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Performance of concrete beams and slabs reinforced with FRP grids

Edwin Robert Schmeckpeper

University of New Hampshire, Durham

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Performance of concrete beams and slabs reinforced with FRP grids

Abstract
The objective of this dissertation was to evaluate the suitability of Fiber Reinforced Plastic (FRP) grids for use as a structural reinforcement in concrete structures such as highway bridge decks. The work concentrated on determining the mechanical properties of FRP materials, testing the flexural behavior and servicability of concrete beams and slabs reinforced with FRP grids, and determining splice and development length requirements for FRP grids. Design recommendations concerning failure mode, deflections, anchorage requirements were developed.

The results from the flexural tests on FRP reinforced concrete beams, with reinforcing ratios ranging from 0.3% to 2.2%, showed that the failure mode, measured deflections and ultimate loads were consistent with predictions. Behavior of FRP reinforced concrete beams subjected to cyclical loading was shown to be comparable with the behavior of steel reinforced concrete beams.

The tests on FRP reinforced concrete bridge deck slabs showed that FRP reinforced slabs can be designed to satisfy AASHTO load and servicability requirements.

The splice/development length tests on FRP reinforced concrete beams showed that the formula derived to predict the development length was conservative and consistent with the ACI 318 code.

Keywords
Engineering, Civil, Engineering, Materials Science

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Performance of concrete beams and slabs reinforced with FRP grids

Schmeckpeper, Edwin Robert, Ph.D.
University of New Hampshire, 1992
PERFORMANCE OF CONCRETE BEAMS AND SLABS
REINFORCED WITH FRP GRIDS

By

Edwin Robert Schmeckpeper
B.S., Valparaiso University, 1978
M.S., University of New Hampshire, 1986

DISSERTATION

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in
Engineering

May 1992
This dissertation has been examined and approved.

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May 8, 1992
Date
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ABSTRACT

PERFORMANCE OF CONCRETE BEAMS AND SLABS
REINFORCED WITH FRP GRIDS

By

Edwin Robert Schmeckpeper
University of New Hampshire, May 1992

The objective of this dissertation was to evaluate the suitability of Fiber Reinforced Plastic (FRP) grids for use as a structural reinforcement in concrete structures such as highway bridge decks. The work concentrated on determining the mechanical properties of FRP materials, testing the flexural behavior and servicability of concrete beams and slabs reinforced with FRP grids, and determining splice and development length requirements for FRP grids. Design recommendations concerning failure mode, deflections, anchorage requirements were developed.

The results from the flexural tests on FRP reinforced concrete beams, with reinforcing ratios ranging from 0.3% to 2.2%, showed that the failure mode, measured deflections and ultimate loads were consistent with predictions. Behavior of FRP reinforced concrete beams subjected to cyclical loading was shown to be comparable with the behavior of steel reinforced concrete beams.

The tests on FRP reinforced concrete bridge deck slabs showed that FRP reinforced slabs can be designed to satisfy AASHTO load and servicability requirements.

The splice/development length tests on FRP reinforced concrete beams showed that the formula derived to predict the development length was conservative and consistent with the ACI 318 code.
CHAPTER I
INTRODUCTION

1.0 Introduction

Highway bridge decks are often subjected to corrosive materials, such as deicing agents and salt water. When these materials penetrate the porous concrete, they cause the reinforcing steel to corrode. The reinforcing steel expands as it corrodes, which in turn results in damage to the surrounding concrete. As one possible solution to this problem, researchers at the University of New Hampshire are currently involved in a research program to determine the suitability of materials which are not susceptible to corrosion, such as Fiber Reinforced Plastic (FRP) reinforcements, for use as the structural reinforcement in concrete structures.

1.1 Purpose and Scope

The objective of this thesis is to evaluate the suitability of Fiber Reinforced Plastic (FRP) grids for use as a structural reinforcement in concrete structures such as highway bridge decks. The work is concentrated on determining the mechanical properties of FRP materials, determining the flexural behavior and serviceability of
concrete beams and slabs reinforced with FRP grids, determining splice and development length requirements of FRP grids, and the development of design recommendations.

1.2 Background

The history of the use of FRP members in the reinforcement of concrete has alternated between periods of increasing attention followed by periods of waning interest. In each of these cycles, interest in FRP as a reinforcement for concrete followed advances in material capabilities or production efficiencies, but then later declined as economic, manufacturing, and construction factors are taken into consideration.

For example, the shortages of steel which occurred during World War II lead to an increased interest in alternative reinforcements for concrete structures. Taking advantage of this interest, in 1941 John G. Jackson applied for a patent on the use of fiberglass bars to reinforce concrete structural elements.[29] By the time the patent was granted in 1947, steel became plentiful, and consequently the use of FRP as a reinforcement for concrete was not adopted by the construction industry.

Subsequent researchers, such as Ivan A. Rubinsky at Princeton University in 1951[53], Ray B. Crepps at Massachusetts Institute of Technology also in 1951[30], N.F. Somes London University in 1963[56], and Wines and Hoff of
the U.S. Army Engineers Waterway Experiment Station in
1966[60], concentrated on the use of FRP reinforcements as a
means to prestress concrete structures. They were
interested in this application since the low modulus of
elasticity and high tensile strength of the FRP
reinforcements should theoretically result in reduced
prestressing losses compared to the losses obtained when
using steel reinforcing strands.

During their test program, Wines and Hoff conducted
long-duration tension tests on fiberglass rods and concrete
beams reinforced with fiberglass rods[60]. The specimens
used in their tension tests were very highly stressed, being
loaded to 75% and 90% respectively of their short-term
static capacity. The fiberglass reinforced concrete beams
in their tests were loaded to 75% of the concrete beam's
ultimate capacity. Since the ultimate load level in the
concrete beam was not governed by the stresses in the
fiberglass reinforcement, but rather by stresses in the
concrete, the load levels in the fiberglass reinforcement
were significantly less than 75% (10% and 20%). Results
from these tests indicated that at low stress levels, the
rate of change in creep in fiberglass reinforced concrete
beams was not significantly different from that in steel
reinforced beams.

In discussing creep rates in various FRP composites,
ASCE Report #63[9] mentions that when loaded to a low level
of stress, the strain in a glass fiber based FRP composite will increase approximately 30% over 10⁶ hours (approximately 100 years). Carbon fiber based FRP composites using the same resin as the fiberglass based FRP composites, are expected to behave in a similar fashion. The manufacturer of NEFMAC has conducted only limited duration creep relaxation tests which support this observation.

These early researchers were less interested in FRP reinforcements for use in non-prestressed concrete, and expressed doubts concerning the potential suitability of FRP for use in this application. One of the earlier researchers in this field, Ivan A. Rubinsky, in articles published in the "Magazine of Concrete Research", September 1954[54] stated that due to the low modulus of elasticity of fiberglass, it was inappropriate to use fiberglass as a non-prestressed reinforcement in concrete members.

Subsequent researchers[15][31][56] reached the same conclusion as Rubinsky concerning the suitability of FRP reinforcements, primarily since they were treating FRP reinforcements as direct replacement for steel reinforcements, subjecting the FRP reinforcements to the same stress criteria used for steel reinforcements. For example, in a 1966 report by J.C. Wines and G.C. Hoff, "Laboratory Investigation of Plastic-Glass Fiber Reinforcement for Reinforced and Prestressed Concrete:
Report 1, Plain Reinforced Concrete", it was concluded that since only a small fraction of the FRP's structural capacity was being utilized, its use as a reinforcement for concrete was neither efficient nor economical.[60]

More recent researchers, such as Nawy, Neuwerth, and Phillips in 1971[41] and 1977[42], reached an entirely different conclusion concerning the suitability of FRP as a reinforcement for concrete. They concluded that due to factors such as ease of fabrication, corrosion control and installation costs, the use of fiberglass to reinforce concrete was both practical and potentially cost effective.[12][25]

In short, the economics of a structure do not solely depend upon how highly stressed an individual member is, but rather upon the fabrication cost, construction cost, and service life cost of the structure.

1.3 Current Research

The use of FRP bars for the reinforcement of concrete has been limited, in part, due to the poor bonding characteristics of FRP bars. For example, Larralde, Renbaum, and Morsi[32], in a report to Transportation Research Board Task Force A2c51, dated August 1988, expressed concern that the lack of adequate bond would eliminate the composite behavior of FRP reinforced concrete members. These concerns were restated in later work done by

The use of FRP grids addresses the problems with poor bond performance by developing bond through direct concrete bearing on bars which are placed transverse to the longitudinal axis of the main reinforcing members.[43] However, the use of grids instead of single bars, leads to increased crack width since the grids develop force transfer at discrete locations, rather than continuously along their entire length.[25] In addition, problems have been encountered with the connection details since, unlike steel, FRP grid reinforcing materials cannot be bent or shaped into the standard connections used with steel reinforced concrete. Finally, in order for FRP grids to be used in concrete structures, splicing and development length requirements must be determined.

The research presented in this thesis has been directed towards determining the performance characteristics of concrete beams and bridge deck slabs reinforced with FRP grids and towards determining the splice/development length requirements for FRP grids. This work is one of the steps required to determine the suitability of the FRP material for structural applications such as bridge decks or pavements.
CHAPTER II
FRP MATERIAL AND MECHANICAL PROPERTIES

2.0 Introduction
This chapter deals with determining the material and mechanical properties of the Fiber Reinforced Plastic (FRP) grid reinforcements. Experimental results are compared to theoretical calculations of the FRP properties.

2.1 Fiber Reinforced Plastic Composites
Fiber Reinforced Plastic (FRP) composites are composed of relatively high strength parallel fibers enclosed in a resin matrix, which serves to bind the fibers into a single structure. The resin matrix provides the means to transfer applied stresses to the fibers and protects the fibers from deleterious interactions from the environment, such as oxidation and corrosion.

2.2 NEFMAC Fiber Reinforced Plastic Composite Grid
The longitudinal and transverse bars in NEFMAC FRP Composite Grids are fabricated using a process in which bundles of carbon or glass fiber filaments are impregnated with a vinyl ester resin, and then woven in two or three
dimensional patterns to form the bars of the reinforcement grid. The finished grids are then pressed between heated steel plates which flatten the upper and lower surfaces of the bars (See Figure 1 and Photograph 2).

![Diagram of NEFMAC FRP Grid]

**Figure 1 - NEFMAC FRP Grid**

The result of this "built-up" fabrication process is that while the fiber content of the bars is accurately controlled, the bars have an irregular cross section. The bars are smooth on the top and bottom with irregular sides.

The FRP grids are produced in several sizes and configurations. The manufacturer designates the FRP bars by a letter which indicates the material type, followed by a number which represents the bar size, followed by the bar interval.

Two different types of fiber reinforced plastic
composites were used in this test program: The first type, designated "C", is predominately carbon fiber, while the second, designated "H", is a hybrid of carbon and glass fibers, with glass fibers predominating. The relative proportions of carbon and glass fibers are shown in Table 3.

The FRP bar size designation indicates the diameter of a Grade 70 steel reinforcing bar with the same ultimate strength as the FRP bar. The FRP bar size is not necessarily related to any actual dimension of the bar. Several different bars sizes of each type were used in this test program. The bars ranged in size from C10 and H10, with the nominal capacity of a 10mm steel bar, to C22 and H22, with the nominal capacity of a 22mm steel bar.

The grids are typically produced in two dimensional sheets, composed of longitudinal and transverse bars. The size of the individual bars and the interval between bars may be varied. The grids used in this test program were produced with equal sized longitudinal and transverse bars, equally spaced in both directions. The centerline to centerline bar spacing was either 100mm (4") or 150mm (6").
2.3 Reinforcing Fibers

Two different types of fibers were used in the manufacturing of the FRP reinforcements used in this test program: The first type of fiber was high strength carbon, while the second type was E-glass. Photograph 2 shows a Scanning Electron Microscope (SEM) view of the carbon and E-glass fibers in a NEFMAC FRP bar.
The manufacturer produced the initial FRP grid reinforcements using type T300 carbon fibers. When the higher strength T700 fibers became commercially available, the manufacturer switched to the type T700 fibers. The fiber properties are shown in Table 1.

