and sliding with the target surface. CONTA174 has the same geometric characteristics.
as the solid or shell element face with which it is connected. Contact occurs when the element surface penetrates one of the target segment elements on a specified target surface. Coulomb and shear stress friction are both allowed in the analysis.

Mechanical properties for the FRP tubes followed those recommended by the manufacturer, as presented in Table 3.2. Material properties are defined in the local coordinate system of the element. Therefore, lengthwise and crosswise directions depend on the directions of the fibers in the pultruded laminates. For the steel plate, a Young’s modulus of $3 \times 10^6$ psi and a Poisson ratio of 0.3 is used.

For the most part, the FRP tube was meshed with $\frac{1}{2}$ in x $\frac{1}{2}$ in square elements. In smaller size models, $\frac{1}{4}$ in x $\frac{1}{4}$ in square elements were used for the analysis of the FRP tube. The steel plate was modeled by $\frac{1}{2}$ in x $\frac{1}{2}$ in x $\frac{1}{2}$ in solid elements.

Symmetry of the problem was taken into account for the various cases of study. For example, in crushing test of a single tube, only 1/8 of the tube was analyzed. Similarly, for static flexural test of deck panels, only 1/4 of the system was analyzed. Special attention was paid to the boundary conditions along the planes of symmetry.

After generating the model geometry and mesh for the FRP tube and the steel plate, surface to surface contacts were developed in a step by step procedure, as follows:

1. The contact zone and contact pairs were selected on those surfaces where real contact phenomenon was observed in the experiments.

2. Contact and target elements were selected. Contact elements are constrained against penetrating the target surface, while target elements can penetrate into the contact surface. Since stiffness of the steel plate is much greater than that of the FRP tube, their contact is considered to be of a rigid-to-flexible type.
Steel plate was chosen as the target surface, and the FRP tube was chosen as the contact surface. For two adjacent tubes, one was chosen as the contact surface, while the other one served as the target element.

3. The element key options and real constant sets were chosen. ANSYS uses a set of 20 real constants and several element key options to control the contact behavior. Of the 20 real constants, two are used to define the geometry of the target surface elements. The remaining is used by the contact surface elements. In the analysis, the system default values were used in most cases. The parameters chosen for each analysis will be reported later for each case.

4. Contact surface was then automatically generated by the program. Before completing the process, the direction of the “outward normal to the surface” for the contact elements was verified to point towards the target surface; as otherwise, the system may lose its stability.

After completion of the meshing process and the contact elements, the constraints and the applied loads were defined, and the analysis results were developed, as discussed in the following sections.

4.2 Modeling of FRP Tubes in Crushing Tests

4.2.1 Single Tube

In this section, analysis of a 20 in long segment of a single 4x4 tube is presented. Due to symmetry, only 1/8 of the tube is modeled. Figure 4.5 shows the meshing and the boundary conditions for 1/8 model of the tube in compression, where the 2 in x 2 in x 10 in “L” shape is the FRP tube modeled with ½ in x ½ in shell elements. The Z axis in the
global coordinate system is set as the lengthwise direction for the FRP tube, while the $X$ and $Y$ axes are crosswise directions for the flange and the web, respectively. The steel plate is meshed using $\frac{1}{2}$ in cubes as solid elements.

From among the 20 contact factors, the “normal penalty stiffness” scaling factor is one of the most critical parameters affecting the results. Usually, a sufficiently high stiffness is preferred such that the “contact penetration” would be acceptably small. Higher stiffness values decrease the penetration, but may lead to ill-conditioning of the global stiffness matrix and convergence difficulties. The scaling factor may be selected between 0.01 and 10, although ANSYS [ANSYS 2002] recommends a much narrower range between 0.01 to 0.1 for bending dominated problems of slender structures. In this study, a scaling factor of 0.25 was found acceptable. The penetration tolerance factor was set at 0.1. The friction coefficient was set at 0.2 for the steel-FRP contact surface. However, the results did not seem to be greatly affected by the friction factor. System defaults were used for all other contact factors.

Figure 4.7 shows the perspective view and the front view of the deformed shape of a single FRP tube in crushing test. The maximum deformations are -0.044 in and -0.052 in along the $X$ and $Y$ axes, respectively. The maximum penetration is 0.011 in at the corner. The deformed shape reflects the same pattern observed during the tests, as the compressed flange buckles and separates from the loading plate. The separation justifies the need for using contact elements instead of a uniformly distributed pressure. Figure 4.6 shows a good agreement between the predicted load-deflection response with that of the experiments. The maximum difference is about 10%, when the tube begins softening near its ultimate capacity.
Figure 4.8 shows two different views of the stress contours in the crosswise or “weak” direction of the ply. The maximum tensile stress shown in the crosswise direction is about 98% of the respective capacity, as reported in Table 3.2. The maximum tensile stress in the lengthwise direction, from a similar plot not shown here, is only 31% of the respective capacity. Therefore, the tube is expected to fail at the corner, with cracks in the longitudinal direction. This is similar to what was observed in the tests.

4.2.2 Multiple Unbonded Tubes

Specimen 4” from Table 3.1 was selected for modeling of multiple unbonded tubes in compression. Due to symmetry, only 1/8 of the specimen was meshed with ¼ in x ¼ in shell elements. Figure 4.9 shows the finite element mesh in its deformed shape. Two contact surfaces were generated: one between the two tubes and another between the steel plate and the FRP tube. The friction coefficient for contact surface between the two FRP tubes was selected as 0.3, and the coefficient for contact surface between the FRP tube and the steel plate was chosen as 0.2. The normal penalty stiffness factors for the FRP-FRP interface and FRP-steel interface were chosen as 0.2 and 0.25, respectively. All other contact parameters were similar to those of the single tube, described earlier.

Figure 4.10 shows a good agreement between the predicted load-deflection response with test results. The maximum difference is only 5%, as the specimen approaches its ultimate capacity. It should be noted that the reaction of the web that is not directly under the steel plate is only 2% of the applied load. Test results also showed no sign of distress in the exterior web, although not at such a drastic ratio as the analysis indicates. Figure 4.11 depicts the predicted stress contours in the crosswise direction for
the tested specimen. Similar to the case of a single tube, the flanges buckle and separate from the steel plate. Also, stress concentration can be seen at the corner of the tubes.

4.3 Modeling of FRP Tubes in Flexure Tests

4.3.1 Bonded Decks

Specimen B3S4 from Table 3.4 was selected for modeling of the bonded decks in flexure. Due to symmetry, only ¼ of the deck was meshed with ½ in x ½ in shell elements (Figure 4.12). Since no separation was expected between the bonded surfaces of the tubes, bonded webs were simply modeled with twice the thickness of a single web. Care was taken to appropriately identify the crosswise and lengthwise properties of the FRP shell elements. At the support, all vertical deflections were restrained for simplicity. Therefore, the “curling” phenomenon observed during the experiments were neglected. However, as will be shown later, this simplification did not introduce significant error in the analysis. Since the steel plate was placed on a neoprene pad in the test setup, the applied load was simulated as a uniformly distributed pressure, rather than a contact surface. Prestressing was also applied as a uniformly distributed pressure on the side of the specimen over the bearing area of the concrete-filled tube at the mid-span or the steel bearing plate along the support line. This clearly introduces some approximation in the analysis, since the threaded rods were not directly modeled. However, the assumption is justified, as long as the deck does not bear on the rods.

Figure 4.13 shows the deflection contours for the specimen, as predicted by the finite element analysis. The contours are plotted at an applied load of 20 kips, i.e., 80% of the capacity used in the experiment. The areas with dark gray colors are under direct
loading. As expected and modeled for, no separation is observed between the tubes. Figure 4.14 shows a good agreement between the predicted load-deflection response with test results. The deflection in the figure is from the node at the edge of the loading areas, since it represents the test conditions better than that of the mid-span.

Figure 4.15 shows the stress contours for the top surface of the FRP elements in the crosswise or “weak” direction. Stress concentration is clearly observed between tubes No. 4 and 5. Also, along the support line where prestressing is applied, some stress concentration is noted.