A vinyl ester resin with a specific gravity of 1.15 was used for the matrix in the manufacture of the NEFMAC FRP composites. The material properties for the fibers used in the NEFMAC fiber reinforced plastic composite material were provided by the manufacturer.[40]
<table>
<thead>
<tr>
<th>Fiber Type</th>
<th>T300 HS-Carbon</th>
<th>T700 HS-Carbon</th>
<th>R2220 E-Glass</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strand Type</td>
<td>tow</td>
<td>tow</td>
<td>roving</td>
</tr>
<tr>
<td>Ultimate Strength (ksi)</td>
<td>3090 (450)</td>
<td>4810 (700)</td>
<td>1400 (200)</td>
</tr>
<tr>
<td>Density (ρ) g/cm³</td>
<td>1.77</td>
<td>1.82</td>
<td>2.54</td>
</tr>
<tr>
<td>Texture (T) g/1000m</td>
<td>800</td>
<td>800</td>
<td>2220</td>
</tr>
<tr>
<td>Young's Modulus (E) (ksi)</td>
<td>230,500 (33,400)</td>
<td>230,500 (33,400)</td>
<td>72,600 (10,500)</td>
</tr>
<tr>
<td>Maximum Elongation</td>
<td>1.3</td>
<td>1.4</td>
<td>2.0</td>
</tr>
</tbody>
</table>

The cross-sectional area of each fiber component is calculated as follows:

\[ A_i = \frac{T_i \times n_i}{\rho_i \times 1000} \]

Where:
- \( n_i \) = number of fiber bundles (roving or tows)
- \( T_i \) = Texture of fiber (g/1000m)
- \( \rho_i \) = Density of fiber (g/cm³)

Note: Roving - a number of strands of glass fibers collected into a parallel bundle with little or no twist.[8]

Tow - an untwisted bundle of continuous carbon filaments.[8]
The amount of fibers in each of the various size and type of bars was provided by the manufacturer.[40]

Table 2: Fiber Area of FRP Bars

<table>
<thead>
<tr>
<th>Bar I.D.</th>
<th># Carbon tows</th>
<th># E-Glass Rovings</th>
<th>Area Carbon sq.mm. (sq.in.)</th>
<th>Area E-Glass sq.mm. (sq.in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>H10</td>
<td>9 (T300)</td>
<td>36</td>
<td>4.1 (0.006)</td>
<td>31.5 (0.049)</td>
</tr>
<tr>
<td>H10 *</td>
<td>9 (T700)</td>
<td>36</td>
<td>4.0 (0.006)</td>
<td>31.5 (0.049)</td>
</tr>
<tr>
<td>H19</td>
<td>34 (T300)</td>
<td>136</td>
<td>15.4 (0.024)</td>
<td>118.9 (0.184)</td>
</tr>
<tr>
<td>H19 *</td>
<td>34 (T700)</td>
<td>136</td>
<td>14.9 (0.023)</td>
<td>118.9 (0.184)</td>
</tr>
<tr>
<td>H22 *</td>
<td>44 (T700)</td>
<td>176</td>
<td>19.3 (0.030)</td>
<td>153.8 (0.238)</td>
</tr>
<tr>
<td>C10</td>
<td>42 (T300)</td>
<td>0</td>
<td>19.0 (0.029)</td>
<td>0 (0)</td>
</tr>
<tr>
<td>C13 *</td>
<td>60 (T700)</td>
<td>0</td>
<td>26.4 (0.041)</td>
<td>0 (0)</td>
</tr>
<tr>
<td>C19</td>
<td>154 (T300)</td>
<td>0</td>
<td>69.6 (0.108)</td>
<td>0 (0)</td>
</tr>
<tr>
<td>C19 *</td>
<td>135 (T700)</td>
<td>0</td>
<td>59.3 (0.092)</td>
<td>0 (0)</td>
</tr>
<tr>
<td>C22 *</td>
<td>175 (T700)</td>
<td>0</td>
<td>76.9 (0.119)</td>
<td>0 (0)</td>
</tr>
</tbody>
</table>

Note: "*" indicates that the bar used Type T700 HS Carbon fibers rather than Type T300 HS Carbon fibers.
2.4 Material Composition

The mechanical properties of the individual bars are dependent upon the area of fibers and the ratio of fibers to resin. The manufacturer provided information stating that the volume fraction of fibers used in the NEFMAC grids was approximately forty-six per cent (46%) of the total solids. Material composition tests were conducted upon the two different types of fiber reinforced plastic composites used in this test program to determine their volume and weight fractions. The material composition, or "Burn-Off", tests were conducted using the general procedure outlined in ASTM D3590-90, "Standard Test Method for Volatiles Content of Epoxy Matrix Prepreg"[7]. The material composition tests were conducted using four samples each from two different bar sizes, from both the C-Type and the H-Type FRP reinforcing bars, for a total of 16 test specimens. As a control, tests were simultaneously conducted on samples of the vinyl-ester resin used in the manufacture of the FRP bars.

The material composition tests were conducted in several stages. The first stage of the test was designed to consume the vinyl-ester resin, while leaving the carbon and glass fibers in place. For this portion of the test, the specimens were placed in a desiccator, weighed on an analytical balance, and then placed in a circulating air oven which had been preheated to the initial test
temperature. After a set period of time, the test specimens were removed from the oven, placed in a desiccator, allowed to cool to ambient temperature, and then measured to determine if there was any weight loss. This process was repeated at the initial test temperature until no further weight loss was detected. The temperature of the oven was then incremented by 25°C, and the process repeated until all the resin was volatilized. The resin was found to be completely consumed at 400°C.

The second stage, which involved one-half of the total samples, involved raising the temperature in the oven to 600°C in order to burn off the carbon fibers. It was not necessary to conduct this portion of the test using small increments of temperature since the E-Glass fibers are refractory. Again, after heating the specimens for a set period of time, the specimens were placed in a desiccator, allowed to cool to ambient temperature, and then measured on an analytical balance to determine if there was any weight loss. This process was repeated until all the carbon fibers were consumed. The results of the material composition tests are shown in Table 3. The information that the manufacturer provided on material composition is shown in Table 4.
Table 3: Results of Material Composition Test

<table>
<thead>
<tr>
<th>Bar Type</th>
<th>% Carbon Fiber Volume (Weight)</th>
<th>% Glass Fiber Volume (Weight)</th>
<th>% Resin Volume (Weight)</th>
</tr>
</thead>
<tbody>
<tr>
<td>H-Type</td>
<td>5 (5)</td>
<td>39 (52)</td>
<td>56 (43)</td>
</tr>
<tr>
<td>C-Type</td>
<td>36 (45)</td>
<td>0 (0)</td>
<td>64 (55)</td>
</tr>
</tbody>
</table>

Table 4: Material Composition Information Provided by Manufacturer

<table>
<thead>
<tr>
<th>Bar Type</th>
<th>% Carbon Fiber Volume</th>
<th>% Glass Fiber Volume</th>
<th>% Resin Volume</th>
</tr>
</thead>
<tbody>
<tr>
<td>H-Type</td>
<td>5</td>
<td>41</td>
<td>54</td>
</tr>
<tr>
<td>C-Type</td>
<td>43</td>
<td>0</td>
<td>57</td>
</tr>
</tbody>
</table>

For each bar type, the relative volume fractions of each component were the same for the different bar sizes.

Note that these values for volume fractions are slightly different from those provided by the manufacturer. Two possible explanations for the discrepancy in volume fractions are:

- The vendor is holding the amount of fiber constant while supplying more resin than specified;
- or the vendor neglected the weight of the fiber's protective coatings.

Note that some manufacturers specify materials using "weight fraction" rather than "volume fraction".

The FRP "Burn-Off" test specimens are shown in Photograph 3.
2.5 Theoretical Mechanical Properties of FRP Composites

The mechanical properties of fiber reinforced plastic composites are a resultant of the properties of the individual materials which form the composite. Unlike steel, the properties of fiber reinforced plastic composites may be made to vary significantly, depending upon the exact material composition.

The deformation behavior of FRP composites is described by the following volume-fraction rule which is based upon equal strain in all components parallel to the fibers.[17]

\[ \varepsilon_{\text{composite}} = \varepsilon_1 = \varepsilon_2 = \ldots \]
Where: \( \epsilon_i = \text{strain in component } i = 1, 2, 3 \ldots \)

Based upon this volume-fraction rule, the modulus of elasticity for a FRP composite material with a tensile load applied parallel to the longitudinal axis of the fibers is defined as:[17]

\[
E_{\text{composite}} = V_1E_1 + V_2E_2 + \ldots
\]

Where: \( V_i = \text{volume fraction of component } i \)

(\text{percent of gross cross sectional area})

\( E_i = \text{Modulus of Elasticity of component } i \)

The stresses in FRP composites are a volumetric weighted average of the stresses in the individual components. The tensile stress in a FRP composite material is defined as:[17]

\[
\sigma_{\text{composite}} = V_1\sigma_1 + V_2\sigma_2 + \ldots \quad \sigma < \sigma_{y_s}
\]

Therefore, the modulus of elasticity and tensile stress in a fiber reinforced composite composed of multiple types of fibers in a homogeneous resin matrix is defined as follows:[17]

\[
E_{\text{composite}} = V_{\text{matrix}}E_{\text{matrix}} + V_{\text{fiber1}}E_{\text{fiber1}} + V_{\text{fiber2}}E_{\text{fiber2}} + \ldots
\]

\[
\sigma_{\text{composite}} = V_{\text{matrix}}\sigma_{\text{matrix}} + V_{\text{fiber1}}\sigma_{\text{fiber1}} + V_{\text{fiber2}}\sigma_{\text{fiber2}} + \ldots \quad \sigma < \sigma_{y_s}
\]

The ratio of the forces resisted by one component of the composite compared to that resisted by a second component is \( V_1E_1/V_2E_2 \).

Since the resin matrix used in the production of FRP composite materials have a limited capability to accommodate
strain and an elastic modulus of approximately 2100 MPa (300 ksi)\[10\] while the fibers typically have a modulus of elasticity in the range of 70,000 MPa to 380,000 MPa (10,000 ksi to 55,000 ksi)\[9\][10][40], the contribution of the resin matrix may conservatively be neglected when calculating the stiffness of the composite.\[10\]

For example, for a FRP composite with a volume fraction of 40% glass fiber (with a modulus of elasticity of 70,000 MPa), and 60% plastic resin matrix (with a modulus of elasticity of 2,100 MPa), the modulus elasticity of the composite would be:\[17\]

\[
E_{\text{composite}} = V_{\text{fiber}}E_{\text{fiber}} + V_{\text{matrix}}E_{\text{matrix}}
\]

\[
= (0.40*70,000)+(0.60*2100) = 29,300 \text{ MPa (4250 ksi)}
\]

Neglecting the contribution of the plastic resin matrix:

\[
E'_{\text{composite}} = (0.40*70,000) = 28,000 \text{ MPa (4,100 ksi)}
\]

Thus \( E'_{\text{comp}}/E_{\text{comp}} = 28,000/29,300 = 96\% \)

The contribution of the resin matrix is even further reduced for composites containing the higher stiffness fibers such as the carbon fibers. For example, for a FRP composite with a volume fraction of 40% carbon fiber (with a modulus of elasticity of 235,000 MPa)\[40\], and 60% plastic resin matrix (with a modulus of elasticity of 3,500 MPa), the modulus elasticity of the composite would be:

\[
E_{\text{composite}} = (0.40*235,000)+(0.60*2100) = 95,300 \text{ MPa (13,800 ksi)}
\]
Neglecting the contribution of the plastic resin:

\[ E'_{\text{composite}} = (0.40 \times 235,000) = 94,000 \text{ MPa} \ (13,000 \text{ ksi}) \]

Thus \( E'_{\text{comp}} / E'_{\text{comp}} = 94,000/95,300 = 99\% \)

In addition, since the manufacturing process produces irregular shaped bars, of constant fiber content but varying resin content, it is appropriate to neglect the stiffness contribution of the resin matrix.

The failure stress of the composites (\( \sigma_r \)) is dependant upon the modulus of elasticity and the maximum strain of the fibers which make up the composite.

\[ \sigma_r = E_{\text{composite}} \varepsilon_{\text{composite}} \]

For multiple fiber composites, \( \varepsilon_{\text{composite}} \) will be bounded by the maximum strains of the individual fibers. For example, if \( \varepsilon_{\text{fiber1}} < \varepsilon_{\text{fiber2}} \), then \( \varepsilon_{\text{fiber1}} < \varepsilon_{\text{composite}} < \varepsilon_{\text{fiber2}} \).

<table>
<thead>
<tr>
<th>Bar Type</th>
<th>Calculated Modulus E (MPa (ksi))</th>
<th>Calculated Failure Stress (Glass Fibers Fail) MPa (ksi)</th>
<th>Calculated Failure Stress (Carbon Fibers Fail) MPa (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>H-Type (T300)</td>
<td>40,000 (5800)</td>
<td>800 (5800)</td>
<td>520 (75)</td>
</tr>
<tr>
<td>H-Type (T700)</td>
<td>40,000 (5800)</td>
<td>800 (120)</td>
<td>560 (80)</td>
</tr>
<tr>
<td>C-Type (T300)</td>
<td>82,700 (12,000)</td>
<td>-</td>
<td>1080 (156)</td>
</tr>
<tr>
<td>C-Type (T700)</td>
<td>82,700 (12,000)</td>
<td>-</td>
<td>1160 (170)</td>
</tr>
</tbody>
</table>
2.6 Section Properties

The nominal cross sectional properties were determined by averaging the results from repeated volumetric measurements using samples of each of the different types and sizes of reinforcing bars. The measured cross sectional area was compared to the theoretical nominal cross sectional area based upon the relative volume fractions of resin and fibers. The average cross sectional properties for the various bar sizes are shown in Table 6.

<table>
<thead>
<tr>
<th>Bar Type</th>
<th>Average Depth mm (in.)</th>
<th>Maximum Width mm (in.)</th>
<th>Maximum Depth mm (in.)</th>
<th>Average Area sq.mm. (sq.in.)</th>
<th>Nominal Area sq.mm. (sq.in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>H10</td>
<td>10.0 (0.39)</td>
<td>10 (0.39)</td>
<td>12 (0.47)</td>
<td>81 (0.126)</td>
<td>80 (0.124)</td>
</tr>
<tr>
<td>H19</td>
<td>20.0 (0.79)</td>
<td>20.0 (0.79)</td>
<td>25 (1.00)</td>
<td>360 (0.558)</td>
<td>300 (0.465)</td>
</tr>
<tr>
<td>H22</td>
<td>26.0 (1.02)</td>
<td>26.0 (1.02)</td>
<td>32 (1.26)</td>
<td>480 (0.744)</td>
<td>400 (0.620)</td>
</tr>
<tr>
<td>C10</td>
<td>8.0 (0.32)</td>
<td>8.0 (0.32)</td>
<td>10 (0.39)</td>
<td>56 (0.087)</td>
<td>52 (0.081)</td>
</tr>
<tr>
<td>C13</td>
<td>9.0 (0.35)</td>
<td>9.0 (0.35)</td>
<td>11 (0.43)</td>
<td>78 (0.121)</td>
<td>72 (0.112)</td>
</tr>
<tr>
<td>C19</td>
<td>12.5 (0.49)</td>
<td>12.5 (0.49)</td>
<td>18 (0.71)</td>
<td>165 (0.256)</td>
<td>160 (0.248)</td>
</tr>
<tr>
<td>C22</td>
<td>21.0 (0.83)</td>
<td>21.0 (0.83)</td>
<td>26 (1.02)</td>
<td>220 (0.341)</td>
<td>210 (0.326)</td>
</tr>
</tbody>
</table>

Note: The maximum width is equal to the average depth.
There are measurable differences in dimensions between bars of the same type and size. For example, in the C19 reinforcing bars (shown in Figure 2), the depth varied from 10 mm to 18 mm, averaging 12.5 mm. Photograph 4 shows sample cross-sections from NEFMAC FRP bars. Note the presence of voids and the uneven distribution of fibers caused by the manufacturing process.

**NEFMAC C19 FRP BARS**

**Typical Cross Sections**

![Diagram of NEFMAC C19 FRP BARS](image)

**Nominal Height Section**

**Maximum Height Section**

*Figure 2 - Typical FRP Bar*
Photograph 4 - Cross Sections of FRP Bars

In addition, note that the carbon fibers are not uniformly distributed throughout the H-Type bars. The fabrication process creates bundles of carbon fibers surrounded by bundles of glass fibers. The two different fiber types are not intermixed.

A comparison of measured area to the theoretical area indicates that the H19 type bars contains voids adding up to an additional 15% in the apparent cross sectional area. The H22 type bars contains voids adding up to an additional 25% in the apparent cross sectional area.
2.7 Experimental Determination of Mechanical Properties

The mechanical properties of the FRP bars which make up the NEFMAC reinforcing grids were determined using axial tensile tests. The first series of tensile tests were conducted using bars with a 50 mm (2") long "dumbbell-shaped" milled section, having a nominal 6 mm x 12 mm (0.25" x 0.5") test region. The second series of tensile tests were conducted using the full cross section of the FRP bars as per ASTM D3039. The results of these two test series were measurably different.

The milled section test coupons were fabricated by grinding a portion of the FRP bar to reduce the cross section to 6mm x 12mm for a length of approximately 50mm. The ends of the reduced section were gradually tapered back to the full cross section. These specimens were from supported in the test machine using serrated grips. During testing there was noticeable evidence of slippage between the grips and the test specimens and evidence of the grips crushing the FRP bars. In order to test the full cross section without having the test machine grips damage the FRP bars, it was necessary to cast the ends of test specimens into 0.3m long steel pipes using a high strength cement grout. The steel pipes were placed in the test machine and supported by 50mm steel plates. The test setups are shown in Figure 3.

The strains in the reinforcing material were recorded
Figure 3 - FRP Tension Test Setup

by using an extensometer and electric resistance strain
gages placed upon opposing faces of the test coupons. The
measured strains in each gage were averaged to determine the
modulus of elasticity for the sample.

The mechanical properties obtained from the coupon
tests are shown in Table 7.
Table 7: Mechanical Properties of NEFMAC Reinforcing Grids (from tests on coupons with 6mm x 12mm milled cross-section)

<table>
<thead>
<tr>
<th>Bar Type</th>
<th>E (MPa) (ksi)</th>
<th>F_{ult} (MPa) (ksi)</th>
<th>(\varepsilon_{ult}) Ulit. Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>H-Type</td>
<td>34500 (5000)</td>
<td>550 (80)</td>
<td>0.018</td>
</tr>
<tr>
<td>C-Type</td>
<td>73000 (10,600)</td>
<td>1030 (150)</td>
<td>0.014</td>
</tr>
</tbody>
</table>

In contrast, the material properties obtained from tests conducted upon the un-milled full cross-section specimens are as follows:

Table 8: Mechanical Properties of NEFMAC Reinforcing Grids (from tests on full cross-section bars)

<table>
<thead>
<tr>
<th>Bar Type</th>
<th>P_{ult} (kN) (Kips)</th>
<th>F_{ult} (MPa) (ksi)</th>
<th>E (MPa) (ksi)</th>
<th>Rigidity (A*E) (kN) (kips)</th>
<th>(\varepsilon_{ult}) Ulit. Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>H19</td>
<td>205 (46)</td>
<td>680 (99)</td>
<td>41,500 (6080)</td>
<td>12,600 (2830)</td>
<td>0.016</td>
</tr>
<tr>
<td>H22</td>
<td>254 (57)</td>
<td>630 (92)</td>
<td>41,200 (5985)</td>
<td>16,500 (3710)</td>
<td>0.015</td>
</tr>
<tr>
<td>C13</td>
<td>106 (23.8)</td>
<td>1470 (210)</td>
<td>89,500 (13,000)</td>
<td>6440 (1450)</td>
<td>0.016</td>
</tr>
<tr>
<td>C19</td>
<td>205 (46)</td>
<td>1280 (185)</td>
<td>85,300 (12,400)</td>
<td>13,600 (3070)</td>
<td>0.014</td>
</tr>
<tr>
<td>C22</td>
<td>258 (58)</td>
<td>1230 (178)</td>
<td>85,200 (12,300)</td>
<td>17,900 (4030)</td>
<td>0.014</td>
</tr>
</tbody>
</table>

Notes:
1.) H10 and C10 bars were unavailable for testing.
2.) Calculations were based upon the theoretical nominal area from Table 6.
The results of tension tests on the full cross-section specimens indicated that while the modulus of elasticity for each bar type was relatively constant, there was a measurable decrease in ultimate stress and ultimate strain with increasing bar size (reference Figure 4 and Figure 5).

The decrease in ultimate strain with increasing bar size is possibly due to the greater probability of defects in the larger bars sizes, and the "shear lag" effects which take place in the transfer of stresses between the resin matrix and adjacent fibers. These effects result in the outer fibers of the member being subjected to higher stresses than the inner fibers.

As shown in Photograph 5, the bars did not fail along a "failure plane", but rather at random locations along the test specimen, exhibiting the characteristic behavior of continuous fiber reinforced composites.[17][27]
Photograph 5 - FRP Tension Test Specimens
Figure 4 - Stress Vs. Strain, C-Type FRP Bars

Figure 5 - Stress Vs. Strain, H-Type FRP Bars
The tests on the full cross-section specimens resulted in higher values for ultimate stress and modulus of elasticity than the tests on the milled coupons. There are several possible explanations for the differences between the two series of tests:

- The distribution of fibers across the FRP bars is not controlled. The milled sections might contain proportionately less fibers than a full bar.

- The measured cross sectional properties of the "dumbbell-shaped" coupons do not take into account any voids present in the specimen, in effect, overestimating the area of solids.

- There is a possibility that fibers were cut or damaged during the machining process.

Note that ASTM D638-90 recommends that milled "dumbbell-shaped" coupons be used only for NON-reinforced plastics. Fiber Reinforced Composites are normally tested using procedures similar to ASTM D3039.
2.6 Calculated Vs. Measured Properties

A comparison between the calculated and measured mechanical properties is shown in the following table. Not all bar sizes were available for testing.

<table>
<thead>
<tr>
<th>Bar Type</th>
<th>Cal'd AE (kips)</th>
<th>MFR AE (kips)</th>
<th>MFR/Calc %</th>
<th>Measured AE (kips)</th>
<th>Meas/Calc %</th>
</tr>
</thead>
<tbody>
<tr>
<td>H10</td>
<td>3,220 (724)</td>
<td>3,250 (730)</td>
<td>100.8%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>H10 *</td>
<td>3,200 (718)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H19</td>
<td>12,170 (2735)</td>
<td>12,260 (2756)</td>
<td>100.7%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>H19 *</td>
<td>12,070 (2713)</td>
<td>12,150 (2731)</td>
<td>100.7%</td>
<td>12,600 (2830)</td>
<td>104.2%</td>
</tr>
<tr>
<td>H22 *</td>
<td>15,630 (3511)</td>
<td>15,700 (3527)</td>
<td>100.5%</td>
<td>16,500 (3710)</td>
<td>105.7%</td>
</tr>
<tr>
<td>C10</td>
<td>4,380 (983)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C13 *</td>
<td>6,080 (1366)</td>
<td>6220 (1398)</td>
<td>102.3%</td>
<td>6,440 (1450)</td>
<td>106.1%</td>
</tr>
<tr>
<td>C19</td>
<td>16,050 (3606)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C19 *</td>
<td>13,680 (3074)</td>
<td>14,000 (3146)</td>
<td>102.3%</td>
<td>13,600 (3070)</td>
<td>99.9%</td>
</tr>
<tr>
<td>C22 *</td>
<td>17,730 (3,985)</td>
<td>18,150 (4078)</td>
<td>102.3%</td>
<td>17,900 (4020)</td>
<td>101.0%</td>
</tr>
</tbody>
</table>

Note: "*" indicates that the bar used Type T700 HS Carbon fibers rather than type T300 carbon fibers.
2.9 Nominal Net Area Vs. Theoretical Fiber Area

Due to the irregular nature of the NEFMAC FRP bars, designers may wish to consider using the area of the reinforcing fibers rather than the nominal composite or net area of the FRP bars in calculating reinforcement requirements. As shown previously, this assumption will have a negligible effect upon bar rigidity calculations since the fibers contribute the predominate portion of the FRP bar’s stiffness. Although the effective area of the bar will be considerably less than the nominal area (Table 6), the effective modulus of elasticity will be proportionately higher than the nominal modulus of elasticity (Table 8).

For example, for a FRP bar with a volume fraction of 40% carbon fibers will have an effective modulus of elasticity which is $1/40\% = 250\%$ greater than the nominal modulus of elasticity, and an effective area which is 40% of the nominal area.

Modulus:  $E_{\text{effective}} = E_{\text{fiber}} = 235,000 \text{ MPa}$

Area:  $A_{\text{effective}} = V_{\text{fiber}}A_{\text{composite}} = 0.40A_{\text{composite}}$

Rigidity:  $A_{\text{effective}}E_{\text{effective}} = (V_{\text{fiber}}A_{\text{composite}})(E_{\text{fiber}})$

$= (V_{\text{fiber}}E_{\text{fiber}})(A_{\text{composite}})$

$= E_{\text{composite}}A_{\text{composite}}$

The effective modulus of elasticity for reinforcements composed of multiple fiber types must be calculated using the volume fraction components of each fiber type. For example, for a FRP bar with both carbon fibers and E-Glass
fibers, the modulus of elasticity would be calculated as follows:

Modulus: \( E_{\text{effective}} = \frac{V_{\text{fiber1}}E_{\text{fiber1}} + V_{\text{fiber2}}E_{\text{fiber2}}}{V_{\text{fiber1}} + V_{\text{fiber2}}} \)

Area: \( A_{\text{effective}} = A_{\text{fiber1}} + A_{\text{fiber2}} \)

The FRP reinforcement mechanical properties calculated from the theoretical fiber areas compare favorably with those obtained experimentally from the tests on the full cross-section specimens. The properties calculated from the theoretical fiber areas are shown in the following table:

<table>
<thead>
<tr>
<th>Bar Type</th>
<th>Carbon Fiber Area (sq.mm.)(sq.in.)</th>
<th>Glass Fiber Area (sq.mm.)(sq.in.)</th>
<th>Total Fiber Area (sq.mm.)(sq.in.)</th>
<th>( E ) (M Pa)(ksi)</th>
<th>( (A*E) ) Rigidity (kN)(kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>H10</td>
<td>4.1 (0.006)</td>
<td>31.5 (0.049)</td>
<td>35.6 (0.055)</td>
<td>90,100 (13,100)</td>
<td>3200 (720)</td>
</tr>
<tr>
<td>H19</td>
<td>15.0 (0.023)</td>
<td>118.9 (0.184)</td>
<td>133.9 (0.208)</td>
<td>90,100 (13,100)</td>
<td>12,060 (2725)</td>
</tr>
<tr>
<td>H22</td>
<td>19.3 (0.030)</td>
<td>153.8 (0.238)</td>
<td>173.1 (0.268)</td>
<td>90,100 (13,100)</td>
<td>15,600 (3510)</td>
</tr>
<tr>
<td>C10</td>
<td>19.0 (0.029)</td>
<td>0 (0)</td>
<td>19.0 (0.029)</td>
<td>230,500 (33,400)</td>
<td>4380 (980)</td>
</tr>
<tr>
<td>C13</td>
<td>26.4 (0.041)</td>
<td>0 (0)</td>
<td>26.4 (0.041)</td>
<td>230,500 (33,400)</td>
<td>6000 (1340)</td>
</tr>
<tr>
<td>C19</td>
<td>59.3 (0.092)</td>
<td>0 (0)</td>
<td>59.3 (0.092)</td>
<td>230,500 (33,400)</td>
<td>13,700 (3070)</td>
</tr>
<tr>
<td>C22</td>
<td>76.9 (0.119)</td>
<td>0 (0)</td>
<td>76.9 (0.119)</td>
<td>230,500 (33,400)</td>
<td>17,700 (3980)</td>
</tr>
</tbody>
</table>

Note: Type T700 Carbon Fibers
2.10 Discussion

Due to the differences between the different bar sizes as measured in the tension tests on the full size FRP bars, the design value for ultimate stress should be less than the ultimate stress of the individual fibers which comprise the FRP. The tension tests on the full cross-section FRP bars indicated that the larger FRP sections had measurably lower values for ultimate stress and ultimate strain compared to the smaller sized FRP bars. These tests showed that the modulus of elasticity may also decreased as the FRP bar dimensions increased in size.