Figure 4.16 compares the predicted load-strains with test data. The finite element results are shown as dashed lines, whereas test data are shown as solid lines. Strains are in the lengthwise direction at the nodes where strain gages were physically attached to the specimen in the tests. The predicted strains are slightly larger than the test data, primarily due to the assumptions made for the modeling of the prestressing force and for the perfect bond assumed between the tubes.

### 4.3.2 Unbonded Decks

Specimen U3S4 from Table 3.4 was selected for modeling of the unbonded decks in flexure. Again, only ¼ of the deck was meshed with ½ in x ½ in shell elements. Except for the bond between the tubes, all other loading and boundary conditions were the same as those described in the previous section. Contact surfaces were developed between the FRP tubes with a normal penalty stiffness factor of 0.2, a friction coefficient of 0.3, and a penetration tolerance factor of 0.1.
Figure 4.17 shows the deflection contours for the specimen, as predicted by the finite element analysis. The contours are plotted at an applied load of 30 kips, i.e., 90% of the capacity used in the test. The slippage between tubes No. 4 and 5 is very clear, as expected from the tests. The displacement at the edge of the loading plate is 1.05 in, which matches the test result of 0.95 in, fairly well.

Figures 4.18 and 4.19 compare the predicted load-deflections and load-strains, respectively, with the test data. The predicted values are shown as dashed lines, whereas test data are shown as solid lines. Good agreement is noted. The difference may be attributed to the collective effect of the assumptions made in the analysis, for example, ignoring the participation of the threaded rods in taking some of the loads. In general, the finite element model provides a good simulation of the load distribution between the various tubes.

Figure 4.20 shows the stress contours for the top surface of the FRP elements in the crosswise or “weak” direction. Stress concentration at the corner of tube No. 5 indicates a longitudinal cracks, just as was observed during the tests (Figure 4.21).

### 4.4 Load Rating of the Bridge with FRP Deck

As stated in Chapter 1, the FRP tubular deck system was used in the field to replace the existing timber deck on the Tyler Road Bridge in Delaware County, Ohio. Figure 4.22 shows an overall view of the bridge before deck replacement. The bridge consisted of two steel trusses, a series of floor beams and closely spaced stringer I-beams with a timber deck and an asphalt overlay. Figure 4.23 shows the bottom view of some the rusted stringers with the rotten timber deck. As part of the deck replacement, all steel
members in the floor system and the supporting trusses were taken out and galvanized in a plant near Columbus, Ohio. Figure 4.24 shows the new bridge with galvanized steel members, the FRP deck, and an asphalt overlay. The bridge was load tested, using a dump truck filled with sand, before and after deck replacement. Figure 4.25 shows the truck load testing of the bridge before deck replacement. In this section, finite element analysis is used to rate the floor system in the new bridge.

Load rating of the bridge followed the procedures suggested by AASHTO for condition assessment of bridges [AASHTO 1994]. The following general equation is used to generate the rating factor \( RF \) for the live load carrying capacity of the structure:

\[
RF = \frac{C - A_1 D}{A_2 L(1 + I)}
\]  \hspace{1cm} (4.1)

where \( C \) is the capacity of the member, \( D \) and \( L \) are the dead load and live load effects on the member, respectively, \( A_1 \) and \( A_2 \) are the factors for the dead load and live load, respectively, and \( I \) is the impact factor to be used with the live load effect, as given by:

\[
I = \frac{50}{l + 125} \leq 0.3 \]  \hspace{1cm} (4.2)

where \( l \) is the span length of the member in foot. The rating factor \( RF \) can then be used to determine the rating tonnage \( RT \) of the bridge, as follows:

\[
RT = (RF)W \]  \hspace{1cm} (4.3)

where \( W \) is the weight of the nominal truck used to determine the live load effect. The rating of the bridge is controlled by the member with the lowest rating in tons.