In addition, the net area and maximum depth should be used in calculating the amount of concrete replaced by each different FRP bar size. This last assumption will be of utmost significance in splice and development length calculations.
2.11 Comparison to Steel Reinforcements

The modulus of elasticity of the NEFMAC FRP composites is significantly less than that of steel. As shown in Figure 6, based upon the nominal cross-section and the volume fraction of fibers present, the modulus for the NEFMAC H-Type Material is approximately 1/6 of that steel, while the modulus for NEFMAC C-Type material is approximately 1/3 that of steel.

Figure 6 - Normalized Stress Vs. Strain, Comparison Between Steel and FRP Reinforcements (Test Results)
2.12 Key Differences Between FRP and Steel

There are three key differences between the material properties of FRP grids and Steel reinforcing bars:

- The Stress Vs. Strain diagrams for the FRP materials is essentially linear up to ultimate with no discernable yield point.
- The failure stress of the FRP materials is significantly higher than the yield stress of steel.
- The modulus of elasticity for FRP is a less than a third of that for steel.

As a result of these differences, the concrete structure with FRP reinforcements can be expected to exhibit significantly greater deflections than steel reinforced concrete structures with the same reinforcing ratio. In addition, for structures with equal stiffness reinforcements, the structure reinforced with FRP may have a considerably greater ultimate capacity than the steel reinforced structure. It is most probable that the failure mode of the FRP reinforced structure will be a diagonal shear-tension failure rather than a failure in which the reinforcement fractures.

In summary, on a equal-area basis, FRP reinforcements are lighter, stronger, but less stiff than steel. On an equal-weight basis, FRP reinforcements can be made stiffer and stronger than steel, since the FRP materials weigh considerably less per unit volume than steel. Since the FRP composite materials have a lower modulus of elasticity than
steel, but have an equal or higher failure strength, the
design of concrete structures reinforced with FRP composites
will typically be governed by shear strength or deflection
limitations, not strength requirements.
CHAPTER III
DESIGN CONSIDERATIONS

3.0 Introduction
This chapter presents the design considerations for the use of FRP reinforcements.

3.1 Design Assumptions
The following assumptions are used in the design of concrete structures reinforced with FRP grids:

- Static equilibrium and strain compatibility are satisfied. The compressive and tensile forces are in equilibrium and there is no slip between the concrete and the reinforcements.

- The maximum usable compressive strain for concrete is assumed to be equal to $\varepsilon_{cu} = 0.003$.

The tensile strength of concrete is neglected in flexural calculations.

- In plastic flexural theory (ultimate strength design) the stresses in the concrete and reinforcement are computed from their respective stress-strain curves.

- In elastic flexural theory (working stress design) it is assumed that stresses in the concrete and in the reinforcement are proportional to strain. Due to the inelastic stress-strain relationship of concrete, this assumption is accurate only for concrete compressive stresses below approximately one-half $f'_c$.

- Plane sections normal to the axis of bending remain plane after bending and the variation in strain is linear across the depth of the member. This assumption
is only an approximation of the actual conditions within a reinforced concrete structure.

**Stress-Strain Behavior of Steel Reinforcement**

The stress in steel reinforcement ($f_r$) stressed below the yield strength $f_y$ is taken as the modulus ($E_s$) times the steel strain ($f_r = E_s \varepsilon_s$). For strains greater than $f_y/E_s$, stress in steel reinforcement is taken to be $f_y$.

**Stress-Strain Behavior of FRP Reinforcement**

For FRP reinforcement, which has no clearly define yield point, the stress in the FRP reinforcement, $f_r$, is taken as the modulus ($E_r$) times the FRP strain ($f_r = E_r \varepsilon_r$).

**Stress-Strain Behavior of Concrete**

The non-linear stress-strain behavior of concrete is shown in Figure 7. The shape of the stress-strain curve for concrete is a function of the concrete compressive strength.
Note that the stress-strain curves become non-linear after approximately 0.5f', the maximum stress occurs at a compressive strain between 0.0015 and 0.002, and that the ultimate strain generally occurs after a compressive strain of 0.003. ACI-318-89 recommends a value of 0.003 for maximum compressive strain.
Stress Distribution in Reinforced Concrete

The stress and strain distributions in a reinforced concrete member are shown in Figure 8.[48] At ultimate capacity, the stress distribution in the concrete conforms to the parabolic stress-strain curves shown in Figure 7.

Figure 8 - Theoretical Stress and Strain Distributions

Where:

- $k_1, k_2, k_3 =$ Shape factors which describe the approximate parabolic stress distribution.
- $k_1 k_3 f'_c =$ average stress in concrete
- $k_2 c =$ depth of centroid of stress distribution
- $k_3 f'_c =$ maximum stress in concrete
- $f'_c =$ specified compressive strength of concrete
- $f_t =$ stress in FRP reinforcement
- $f_s =$ stress in steel reinforcement
\( E_f \) = modulus of elasticity of FRP reinforcement
\( E_s \) = modulus of elasticity of steel reinforcement
\( \epsilon_c \) = compressive strain in concrete, maximum usable value is assumed by ACI-318 to be 0.003
\( \epsilon_f \) = strain in tension FRP reinforcement, \((f_f/E_f)\)
\( \epsilon_s \) = strain in tension steel reinforcement, \((f_s/E_s)\)

**Stress Distribution - Elastic Theory**

For concrete compressive stresses less than 0.45\( f'_c \), ACI 318-89[2] permits the stress-strain distribution to be modeled using the elastic stress-strain distribution shown in Figure 9. This model is used to determine service load deflections and stresses. It may not be used to determine the ultimate capacity of reinforced concrete members.

**Figure 9 - Elastic Theory**
Stress Distribution - Equivalent Rectangular Stress Block Theory

The current ACI 318 [2] and PCA[48] standards model the actual stress-strain distribution by an equivalent rectangular stress block. This model is used to determine the ultimate design capacity of a reinforced concrete member.

Figure 10 - Equivalent Rectangular Stress Block Stress and Strain Distribution

Where:

\[ a = \text{depth of equivalent rectangular stress block} \]

\[ \beta = \text{ratio of depth of equivalent rectangular stress block to depth of neutral axis} \]
3.2 Flexural Capacity

Based upon the equivalent rectangular stress block, the flexural capacity of a singly reinforced concrete member with no shear reinforcement is governed by.

- Failure of the tension reinforcement, in which case the member is referred to as an "under-reinforced section". In steel reinforced concrete the reinforcement gradually yields. In contrast, for FRP reinforced concrete the reinforcement would fracture.

- Compression failure in the concrete, in which case the member is referred to as an "over-reinforced section"

- Simultaneous failure in both the concrete and reinforcement, in which case the member is referred to as a "balanced section" in that the strains in the concrete and reinforcement are balanced. This is also referred to as a "balanced strain" condition.

In addition, the member must be analyzed to determine if the flexural capacity is limited by the allowable shear capacity.
3.3 Balanced Strain Condition

Based upon plane sections remaining plane after bending and stress and strain compatibility, the following equation is used to calculate the reinforcement ratio, $\rho_b$, which produces a balanced strain condition in the equivalent rectangular stress block model:

$$\rho_b = \frac{0.85 \beta_1 f'/c}{f_{\text{reinf}}} \left( \frac{\epsilon_{cu}}{\epsilon_{cu} + \epsilon_{\text{reinf}}} \right)$$

(1)

Where:

- $\rho$ = Area of reinforcing divided by area of concrete
- $\beta_1$ = ratio of depth of equivalent rectangular stress block to depth of neutral axis
- $E_{\text{reinf}}$ = modulus of elasticity reinforcement
- $f'_c$ = specified compressive strength of concrete
- $f_{\text{reinf}}$ = failure or yield strength of reinforcement
- $\epsilon_{cu}$ = maximum compressive strain in concrete, $\epsilon_{cu}$ is assumed by ACI-318-89 Section 10.3 to be 0.003
- $\epsilon_{\text{reinf}}$ = failure or yield strain in tension reinforcement ($f_{\text{reinf}}/E_{\text{reinf}}$)

Substituting 0.003 for $\epsilon_{cu}$ and multiplying through by the modulus of elasticity of the reinforcement ($E_{\text{reinf}}$), equation (1) then becomes:

$$\rho_b = \frac{0.85 \beta_1 f'/c}{f_{\text{reinf}}} \left( \frac{0.003 E_{\text{reinf}}}{(0.003 E_{\text{reinf}}) + f_{\text{reinf}}} \right)$$

(2)
For steel reinforcement with $E_s = 29,000,000$ psi, this equation becomes:

$$
\rho_b = \frac{0.85 \beta_1 f'c}{f_y} \left( \frac{87000}{87000 + f_y} \right)
$$

Where:

$f_y$ = yield stress in steel reinforcement

$\epsilon_y$ = yield strain in tension steel FRP reinforcement

For C-Type FRP reinforcement with $E_f \approx 12,000,000$ psi, the equation becomes:

$$
\rho_b = \frac{0.85 \beta_1 f'c}{f_u} \left( \frac{36000}{36000 + f_u} \right)
$$

Where:

$f_u$ = failure stress in FRP reinforcement

$\epsilon_u$ = failure strain in tension FRP reinforcement

Similarly, for H-Type FRP reinforcement with $E_f \approx 6,000,000$ psi, the equation becomes:

$$
\rho_b = \frac{0.85 \beta_1 f'c}{f_u} \left( \frac{18000}{18000 + f_u} \right)
$$

Note that these equations for calculating $\rho_b$ assume that plane sections remain plane during bending, which is only an approximation of the actual conditions.

It should also be noted that all flexural failures using FRP reinforcements are anticipated to be brittle failures, since the FRP does not yield prior to failure.
3.4 Moment of Inertia

For singly reinforced concrete members with low reinforcing ratios, in which the tension reinforcement fails before concrete crushing occurs ($\rho < \rho_b$), the neutral axis of the transformed cracked section is located using the following equation.

$$k = \sqrt{2\rho n + (\rho n)^2} - \rho n$$  \hfill (3)

This equation is not appropriate for use in members in which the tension reinforcement remains elastic up to concrete failure, such as "over reinforced" steel reinforced concrete ($\rho > \rho_b$) or for FRP reinforced concrete. For the condition of concrete compression failure, the depth of the neutral axis ($kd = c$) must be calculated based upon the strain in the concrete reaching $\epsilon_{cu}$ while the stress in the reinforcing ($f_{rcm}$) is less than failure stress ($f_{rein} < f_{fail}$). The formula for determining the depth of the neutral axis of a singly reinforced concrete member in which the tension reinforcement does not fail prior to concrete failure is derived as follows:

From equilibrium of forces in the equivalent rectangular stress block:

$$C = T$$

$$b(0.85f'c)(\beta_i kd) = A_{rein}f_{rein} = (\rho bd)(E_{rein}\epsilon_{rein})$$

$$0.85f'c\beta_i k = \rho E_{rein}\epsilon_{rein}$$

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From compatibility of strains and the assumption that plane sections remain plane:

\[ \varepsilon_{reinf} = \varepsilon_{cu} \left( \frac{d - kd}{kd} \right) \]  

(4)

Which yields:  

\[ 0.85f'c\beta_1k^2 = \rho E_r\varepsilon_{cu} - k\rho E_r\varepsilon_{cu} \]

Solving for \( k \) (\( k = c/d \) = the ratio of depth of neutral axis to total section depth), the formula for locating the neutral axis is shown in equation (5).

\[
k = \left( \frac{1 + 4\beta_1 \left( \frac{0.85f'c}{\rho E_r\varepsilon_{cu}} \right) - 1}{2\beta_1 \left( \frac{0.85f'c}{\rho E_r\varepsilon_{cu}} \right)} \right)
\]

(5)

After calculating the location of the neutral axis (kd), the moment of inertia of the cracked transformed section, \( I_{cr} \), may be determined using equation (6).

\[
I_{cr} = \frac{b(kd)^3}{3} + nA(d-kd)^2
\]

(6)

Where:

- \( A \) = Area of reinforcement
- \( n \) = ratio of modulus of elasticity of reinforcement to modulus of elasticity of concrete

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The effective moment of inertia \( (I_e) \) may then be calculated using the equation proposed by Branson in 1963.[13] This equation provides a transition from the section properties of the uncracked section to those of the fully cracked section.

\[
I_e = \left( \frac{M_{cr}}{M_a} \right)^m I_g + \left[ 1 - \left( \frac{M_{cr}}{M_a} \right)^m \right] I_{cr}
\]  

(7)

Where:

- \( I_e \) = effective moment of inertia
- \( f_r \) = modulus of rupture = \( 7.5 \sqrt{f'c} \)
- \( y_t \) = distance from neutral axis of uncracked section to extreme tension fiber
- \( I_g \) = gross moment of inertia of uncracked section
- \( I_{cr} \) = cracked transformed section moment of inertia
- \( M_{cr} \) = \( f_r I_g / y_t \) = cracking moment
- \( M_a \) = applied moment
- \( m \) = empirical transition constant, derived from test data

Based upon statistical studies of steel reinforced concrete beams, ACI has incorporated the Branson equation into the ACI Building Code 318-89 as Equation ACI 9-7, using a value of 3 for the transition constant "m".

As noted by Nayy in 1977[41], due to the low modulus of elasticity of the FRP reinforcements relative to the modulus of steel reinforcements, the Branson equation overestimates \( I_e \) for FRP reinforcements. The European Concrete Committee
(CEB) takes a different approach than ACI in calculating deflections, using the following bilinear equations to calculate deflections. [13]

\[ \Delta = \frac{KL^2}{E_c} \left[ \frac{M_a}{I_g} \right] \]  \hspace{1cm} (for \( M_a < M_{cr} \))  \hspace{1cm} (8)

and also

\[ \Delta = \frac{KL^2}{E_c} \left[ \frac{M_{cr}}{I_g} + \frac{M_a - M_{cr}}{0.85 I_{cr}} \right] \]  \hspace{1cm} (for \( M_a > M_{cr} \))  \hspace{1cm} (9)

Where:

\[ \Delta \]  =  Deflection at point of interest.

\[ K \]  =  A deflection coefficient which depends upon the load distribution, support conditions, and point of interest.

For example, to find the midspan deflection in a simply supported beam with a single concentrated load at midspan, \( K = 1/24 \) and \( M_a = PL/2 \).

The bilinear formula provides an abrupt transition between the uncracked and the fully cracked section. Note that for applied moments greater than \( M_{cr} \), if larger values for "m" are used in the Branson equation, then the calculated deflections converge to those obtained using the CEB method. A comparison between the ACI and the CEB methods is shown in Figure 11.
Figure 11 - Predicted and Measured Load Vs. Deflection for a FRP Reinforced Beam (T-Series)
3.5 Flexural Capacity - Tension Failure

For steel reinforced concrete, the nominal flexural capacity of a singly reinforced concrete member is governed by failure of the tension reinforcement when the reinforcement ratio is less than that required for a balanced strain condition ($\rho < \rho_b$). The flexural capacity ($M_n$) of a reinforced concrete member when tension capacity governs is shown in equation (10).

$$M_n = A_t f_t \left( d - \frac{a}{2} \right)$$

(10)

$$a = \frac{A_t f_t}{0.85 b f'c}$$

The moment capacity is then multiplied by a strength reduction factor to obtain the allowable design moment, $M_{\text{design}} = \phi M_n$, where $\phi = 0.90$ for steel reinforcements[2]. The appropriate strength reduction values for FRP reinforced concrete have not yet been established.

Tests were conducted on FRP reinforced concrete beams with reinforcement ratios ranging from 0.29% to 2.2%. No FRP failures were observed, the beams all failed in concrete compression or shear. Due to the low modulus of elasticity and high strength of FRP reinforcements, practical reinforcing ratios appear to always result in concrete failure.
3.6 Flexural Capacity - Compression Failure

The nominal flexural capacity of a singly reinforced concrete member is governed by compression failure of the concrete when the reinforcement capacity is greater than that of the concrete. This results in a condition where \( \varepsilon_{\text{concrete}} = \varepsilon_{\text{cu}} \) and \( \varepsilon_{\text{reinf.}} < \varepsilon_{\text{fail}} \). The moment capacity for this situation is shown in equation (11).

\[
M_n = 0.85 f'_c a b \left(d - \frac{a}{2}\right)
\]  

(11)

Where \( a = \beta, k d \), as calculated in Equation (5).

The design moment is obtained by multiplying the moment capacity by a strength reduction factor \( M_{\text{design}} = \phi M_n \).\cite{2}\cite{38}
3.7 Shear Capacity

The shear capacity \( V_c \) of singly reinforced concrete members without shear reinforcement is given by ACI 318-89 11.3 approximately as,[2]

\[
V_c = 2bd\sqrt{f'c}
\]  \hspace{1cm} (12)

An alternate formula, based upon statistical work done by Zsutty, includes the effects of \( f'c \), \( \rho \), and the shear span to depth ratio \( (a/d) \), where \( a = \) shear span:[63]

\[
V_c = 59bd\left(f'c \rho \frac{d}{a}\right)^{1/3}
\]  \hspace{1cm} (13)

Finally, for rectangular beams with very low reinforcing ratios, Rajagopalan and Ferguson have proposed the following equation.[49] Unlike ACI Equation 11.3, this equation accurately predicts the shear capacity for beams with reinforcing ratios less than 1%.

\[
V_c = (0.8 + 100\rho)bd\sqrt{f'c}
\]  \hspace{1cm} (14)

Using the shear span length as the moment arm and the minimum value for \( V_c \) from the preceding equations, the moment capacity based upon shear limitations is calculated as:

\[
M_n = aV_c
\]
The design moment is calculated by multiplying the moment capacity by a strength reduction factor, \( M_{\text{design}} = \phi M_u \). The ACI code has selected \( \phi = 0.85 \) for the strength reduction factor for shear and torsion.[2] It has not been established that this strength reduction factor is appropriate for use in FRP reinforced concrete.

### 3.8 Design Moment Capacity

The design moment of the member is taken as the minimum of that obtained from the tensile reinforcement eq.(10), compressive concrete eq.(11), and shear capacity equations (12)(13) or (14):

\[
M_{\text{design}} \leq \phi_{\text{reinf.}} M_u \quad \text{(FRP reinforcement)} \\
M_{\text{design}} \leq \phi_{\text{comp.}} M_u \quad \text{(conc. compression)} \\
M_{\text{design}} \leq \phi_{\text{shear}} M_u \quad \text{(shear)}
\]

The strength reduction factors \( \phi_{\text{reinf.}}, \phi_{\text{comp.}}, \) and \( \phi_{\text{shear}} \) have not yet been determined for FRP reinforced concrete.

The design process could also utilize the "Alternate Design Method" outlined in Appendix A of ACI 318-89. This design process uses the Elastic stress distribution theory for analyzing the concrete stresses, which are then limited to \( 0.45f'_c \). The actual loads, not factored loads, are used in this situation.
3.9 Anchorage Failure

Due to the relatively large cross section of FRP reinforcement compared to steel reinforcement, the concrete structures reinforced with FRP must be analyzed for anchorage failure. To prevent this type of failure, the transverse force in the FRP reinforcement must not exceed the force which can be transmitted by shear to the concrete directly above the reinforcement.

After the onset of cracks due to flexural stresses, the equilibrium equations for a section of the beam between two adjacent cracks may be expressed as follows:

\[ T = \frac{M}{jd} \]
\[ T + \Delta T = \frac{(M + \Delta M)}{jd} \]
\[ \Delta T = \frac{\Delta M}{jd} \]

Equating moments,
\[ \Delta M = V\Delta x \]
and therefore \[ \Delta T = \frac{V\Delta x}{jd} \]

Where: \( j = \) distance from centroid of concrete compression to centroid of tensile reinforcement.

Since the cracks develop at the location of the transverse bars, \( \Delta x = s \)

And therefore \[ \Delta T = \frac{Vs}{jd} \]

This change in force in the reinforcement must be
resisted by the concrete directly above the FRP
reinforcement. Due to the width of the longitudinal and
transverse FRP bars which make up the grid, the net area of
concrete available to resist the shearing forces across this
plane is considerably less than the gross area of concrete
at the plane.

The shearing stress in the plane of concrete directly
above the reinforcement is calculated as.[35]

\[ \nu = \frac{\Delta T}{\sum A_c} = \frac{V S}{Jd(NA_c)} \leq 3.5\sqrt{f'_c} \]

Where:  \( N = \) Number of longitudinal bars (number of grids
in width)

\( A_c = \) The area of concrete enclosed by each FRP
grid.

\( A_c = (S_{long} - w_{long}) \times (S_{trans} - w_{trans}) \)

\( S = \) transverse (longitudinal) bar spacing

\( w = \) width of transverse (longitudinal) bar

Assuming the bar width to be equal to the average bar
depth (\( d_{av}\)) and equal bar size longitudinally and
transversely with equal space longitudinally and
transversely, \( A_c \) can be written as:

\[ A_c = (S - d_{av})^2 \]

The equation for calculating the shearing stress in the
plane of concrete directly above the reinforcement becomes:

\[ \nu = \frac{V S}{Jd N(S-d_{av})^2} \leq 3.5\sqrt{f'_c} \]
3.10 Summary of Design Considerations

Sample calculations based upon these design considerations are presented in the Appendix. These calculations show that, in general, anchorage failure will not govern the capacity of a FRP reinforced member. If the area of concrete resisting shear is reduced, due to the presence of voids or improperly placed reinforcements, \( A_s \) may be reduced such that anchorage failure will result.

The design moment capacity of a member is taken as the minimum of that obtained from the reinforcement failure, concrete compression, and shear capacity equations. In theory, reinforcement failure will govern only in situations where the reinforcing ratio is extremely low. In actual practice, such low reinforcing ratios are not practical. Deflections, concrete compression or shear capacity will generally govern the design of FRP reinforced concrete.

Due to the relatively low modulus of elasticity of the reinforcement in FRP reinforced structures, the transition from the uncracked section to the fully cracked section is more abrupt than in steel reinforced structures. It is anticipated that in the design of FRP reinforced beams deflection control may be the limiting factor rather than load control. The European Concrete Committee (CEB) bilinear formula provides an alternative to the ACI 318 approach to deflection calculations.
CHAPTER IV

EXPERIMENTAL RESULTS

4.0 Introduction

The testing portion of this research program was conducted in several stages: The first section focused upon the monotonic testing of thin FRP grid reinforced concrete beams. The next section deals with flexural testing of beams with low reinforcing ratios. The third section deals with FRP reinforced concrete beams subjected to cyclical loads. The fourth section discusses with the testing of full scale bridge deck slabs reinforced with FRP grids. This is followed by results of the splice and development length tests. The final section summarizes the results of testing.

4.1 Monotonic Testing of FRP Grid Reinforced Concrete Beams

Test Objective

The primary objective of this portion of the test program was to determine the flexural behavior characteristics of concrete beams reinforced with a two dimensional FRP grid and to determine the potential for using the shear and flexural provisions of the ACI code for designing FRP reinforced concrete structures. In addition, these tests were intended to determine if the FRP grids experienced the development and bond problems associated
encountered with round FRP reinforcing bars.[19][20][32]

Test Specimens

In order to study the flexural behavior of concrete beams reinforced with a FRP 2-D grid, the position and the reinforcement ratio were varied while all other material properties were held constant. Eight concrete beams were cast. The dimensions of each beam were 2.8 m (6' - 0") long with a cross section 100 mm (4") deep and 200 mm (8") wide cross section (refer to Figure 13). Two control were cast using steel reinforcing bars.