Each highway bridge should be rated for two different levels: inventory and operating. The inventory level corresponds to the customary design level of stresses, but reflects the existing bridge and its material conditions with regard to deterioration and
loss of the section. Load ratings based on the inventory level allow comparisons with the capacity for new structures, and therefore, result in a live load which can safely utilize an existing structure for an indefinite period of time. Load ratings based on the operating level generally describe the maximum permissible live load, to which the structure may be subjected. Allowing unlimited numbers of vehicles to use the bridge at the operating level may shorten the life of the bridge. Therefore, a different member capacity is used for the inventory and the operating levels of rating.

The AASHTO manual [AASHTO 1994] describes two methods for load rating of bridge members: the allowable stress method and the load factor method. In the allowable stress method, both load factors of $A_1$ and $A_2$ are chosen as 1, while the capacity $C$ depends on the rating level desired, with the higher values selected for the operating level. In the load factor method, the capacity $C$ is the nominal strength of the member, and $A_1$ is selected as 1.3, irrespective of the rating level desired. On the other hand, the value of $A_2$ is either 1.3 for the operating level or 2.17 for the inventory level. In this study, the allowable stress method is used for rating of the new floor system in the Tyler Road Bridge.

As stated earlier, the new bridge consists of a prestressed tubular FRP deck system attached to series of longitudinal stringers using clip angles. The stringers rest on a number of transverse floor beams, which are in turn supported by the two main steel trusses. Figures 4.26 and 4.27 show the details of the steel truss and the floor system, respectively. At 19 ft 4 in, although wide enough to accommodate two passenger cars, the bridge is expected to carry a single truck at a time, especially since it is located in a rural area. Therefore, the live load was modeled with a single lane centered across the bridge.
As such, the bridge was considered symmetric for both dead and live loads, and only one half of the bridge was modeled (Figure 4.28).

The FRP deck was modeled using a single layer of SHELL63 elements with orthotropic properties. An equivalent thickness was therefore, calculated for the FRP deck based on its moment of inertia. The crosswise or “weak” direction of the shell elements was considered to be in the direction of traffic. Both the stringers and the floor beams were modeled using three-dimensional BEAM4 elements. Truss members were modeled using three-dimensional LINK8 bar elements with appropriate sectional properties for each member. To simplify the modeling process, slight adjustments were made to the dimensions on the plans, as shown in Table 4.1.

<table>
<thead>
<tr>
<th>Description</th>
<th>Dimension on Plans (in)</th>
<th>Dimension in Model (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Center to Center Spacing of Floor Beams</td>
<td>191</td>
<td>190</td>
</tr>
<tr>
<td>Bridge Length</td>
<td>1,337</td>
<td>1,330</td>
</tr>
<tr>
<td>Center Line of Truss to Deck Edge</td>
<td>9¼</td>
<td>10</td>
</tr>
<tr>
<td>Deck Edge to First Stringer</td>
<td>6¼</td>
<td>5</td>
</tr>
</tbody>
</table>

The capacity of stringers and floor beams were governed by the compression in extreme fibers of the section [(AASHTO 1994)]. Detailed calculations showed that since unsupported lengths of the stringers and the floor beams are much less than their respective critical values, the equation for partial lateral support does not govern for either case. Therefore, a capacity of 0.55F_y and 0.75F_y are used for the inventory and operating levels of rating, respectively. These values translate into safety factors of 1.82 and 1.33 for the inventory and operating levels of rating, respectively. The yield strength for the ASTM A572 GR50 Galvanized steel members used in this project is 50 ksi. The capacity of the FRP deck was provided by the manufacturer, as shown in Table 3.2. The
manufacturer also suggests a safety factor of 2.5 for the FRP deck. Therefore, for the crosswise “weak” direction, a capacity of $7,000/2.5 = 2,800$ psi was used in the load rating. The same safety factor was used for both operating and inventory levels, due to lack of specific AASHTO guidelines for FRP decks.

Dead loads and superimposed dead loads for the bridge were identified on the plans as $5$ psf for the FRP deck, $30$ psf for the asphalt wearing surface, and $15$ psf for the future wearing surface, at a total of $50$ psf.