The first control beam (TC1) was reinforced with a single #4 bar resulting in a reinforcement area equal to 1.3% of the concrete cross section which is equal to 0.46ρ_s.[2] The second control beam (TC2) was reinforced with two #4 bars for a reinforcement area equal to 2.6% of the concrete cross section which is equal to 0.93ρ_s. The first three test Three FRP beams (TH1, TH2, TH3) were reinforced with a single FRP H10 2-D grid consisting of two longitudinal bars with transverse bars located at 100 mm (4") intervals. These beams were designed such that the reinforcement would remain elastic up to concrete failure. The reinforcement area in these beams was equal to 1.0% of the concrete cross section. This reinforcement ratio was chosen to allow a direct comparison with tests conducted by the manufacturer.[43]
The next two test beams, (TH4 and TH5) were cast with three separate longitudinal FRP H10 bars placed in one layer, producing a reinforcement area of 1.4% of the concrete cross section. The reinforcing for these beams had short transverse stubs (approximately 12mm (1/2") long) at 100mm (4") longitudinal intervals. The longitudinal bars were placed so that the transverse stubs were adjacent to one another in test beam TH4, and offset by 50 mm (2") from one another in test beam TH5. Two layers of the H10 reinforcing grid were placed in the final beam (TH6). The first layer was placed 20mm (3/4") above the bottom surface and the second layer was placed at mid-depth of the beam.
Materials

The concrete beam specimens for this series of tests were cast at a local concrete batch plant using the New Hampshire Department of Transportation standard bridge mix design.[46] This design specifies a minimum compression strength of 4000 psi, a slump ranging between 1.5 inches and 3 inches, and a maximum aggregate size of 3/4 inches. The concrete used for the test samples had an average slump of less than two inches. Test cylinders broken the day of the beam flexural tests indicated an average ultimate compressive strength of 4,400 psi.
Test Setup

A 900 kN (200 kip) capacity Instron Model 1335 universal testing machine was used to perform the flexural tests. A loading frame with back to back A36 steel C10X30 channels was used to support the test beams in the testing machine (refer to Figure 14). The loading frame was designed such that at the ultimate load of the concrete beams, the deflection of the loading frame would be in the range of two to three orders of magnitude less than that of the test beam deflections. A four point load system with a clear span of 2.36m (60") between supports and loading points at 150mm (6") from the beam centerline was used. 25mm (1") radius bearing supports were used at each load or support point. The support points were designed to allow for rotation in two directions to insure that the test beam was symmetrically loaded.
This basic equipment setup was used for the monotonic beam tests (T-Series and Y-Series), the cyclical beam tests (N-Series), and the Q-Series and R-Series splice tests.

**Experimental Results of Monotonic Tests**

The results of these tests are summarized in Table 11. The test data is shown in Figure 14. This data shows that the initial portion of the load vs. deflection curve was linear for all beams.
### Table 11: Summary of Monotonic Flexural Test Results  
(T-Series Beams)

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>TH1</td>
<td>4610</td>
<td>2-H10</td>
<td>4180</td>
<td>50,160</td>
<td>1.50</td>
<td>4380</td>
<td>52,560</td>
<td>2.16</td>
</tr>
<tr>
<td>TH2</td>
<td>4610</td>
<td>2-H10</td>
<td>4230</td>
<td>50,760</td>
<td>1.51</td>
<td>4780</td>
<td>57,360</td>
<td>2.31</td>
</tr>
<tr>
<td>TH3</td>
<td>4610</td>
<td>2-H10</td>
<td>4210</td>
<td>50,520</td>
<td>1.51</td>
<td>4490</td>
<td>53,880</td>
<td>2.21</td>
</tr>
<tr>
<td>TH4</td>
<td>4200</td>
<td>3-H10</td>
<td>5140</td>
<td>61,680</td>
<td>1.20</td>
<td>5280</td>
<td>63,360</td>
<td>1.27</td>
</tr>
<tr>
<td>TH5</td>
<td>4200</td>
<td>3-H10</td>
<td>5920</td>
<td>71,040</td>
<td>1.25</td>
<td>6110</td>
<td>73,320</td>
<td>1.43</td>
</tr>
<tr>
<td>TH6</td>
<td>4200</td>
<td>2x2-H10</td>
<td>4780</td>
<td>57,360</td>
<td>1.44</td>
<td>4900</td>
<td>58,800</td>
<td>1.52</td>
</tr>
<tr>
<td>TC1</td>
<td>4200</td>
<td>1-#4</td>
<td>2500</td>
<td>30,000</td>
<td>0.36</td>
<td>2510</td>
<td>30,120</td>
<td>1.20</td>
</tr>
<tr>
<td>TC2</td>
<td>4610</td>
<td>2-#4</td>
<td>5700</td>
<td>68,400</td>
<td>0.42</td>
<td>6080</td>
<td>72,960</td>
<td>0.90</td>
</tr>
</tbody>
</table>

The initial cracking in control beam TC1 developed in the constant moment region of the beam (on the bottom surface) between the two points of loading. The initial cracking in control beam TC2 developed directly under the load application points on the bottom surface. In both control beams, subsequent cracks developed at random intervals between the load points in the tension zone of the concrete beams.

In test beams TH1, TH2, TH3, and TH4 the initial cracks formed in the constant moment region directly below the location of the transverse bars which were located closest to the point of loading. This was independent of whether the bars were aligned (TH1, TH3, TH4) or offset from the point of load application (TH2). In contrast, for test
beams TH5 and TH6, the initial cracking developed directly under the point of load application. For all the test beams, except beam TH5, subsequent tension zone crack patterns followed the location and spacing of the transverse reinforcing bars.

For control beam TC2, approximately 70% of the maximum displacement occurred after yield and before failure. In contrast, for test beams TH1, TH2, and TH3 approximately 32% of the maximum displacement occurred after yield and before failure. The amount of post yield displacement for test beam TH4, TH5, and TH6 was approximately 6%, 13% and 5% respectively. The post yield displacement in the FRP reinforced test beams does not represent yielding of the FRP reinforcement, rather this behavior was a result of the propagation of compression cracks.

Summary of Results of T-Series Beams

The load vs. deflection curves are shown in Figure 15. This figure shows that the slope of the load vs. deflection curves for the FRP reinforced concrete beams was proportional to those of the steel reinforced concrete beams, based upon the relative coefficient of stiffness of the fully cracked transformed sections ($E_sI$).

A theoretical load vs. deflection curve was calculated for each beam using the provisions of the ACI-318 code.[2] A comparison of the actual and theoretical load vs.
deflection curves is shown in Figure 16. This graph shows close agreement between the theoretical and the measured displacements.

The failure mode of each beam was consistent with that expected for the condition of over or under reinforcement relative to a balanced section. The FRP reinforced test beams failed in compression while the steel reinforced control beams failed in tension.

Concrete spalling on the top surface of each FRP reinforced beam was evident prior to ultimate failure. Ultimate failure in the FRP reinforced test beams TH4, TH5,
and TH6 was controlled by compression failure in the concrete. In test beams TH1, TH2, and TH3 ultimate failure occurred by a rupture of the longitudinal reinforcing bars at intersection with the transverse bar closest to the point of loading.

![Graph showing Load Vs. Deflection for Concrete beams Reinforced With Two FRP Bars](image)

**Figure 16 - Load Vs. Deflection**

Concrete beams Reinforced With Two FRP Bars

**Discussion of T-series Test Results**

This research has shown that FRP reinforced concrete beams demonstrate a bending behavior similar to that of concrete beams singly reinforced with steel bars. Force transfer, however, is developed differently between the two
reinforcement types. Steel reinforcing bars transfer forces continuously through bond developed between the concrete and the steel along the entire length of the deformed bar. FRP grid reinforcing transfers force through bearing of the FRP on the concrete at a series of discrete transverse bar locations.

Since FRP composites are a classified as a brittle material, concrete beams reinforced with FRP grids will show less ductility than a corresponding concrete beam reinforced with steel. The maximum modulus of elasticity (E) found for FRP grids is less than 1/3 that of steel. Thus, the deflection in a FRP reinforced concrete beam will be significantly greater than a concrete beam reinforced with steel with the same reinforcement area.

The results of these tests support the conclusion that flexurally loaded FRP reinforced concrete beams can be designed such that the failure sequence is analogous to that of "over-reinforced" concrete beams reinforced with mild steel. There was no evidence of relative slip between the transverse reinforcing bars and the concrete, indicating that the transverse bars adequately transferred the tensile forces in the reinforcements to the concrete. Each of the concrete beams reinforced with FRP responded consistently with respect to the phases that characterize the failure sequence of steel reinforced concrete beams.
Creep

When a structural member made from FRP composites is held under a sustained stress, strain continues to increase with time, and the magnitude of stress required to produce failure diminishes with time. In addition, the time to failure in a FRP composite increases with decreasing levels of stress. For FRP composites constructed with glass fibers, the effects of sustained stress may be significantly influenced by environmental factors such as moisture or alkali reactions.

Creep Testing

Preliminary creep tests were conducted using two T-Series test beams, one reinforced with steel reinforcing, the other with H10 FRP grids. These tests indicated that the concrete beam reinforced with fiber reinforced plastic composites experienced the general same creep rate as the steel reinforced concrete beam. The duration of this test was not long enough to be conclusive.
4.2 Monotonic Testing of FRP Grid Reinforced Concrete Beams with Low Reinforcing Ratios

Test Objective

The primary objective of this portion of the test program was to determine the flexural response of FRP reinforced beams containing low amounts of FRP reinforcing. This testing was conducted in order to determine if the design considerations presented in Chapter III were applicable to beams with reinforcement areas less than 1% of the concrete area.

Materials

This series of test specimens were also cast using the New Hampshire Department of Transportation standard bridge mix design. \[46\] Test cylinders broken the day of the flexural loading tests indicated an average compressive strength of 5,500 psi.

Test Specimens

Six concrete beams were cast for use by this project. The dimensions of the Y-Series beams were 1.83m (6'-0") long with a cross section 300 mm (12") deep and 200mm (8") wide. The dimensions of the Z-Series beams were the same length but had a cross section 200mm (8") deep and 300mm (12") wide.
All the test beams were reinforced with a single FRP 2-D grid consisting of two longitudinal H10 bars with transverse bars 7.5 inches long located at four inch intervals. These beams had a reinforcement area equal to 0.3% of the concrete cross section, which was less than 50% of that required for a balanced strain condition.

Test Setup

The apparatus from the T-Series tests was used for the Y-Series and Z-Series tests. This equipment was shown in Figure 14. The test specimens were loaded in four point bending with 0.79 m (31") shear spans and a 0.20 m (8") constant moment section.

Summary of Results of Y-Series and Z-Series

The results of the Y-Series load vs. deflection test are shown in Figure 17. The results of the Z-Series load vs. deflection test are shown in Figure 18. The test data is compared to the load vs. deflection predictions using the CEB bi-linear deflection equation presented in Chapter III. The ultimate load in the Y-Series beams was governed by shear failure, while that in the Z-Series was controlled by concrete compression failure. As noted in Table 12 the test results compare closely with calculations based upon the design considerations presented in Chapter III. The calculations for these test beams are shown in the Appendix.
Figure 17 - Load Vs. Displacement
Y-Series Test Beams

Figure 18 - Load Vs. Displacement
Z-Series Test Beams
The decreases in the load shown in the test data are characteristic of hydraulic test machines. The gross moment of inertia of the test specimen was so much greater than the cracked moment of inertia, so that after each crack formed in the test specimen the test machine was temporarily unable to maintain loads. \( I_{\text{gross}} = 1152 \text{ in}^4, I_{\text{cr}} = 37 \text{ in}^4 \).

<table>
<thead>
<tr>
<th>Beam I.D.</th>
<th>Measure. Ult. Load kN (kips)</th>
<th>Predict Ult. Load kN (kips)</th>
<th>Ratio Meas/Predict %</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>Y1</td>
<td>64.8 (14.6)</td>
<td>62.8 (14.1)</td>
<td>104 %</td>
<td>Shear</td>
</tr>
<tr>
<td>Y3</td>
<td>60.0 (13.5)</td>
<td>62.8 (14.1)</td>
<td>97 %</td>
<td>Shear</td>
</tr>
<tr>
<td>Z1</td>
<td>53.4 (12.0)</td>
<td>52.1 (11.7)</td>
<td>103 %</td>
<td>Conc. Comp.</td>
</tr>
<tr>
<td>Z2</td>
<td>53.0 (11.9)</td>
<td>52.1 (11.7)</td>
<td>102 %</td>
<td>Conc. Compress</td>
</tr>
<tr>
<td>Z3</td>
<td>54.5 (12.2)</td>
<td>52.1 (11.7)</td>
<td>105 %</td>
<td>Conc. Comp.</td>
</tr>
</tbody>
</table>
4.3 Cylindrical Testing of FRP Grid Reinforced Concrete Beams

Test Objective

The primary objective of this series of tests was to determine the flexural response of FRP grid reinforced concrete beams subjected to cyclic loads.

Test Specimens

Eight concrete beams were cast for use in this series of tests. The dimensions of each beam were 2.83 m (6'-0'') long with a cross section 100 mm (4'') deep and 200 mm (8'') wide. All the test beams were reinforced with a single FRP H10 2-D grid consisting of two longitudinal bars with transverse bars located at 100mm (4'') intervals. The H10 reinforcements were manufactured using the T700 carbon fibers in place of the original T300 fibers. The reinforcement properties are shown in Table 5 and Table 6. The reinforcement area in these beams was equal to 1.0% of the concrete cross section. This reinforcing ratio was chosen to allow for a direct comparison with the specimens in the previous test series.

Materials

The concrete cylindrical beam test specimens were cast at a local concrete batch plant using the New Hampshire Department of Transportation standard bridge mix design.[46]
The concrete used for the test samples had an average slump of less than one and one-half inches. Test cylinders broken the day of the cyclical loading tests indicated an average compressive strength of 5,600 psi.

Test Setup

The test apparatus from the monotonic tests was used for the cyclical loading tests. The equipment was shown in Figure 14.

Load Procedures

The beams were tested using one of two loading procedures. The first group of beams were loaded cyclically twenty (20) times to fifty percent (50%) of the maximum static load capacity of the beam.[23] The second group of beams were loaded cyclically according to the loading sequence shown in the Table 13. The purpose of the second series of tests was to determine the accumulative effects cyclically loading at various levels.

A third group of beams was scheduled to be cyclically loaded until failure at loading of fifty percent (50%) of the maximum static capacity of the beam, equipment failure caused this test to be stopped after approximately 16,000 cycles.
Table 13: Loading Sequence

<table>
<thead>
<tr>
<th>Percent of Maximum Static Load</th>
<th>Number of Cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>20 %</td>
<td>10</td>
</tr>
<tr>
<td>35 %</td>
<td>10</td>
</tr>
<tr>
<td>20 %</td>
<td>10</td>
</tr>
<tr>
<td>50 %</td>
<td>10</td>
</tr>
<tr>
<td>20 %</td>
<td>10</td>
</tr>
<tr>
<td>65 %</td>
<td>10</td>
</tr>
<tr>
<td>20 %</td>
<td>10</td>
</tr>
<tr>
<td>80 %</td>
<td>10</td>
</tr>
</tbody>
</table>

Experimental Results

In all tests the beam initially cracked directly below the location of the transverse bars located closest to the point of loading. This occurred independently of whether the transverse bars were aligned (NH1, NH3) or offset (NH2) from the point of load application. Subsequent tension zone crack patterns followed the location and spacing of the transverse reinforcing bars.

The results of the first group of cyclical loading tests, Figure 19, where the loading was cycled twenty times between 0 and 50% of ultimate, showed a growth in deflection with each cycle. After the first loading cycle a hysteresis loop developed with a growth in deflection with each cycle. The amount of increase in deflection decreased with each cycle and appeared to be asymptotically declining. The
growth in permanent deformation was about less than half the rate of growth in maximum deflection. Note that the uncracked section of the load vs. deflection curve for the test beams was slightly greater than that for the monotonically loaded beams presented in section 4.1 of this report due to an increase in ultimate concrete compressive strength of approximately 800 psi for the beams subjected to cyclical loading.
The results of the first series of cyclical load tests are shown in Figure 20 and summarized in the Table 14 and 15. Table 14 shows the beam deflection at the maximum load, the Table 15 shows the permanent deflections of the unloaded beams.

As shown in Figure 20 and Tables 14 and 15, the amount of deflection increased with each load cycle. There is a growth in deflection which appears to be asymptotic in 20 cycles when the load is cycled between zero and a maximum load.
<table>
<thead>
<tr>
<th>Number of Cycles</th>
<th>Maximum Deflection (in.)</th>
<th>Increase in Deflection (in.)</th>
<th>Percent Increase</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.567</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>0.628</td>
<td>0.061</td>
<td>10.8 %</td>
</tr>
<tr>
<td>10</td>
<td>0.646</td>
<td>0.018</td>
<td>2.9 %</td>
</tr>
<tr>
<td>20</td>
<td>0.661</td>
<td>0.015</td>
<td>2.3 %</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Number of Cycles</th>
<th>Minimum Deflection (in.)</th>
<th>Increase in Deflection (in.)</th>
<th>Percent Increase</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.227</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>0.232</td>
<td>0.005</td>
<td>2.2 %</td>
</tr>
<tr>
<td>10</td>
<td>0.236</td>
<td>0.004</td>
<td>1.0 %</td>
</tr>
<tr>
<td>20</td>
<td>0.237</td>
<td>0.001</td>
<td>0.4 %</td>
</tr>
</tbody>
</table>

FRP reinforced concrete beams experience an increase in permanent deformation each time the applied load is larger than the maximum of all previous loadings. There appears to be no growth in deflection when the load is cycled between zero and a maximum which is less than previously cycled loads on the beam. Independent of the number of previous loading cycles on a beam, the load deflection curve for a new maximum loading appears to follow the load vs. deflection curve for a single ultimate load test. A hysteresis loop was not apparent until the beam has been loaded to approximately 25% of ultimate.
The test beams were cut apart after the conclusion of each test to determine concrete failure locations. There were no visible differences between the crack patterns in the statically loaded beams (T-Series) and the cyclically loaded beams (N-Series). This indicates that crack propagation appeared to stabilize after a relatively few number of cycles.

As shown in Figure 21, the second group of cyclical loading tests followed a behavior pattern similar to the first series. Each time a load cycle was incremented in maximum load it followed the same load vs. deflection hysteresis loop as developed for the single set of 20 cycles at a fixed maximum load of 50% of ultimate. The subsequent cycling at 20% of ultimate followed a similar hysteresis loop with no measurable growth in deflection either at maximum or minimum load. The maximum deflection and the permanent deflection at minimum load coincided with values on the last cycle at the higher load. There did not appear to be growth in either the maximum or permanent deflection at the 20% loading level.
The results from the third cyclical loading test are shown in Figure 22, in which the variation of normalized maximum midspan deflection of a FRP reinforced beam is compared to test data on cyclically loaded steel reinforced concrete beams presented by Lovegrove and El Din.[36] The cyclical load vs. deflection behavior exhibited by the FRP reinforced concrete beam is similar to that exhibited by the cyclically loaded steel reinforced concrete beams.
Discussion of N-Series Test Results

This portion of the research program has shown that FRP reinforced concrete beams demonstrate a cyclical loading bending behavior similar to that of beams with steel reinforcement.[36]

Note that while the maximum and minimum deflections increases with each cycle, the rate of increase appears to asymptotically decrease.
4.4 Bridge Deck Slab Tests

This series of tests (D-Series) involved the investigation of the flexural behavior of FRP reinforced bridge deck slabs. This testing was conducted at the Federal Highway Administration Turner-Fairbanks Highway Research Center. These full scale test slabs were designed to be direct replacements for existing concrete bridge deck slabs reinforced with steel bars.

Bridge Deck Test Specimens

The concrete bridge deck test slabs were 0.22m (8'-1/2") deep, 1.22m (4'-0") wide, and 3.05m (10'-0") long. These dimensions were chosen to maintain compatibility with existing bridge deck slabs. The depth of cover on all slabs was 25mm (1"), resulting in a different value of "d" for each reinforcement type. The test slabs were designed to be supported in simple bending, with an 2.44m (8'-0") span length. The design load was one end of one axle from the AASHTO HS-25[1] design loading. The slab was loaded as if a dual tire from a fully loaded tractor trailer was supported at midspan. Per AASHTO guidelines, the design load was increased by 30% to account for impact. The resultant load, which will be referred to as the "Service Load", was equal to 117kN (26 Kips). The Service Load was distributed over an 0.25 m x 0.64 m (10" x 25") area of the slab representing the tire contact area of dual tires inflated to 689 kPa (100
psi). The test slabs were designed to the proposed AASHTO service limit state, which requires that at Service Load the radius of curvature of the slab will be greater than 6000 inches (EI/M > 6000`). For a 2.44m (8'-0") span length, this corresponds approximately to a midspan deflection of L/500. Three reinforcement designs were tested, H22 bars with 100mm (4") center to center spacing, C22 bars with 100mm (4") spacing, and as a control, #5 Grade 60 Steel bars with 122mm (4.8") spacing. These designs are summarized in the following table:

<table>
<thead>
<tr>
<th>Bar Type</th>
<th>Nominal Reinf. Dia.</th>
<th>Slab &quot;d&quot;</th>
<th>Total Area</th>
<th>(\rho) ((\text{A}_{	ext{net}}/bd))</th>
<th>(E), MPa</th>
<th>(A_{\text{E}}), MN</th>
</tr>
</thead>
<tbody>
<tr>
<td>H22</td>
<td>26 mm (1.02)</td>
<td>178 mm (6.99)</td>
<td>4800 sq.mm (74.4)</td>
<td>0.022 (0.002)</td>
<td>41,200 (5985)</td>
<td>198 (44,500)</td>
</tr>
<tr>
<td>C22</td>
<td>21 mm (0.83)</td>
<td>180 mm (7.09)</td>
<td>2520 sq.mm (39.1)</td>
<td>0.012</td>
<td>84,700 (12,300)</td>
<td>214 (48,000)</td>
</tr>
<tr>
<td>#5</td>
<td>16 mm (0.63)</td>
<td>183 mm (7.19)</td>
<td>2000 sq.mm (31.0)</td>
<td>0.009</td>
<td>200,000 (29,000)</td>
<td>400 (89,900)</td>
</tr>
</tbody>
</table>

Note: for all slabs \(b = 1220\text{mm (48")}\) and \(h = 220\text{mm (8.5")}\)

The design of the steel reinforced concrete slab was taken from a standard AASHTO bridge deck. [1]
Concrete Properties

The concrete used for the bridge deck test specimens was specified to be produced to the Virginia DOT "Post and Rail" standard mix design.[58] This design specifies that the concrete shall have a maximum aggregate size of 13mm (1/2"), a slump ranging between 40mm (1-1/2") and 80mm (3"), and a minimum compressive strength after 28 days of 27.5 MPa (4000 psi). The concrete was obtained from a local ready mix concrete supplier.

Due to the volume of concrete required, the bridge deck test specimens were cast in two separate pours. Specimens H3, C4, C5, St-1 were cast on 07/30/91 (Pour #1), while specimens H1, H2, and C6 were cast on 07/31/91 (Pour #2).

Compression tests were conducted on standard 150 mm x 300 mm (6"x12") concrete cylinders to determine the compressive strength of the concrete used in the bridge deck test specimens. These tests were conducted following the provisions of ASTM C39-84.[3] The concrete strength was determined prior to the removal of the specimens from the forms and again at the commencement of testing. Due to the extended duration of the bridge deck test program, the concrete strength was also tested at the conclusion of the bridge deck testing. The results of the concrete compression tests are shown in Figure 23.
Figure 23 - Compressive Strength Vs. Age
Concrete From FRP Reinforced Bridge Deck Specimens

Note: Each data point represented a minimum of four test cylinders.

The modulus of elasticity of the concrete was calculated based upon the provisions of ASTM C469-87a and ACI 318-89. [2]

ASTM C469 $E = 30,300 \text{ MPa (4400 ksi)}$

ACI-318 $E = 31,000 \text{ MPa (4500 ksi)}$

Using the lowest modulus from each FRP type from Table 8, the ratio of the modulus of elasticity of the FRP...
reinforcements to that of the concrete is as follows:

<table>
<thead>
<tr>
<th>Bar Type</th>
<th>Modular Ratio (n)</th>
</tr>
</thead>
<tbody>
<tr>
<td>H-Type</td>
<td>1.3</td>
</tr>
<tr>
<td>C-Type</td>
<td>2.7</td>
</tr>
</tbody>
</table>

The ratio of the modulus of elasticity for the FRP reinforcement to that of concrete is considerably lower than the minimum value of six (6) specified in ACI 318-89.[2] Since the modulus of elasticity for concrete is only slightly less than the modulus of elasticity of the H-Type NEFMAC reinforcing grid, the strain to which the concrete will be subjected will be considerably greater than would be case with steel reinforcement. Compared to a steel reinforced concrete specimen, the low modulus of elasticity of the FRP reinforcements will cause the neutral axis of the concrete specimens to be further away from the tension reinforcement, resulting in a reduced concrete compression area and tending to promote compression failure.
Bridge Deck Test Setup

The concrete bridge deck slabs were tested using the equipment setup shown in Figure 24. The test decks were supported on fixtures fabricated from structural steel, providing the required simple span length of 2.4m (8'-0''). A 1300kN (300Kip) hydraulic jack was used to apply loads to the test decks. The loads were distributed to the deck through a steel plate, 50mm (2'') thick, 250mm (10'') wide by x 640mm (25'') long on 12mm (1/2'') thick neoprene bearing pads in order to simulate the tire contact area of a HS25 tractor trailer.

![Diagram of Bridge Deck Test Setup](image)

**Figure 24 - Bridge Deck Test Setup**
Test Results - Load vs. Deflection

As shown in Figure 25 due to the reinforcement rigidity (AE), the FRP reinforced slabs exhibited significantly greater deflections, and thus greater crack widths and lengths, than the control steel reinforced deck. In addition, note that while the ultimate slab capacity for all types of reinforcement was approximately equal, the steel reinforced slab exhibited significantly more ductility than the FRP reinforced decks.

![Concrete Decks - Midspan @ Centerline](Image)

**Figure 25 - Load Vs. Deflection - Monotonic Loading**
The following figures (Figure 26, Figure 27, and Figure 28) show Load Vs. Deflections at the midspan and at quarter span, along the centerline of the slab and at the edge of slab.