The live load for the bridge was the standard AASHTO HS20 loading, as shown in Figure 3.57 [AASHTO 1999]. The maximum effects in the FRP deck, stringers and floor beams were found based on the five loading cases shown in Figure 4.29. The loading cases included two different positions for the truck loading and three different positions for the lane loading. The figure shows a single stringer supported by several floor beams, which are marked as triangles.

The impact factor for the live load turns out to be the maximum allowed value of $0.3$ for the FRP deck, stringers and floor beams. Both load factors $A_1$ and $A_2$ were chosen as $1$ for the allowable stress method.

Table 4.2 summarizes the rating factors and the bridge rating (tons) for the various members in the floor system. The rating $RT$ is calculated as the product of the rating factor $RF$ and the tonnage of HS20 truck (36 tons). The most critical member is the floor beam under Truck Load Position A (Load Case 1), as shown in Figure 4.29. The bridge is therefore, rated for $30$ tons and $19$ tons at the operating and inventory levels, respectively.
Table 4.2 Rating Factor and Bridge Rating of the Floor System for the Tyler Road Bridge

<table>
<thead>
<tr>
<th>Member</th>
<th>Operating Level</th>
<th>Inventory Level</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Rating Factor (RF)</td>
<td>Rating (RT) (Tons)</td>
</tr>
<tr>
<td>FRP Deck</td>
<td>1.47</td>
<td>53</td>
</tr>
<tr>
<td>Stringer</td>
<td>1.11</td>
<td>40</td>
</tr>
<tr>
<td>Floor Beam</td>
<td>0.83</td>
<td>30</td>
</tr>
</tbody>
</table>
\( x_{II} = \text{Element } x\text{-axis if ESYS is not supplied.} \)

\( x = \text{Element } x\text{-axis if ESYS is supplied.} \)

Figure 4.1 Shell63 Elastic Shell Element [ANSYS 2002]

Figure 4.2 Solid45 Structural Solid Element [ANSYS 2002]
Figure 4.3 Targe170 Target Surface Element [ANSYS 2002]

Figure 4.4 Conta174 3-D Surface to Surface Contact Element [ANSYS 2002]
Figure 4.5 Meshing and Boundary Conditions for a Single 4x4 Tube in Compression

![Meshing and Boundary Conditions for a Single 4x4 Tube in Compression](image)

Figure 4.6 Comparison of Finite Element Results with Experiments for a Single 4x4 Tube

![Comparison of Finite Element Results with Experiments for a Single 4x4 Tube](image)
Figure 4.7 Deformed Shapes for a Single 4x4 Tube
Figure 4.8 Stress Contours in Crosswise Direction for a Single 4x4 Tube
Figure 4.9 Deformed Shape of Three 4x4 Tubes

Figure 4.10 Comparison of Finite Element Results with Experiments for Three 4x4 Tubes
Figure 4.11 Stress Contours in Crosswise Direction for Three 4x4 Tubes

Figure 4.12 Meshing of Specimen B3S4
Figure 4.13 Deflection Contours for Specimen B3S4

Figure 4.14 Comparison of Finite Element Results with Experiments for Specimen B3S4
Figure 4.15 Stress Contours at the Top for Specimen B3S4

Figure 4.16 Comparison of Finite Element Load-Strain Results with Experiments for Specimen B3S4
Figure 4.17 Deflection Contours for Specimen U3S4

Figure 4.18 Comparison of Finite Element Results with Experiments for Specimen U3S4
Figure 4.19 Comparison of Finite Element Load-Strain Results with Experiments for Specimen U3S4

Figure 4.20 Stress Contours at the Top of Specimen U3S4
Figure 4.21 Failure of Specimen U3S4 Observed During the Tests

Figure 4.22 Tyler Road Bridge Before Deck Replacement
Figure 4.23 Bottom View of the Floor System Before Deck Replacement

Figure 4.24 Tyler Road Bridge with FRP Deck
Figure 4.25 Truck Load Testing of Tyler Road Bridge

Figure 4.26 Details of the Truss in Tyler Road Bridge
Figure 4.27 Details of the Floor System in Tyler Road Bridge

Figure 4.28 Finite Element Model of the Tyler Road Bridge
Case 1: Truck Load Position A

Case 2: Truck Load Position B

Case 3: Lane Load Position A

Case 4: Lane Load Position B

Case 5: Lane Load Position C

Figure 4.29 Live Load Cases for Bridge Rating
CHAPTER 5 CONCLUSIONS

The research reported in this thesis involved laboratory testing and analytical modeling of a new FRP deck system. The new deck consists of a series of pultruded FRP tubes with square section, laid side by side on the existing stringers, perpendicular to the direction of traffic. The tubes are then post-tensioned at mid-point between the stringers in the direction of traffic. In order to distribute the prestressing force, the last couple of tubes at each end of the bridge are filled with concrete.

The experimental work consisted of three components, as follows: (a) Crushing tests of seven FRP tubular specimens laid on their sides to establish their squash load capacity, (b) Flexural static tests of eleven FRP decks to assess their flexural behavior, and (c) Flexural fatigue tests of four FRP decks to determine their endurance limits.

The analytical work consisted of three components, as follows: (a) Modeling and simulation of crushing test for a single FRP tube and multiple unbonded FRP tube specimens, (b) Modeling and simulation of static flexural test for a bonded and an unbonded FRP deck panel, and (c) load rating of the floor system for the Tyler Road Bridge, with the new FRP deck in place.

The following conclusions were made from this study:

1. Failure mode, stiffness and capacity of the deck system are all functions of the FRP material properties and tube size, span length, interface bond and prestress level. In general, longer span decks fail in bending, whereas shorter span decks suffer from local shear failure due to stress concentrations at the corner of the tubes most adjacent to the applied load or at the support.
2. The deck system has some redundancy and reserved strength built into it by means of prestressing strand or bar. Although this reserved strength was not accounted for neither in the experiments nor in the analysis, it provides some safety factor for the deck. This point may be further underscored by the fact that observed failures for the most part were local to the individual tubes.

3. Panel action in the deck system seemed inadequate for the most part, as only the tubes that were directly under the load carried majority of the load. Slippage between these tubes and their adjacent tubes, as observed during the static and fatigue tests, may lead to cracking in the asphalt overlay. Panel action is generally improved at higher prestress levels or by epoxy bonding of the sides of the tubes.

4. Fatigue susceptibility of short span decks is alarming, and requires further investigation. The fatigue problem is less critical with an increase in the panel action, longer span decks, and smaller tube sizes. Another alarming issue that was raised during the tests was the creep effect. However, this is not expected to be of concern in bridge deck applications.

5. The finite element analysis can provide a good simulation of the FRP deck system with its various components, in crushing or bending tests.

6. Load rating of the bridge shows that the most critical element in the floor system is not the deck, but rather the floor beams. The bridge was rated for 30 tons and 19 tons at the operating and inventory levels, respectively.

7. In summary, the new deck system seems feasible and cost-effective for bridges with limited truck traffic and closely spaced stringers, where lack of panel action is not a concern.
Performance of the proposed deck system may be improved by enhancing its panel action and strengthening it in its weak direction. This may be achieved by one or both of the methods outlined below:

1. Epoxy bonding of the sides of the tubes, and sandwiching a limited number of tubes in between a top and bottom bi-directional FRP plate to produce a deck panel. The deck panels can be placed on the stringers, and then post-tensioned through the holes left in the tubes.

2. Filling the tubes with lightweight concrete or foam to improve their stiffness, and to prevent local buckling of the individual tubes.

Through the study undertaken for this thesis, a few research needs were identified that could be explored in future studies. Perhaps the most important aspect is the performance of the deck system with the asphalt overlay to ensure its endurance under fatigue loading. Slippage between the adjacent tubes could potentially cause distress and cracking in the overlay, unless a flat FRP plate is placed on top of the deck system, as proposed in Item 1 above. Another observed behavior is the curling of the end tubes, particularly in short span decks. In the field, as discussed in Chapter 1, the tubes are attached to the stringers with one clip angle at every 12 in. Therefore, it is important to study the overall system behavior with the deck and the stringers together to ensure no such curling occurs under traffic loads.
REFERENCES