**Figure 26 - Load Vs. Deflection, Steel Reinforced Slab**

Note that there was more relative deflection between the centerline and edge of the slabs at midspan compared to at 1/4 span.
Figure 27 - Load Vs. Deflection, H22 Reinforced Slab

Figure 28 - Load Vs. Deflection, C22 Reinforced Slab
Cyclical Loading of Bridge Deck Slabs

Two slabs of each type of FRP reinforcement were cyclically loaded to failure. The goal of the cyclical loading tests was to investigate the progression of damage and deformations in the concrete slabs. The residual damage, measured either by deformation and crack length growth, was monitored during each load/unload cycle of the test.

Although these cyclical tests are not fatigue tests, the results from these tests give an indication of how the slabs will perform under fatigue loading. Poor fatigue performance is predicted if the residual damage continues to increase during the cycling at a given load. If the residual damage does not increase, or the amount of increase asymptotically decreases with each successive cycle, then good fatigue performance is predicted.
Two H22 reinforced and two C22 Reinforced bridge deck slabs were tested using the following loading sequence:

<table>
<thead>
<tr>
<th>Series #</th>
<th>Start Load</th>
<th>Maximum Load</th>
<th>End Load</th>
<th># Cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>Service 116kN (26K)</td>
<td>0</td>
<td>10</td>
</tr>
<tr>
<td>2</td>
<td>0</td>
<td>2 X Service 232kN (52K)</td>
<td>0</td>
<td>10</td>
</tr>
<tr>
<td>3</td>
<td>0</td>
<td>Service 116kN (26K)</td>
<td>0</td>
<td>10</td>
</tr>
<tr>
<td>4</td>
<td>0</td>
<td>3X Service 348kN (78K)</td>
<td>0</td>
<td>10</td>
</tr>
<tr>
<td>5</td>
<td>0</td>
<td>Service 117kN (26K)</td>
<td>0</td>
<td>10</td>
</tr>
<tr>
<td>6</td>
<td>0</td>
<td>Ultimate</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Summary of Cylcical Loading of Bridge Deck Slabs

For the slabs reinforced with C22 bars, the ultimate failure loads ranged between 92 kips (409 kN) and 100 kips (445 kN). For the slabs reinforced with H22 bars, the ultimate failure loads ranged between 74 kips (329 kN) and 94 kips (418 kN). Comparisons between the performance of the test decks when subjected to monotonic loadings and those subjected to cyclical loadings are shown in Figure 29 and Figure 30.
LOAD VS. DEFLECTION
C22 Reinforced Decks

Figure 29 - Monotonic and Cyclic Loading
C22 Reinforced Bridge Deck Slab

LOAD VS. DEFLECTION
H22 Reinforced Decks

Figure 30 - Monotonic and Cyclic Loading
H22 Reinforced Bridge Deck Slab
Crack Patterns In Bridge Deck Test Slabs

In a manner similar to decks reinforced with welded wire mesh, cracks develop in the NEFMAC FRP test decks corresponding to the 100mm (4") transverse bar spacing.[33] In the FRP reinforced decks, the cracks initiated at the transverse bars closest to the point of load application. This cracking pattern is due to the force transfer mechanism of FRP grids, in which the tensile forces in the concrete beam are developed in the longitudinal FRP bars through direct bearing of the transverse members upon the concrete. In the steel reinforced control deck, the cracks initiated at the point of maximum moment.

The flexural cracking patterns at the bottom of the slab followed the location of the transverse bars. As evidenced by the lack of longitudinal cracking, there was very little indication of two-way action, except near the point of loading.
Crack Pattern Progression

The crack pattern progression is shown in the following photographs and in Figure 33. Note that diagonal shear tension cracks appeared at approximately 3xService Load 350 kN (78 kips). These cracks formed independently from the flexural cracks.
Figure 32 - Crack Pattern, H22 Reinforced Slab
Photograph 6 - H22 Reinforced Slab, Crack Pattern After 10 Cycles at Service Load

Photograph 7 - Crack Pattern @ 2xService Load
Photograph 8 - Details of Shear Tension Failure

The diagonal "shear-tension" cracks formed independently of the flexural cracks. The horizontal cracks at the top of the FRP reinforcement formed after failure was induced by the "shear-tension" cracks. While the a/d ratio for all of these slabs was approximately 5, typical of intermediate length beams (2.5 < a/d < 6), the failure mode for the FRP reinforced beams was closer to that of short beams (1 < a/d < 2.5)[38][59].
Figure 33 - Crack Pattern Progression Specimen C22-6
Material Strains in Bridge Deck Test Slabs

The stress vs. strain diagram for the components of a steel reinforced concrete deck subjected to a monotonic loading are shown in Figure 34. The strain in the steel is linear up to yield, and then non-linear up to ultimate. Note the difference between the strains at mid-span and those at 1/4 span.

![Graph showing load vs. strain for different components]

Figure 34 - Load Vs. Strain, Steel Reinforced Slab

The stress vs. strain diagram for the components of a H22 reinforced concrete deck subjected to a monotonic loading are shown in Figure 35. The strain in both the FRP
reinforcement and the concrete is essentially linear up to the ultimate load of the test specimen. At the ultimate load of the test specimen the strain in the FRP was significantly less than the ultimate strain of the FRP, indicating that the concrete strength governs rather than FRP strength. Due to the relatively large depth of the H22 reinforcement grid, 27mm (1.06 inches), there was a measurable difference between the strain in the top surface of the H22 grid and the bottom surface.

![Load Vs. Strain, H22 Reinforced Deck](image)

*Figure 35 - Load Vs. Strain, H22 Reinforced Deck*
The stress vs. strain diagram for the components of a C22 reinforced concrete deck subjected to a monotonic loading are shown in Figure 36. The strain in both the FRP reinforcement and the concrete is also linear up to the ultimate load. In addition, the strain in the FRP is less than the ultimate strain of the FRP, indicating that the concrete strength governs rather than FRP strength. Note that due to the relatively smaller depth of the C22 reinforcement grid compared to the H22 reinforcement grid, 21mm (0.83 inches) vs. 27mm (1.06 inches), there was less difference between the strain in the grid's top and bottom surfaces.
The stress vs. strain diagram for the components of a H22 reinforced concrete deck subjected to a cyclical loading are shown in Figure 37. The strain in both the FRP reinforcement and the concrete was linear up to the ultimate load. Again, note that due to the relatively large depth of the H22 reinforcement grid there was a measurable difference between the strain in the top surface of the H22 grid and the bottom surface.
The stress vs. strain diagram for the components of a C22 reinforced concrete deck subjected to a cyclical loading are shown in Figure 38 and Figure 39. The strain in both the FRP reinforcement and the concrete was linear up to the ultimate load. These figures also show that the assumption that plane sections remain plane after bending is only an approximation of the actual strain distributions within the section.
Figure 38 – C22 Reinforced Deck

Due to the relatively shorter depth, the differences in strain measurements between the top and bottom surfaces of the C22 grid were smaller than the differences found in all samples reinforced with the H22 grid.
Summary of FRP Reinforced Bridge Deck Tests

As shown Table 18, the ultimate load of all the FRP grid reinforced slabs significantly exceeded the design service load. The FRP grid reinforced slabs can be designed to satisfy AASHTO serviceability requirements.

The FRP reinforced slabs performed satisfactorily when subjected to multiple load/unload cycles. The FRP grid reinforced slabs tested during the limited scale cyclical test portion of this program did not exhibit stiffness degradation.
<table>
<thead>
<tr>
<th>Deck I.D. (Bar Type)</th>
<th>Loading Type</th>
<th>Ultimate Load kN (Kips)</th>
<th>Ult/Service %</th>
</tr>
</thead>
<tbody>
<tr>
<td>H1 (H22)</td>
<td>Cycle.</td>
<td>436 (97.9)</td>
<td>376 %</td>
</tr>
<tr>
<td>H2 (H22)</td>
<td>Mono.</td>
<td>321 (72.1)</td>
<td>277 %</td>
</tr>
<tr>
<td>H3 (H22)</td>
<td>Cycle.</td>
<td>347 (78.0)</td>
<td>300 %</td>
</tr>
<tr>
<td>C4 (C22)</td>
<td>Cycle.</td>
<td>464 (104.3)</td>
<td>401 %</td>
</tr>
<tr>
<td>C5 (C22)</td>
<td>Mono.</td>
<td>424 (95.2)</td>
<td>366 %</td>
</tr>
<tr>
<td>C6 (C22)</td>
<td>Cycle.</td>
<td>402 (90.3)</td>
<td>347 %</td>
</tr>
<tr>
<td>ST7 (St #5)</td>
<td>Cycle.</td>
<td>402 (90.3)</td>
<td>347 %</td>
</tr>
</tbody>
</table>

As calculated using the provisions of Chapter III, all FRP reinforced slabs failed due to diagonal shear-tension failure or concrete compression failure. Example calculations are contained in the Appendix. There was no indication of FRP reinforcement failure. Flexural crack locations coincided with the transverse bar locations.
4.5 Splice/Development Length Requirements for FRP Grid

The research presented in this portion of the test program was directed towards determining development length and splice requirements for FRP grids used in the reinforcement of concrete structures. This work is one of the steps required to determine the suitability of the FRP material for various structural applications, such as bridge decks or pavements.

The use of FRP bars for the reinforcement of concrete has been limited, in part, due to the poor bonding properties of FRP. The use of FRP Grids addresses the problems with poor bond performance by developing bond through direct concrete bearing on bars which are placed transverse to the longitudinal axis of the main reinforcing bars.

The manufacturer of the NEFMAC FRP grid has not tested the material for splice or development requirements. The first portion of the work required to determine splice and development length requirements was initiated at UNH. This work focused upon thin flexural members, reinforced with the smaller NEFMAC grids and short beams reinforced with C19 or H19 grids. In order for FRP grid to be used for bridge deck construction, splicing and development length requirements must be determined.
Splice Details

Three different splice designs have been analyzed. As shown in Figure 40, the splices are referred to as "Cover", "Lap", and "Interlock".

- The "Cover" splice uses a section of grid to splice two adjacent reinforcing grids. The length of this cover piece is slightly greater than 2*L.d.b.

- The "Lap" splice is fabricated by overlapping one reinforcing grid with an adjacent grid. Note that the effective depth of concrete will be different on each side of the splice.

- The "Interlock" splice is created by cutting out portions of the transverse bars, and interlocking the longitudinal bars. This splice detail requires significantly more work to fabricate than the other two methods. Preliminary tests indicated that this splice configuration was not capable of developing a significant portion of the reinforcement capacity.
Splice Design Considerations

The most significant difference between FRP grids and individual bars is that, as a result of the two-dimensional layout of the FRP grid, adjacent sections of FRP at a lap splice are not in the same plane. This same situation does not occur in the use of steel bar reinforcement, although it does occur when using welded wire mesh. However, in designing concrete members with welded wire mesh reinforcements, it is usually permissible to treat the
reinforcement on both sides of the splice as if lying in the same plane. This situation is allowed since the steel in the welded wire mesh has a yield point significantly below its ultimate stress, and welded wire mesh is typically of relatively small diameter, generally no larger than 16 mm (0.628 in.), ACI W31 or D31. In contrast, the FRP reinforcing exhibits no appreciable yield before reaching ultimate stress and the nominal FRP bar sizes are much larger, ranging up to 35 mm (#11 bar).
Figure 41 - Reinforcement Details

S-Series Splice Test Specimens

The three S-Series reinforcement configurations tested are shown in the Table 19. The first design used two NEFMAC H19 bars, the second used three NEFMAC C19 bars, and the control used a single #5 Grade 60 steel bar.
<table>
<thead>
<tr>
<th>Bar Type</th>
<th>Nominal Reinf. Depth $d$ (mm)</th>
<th>Beam Depth $d'$ (mm)</th>
<th>Total Area $P_{min}$ (sq.mm)</th>
<th>$\rho$ (MPa)</th>
<th>$E$ (MPa)</th>
<th>$AE$ (MN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-H19 SH</td>
<td>20 (0.79)</td>
<td>168 (6.61)</td>
<td>600 (0.93)</td>
<td>0.012 (0.002)</td>
<td>41,500 (6080)</td>
<td>25 (5650)</td>
</tr>
<tr>
<td>3-C19 SC</td>
<td>12.5 (0.49)</td>
<td>171 (6.76)</td>
<td>480 (0.74)</td>
<td>0.009 (0.001)</td>
<td>85,300 (12,400)</td>
<td>41 (9100)</td>
</tr>
<tr>
<td>1#5 SS</td>
<td>16 (0.63)</td>
<td>170 (6.69)</td>
<td>200 (0.31)</td>
<td>0.004 (0.003)</td>
<td>200,000 (29,000)</td>
<td>40 (8990)</td>
</tr>
</tbody>
</table>

Notes: 1. All beams $b = 300mm$ (12"), and $h = 200mm$ (8”).

Due to 2-D nature of the FRP grid, there is a difference in depth to reinforcing from one side of the lap splice to the other. In order to account for this difference, there are two different grid reinforced control beams for each grid type. The first was fabricated with 1" cover, the second was fabricated with depth of cover equal to 25mm (1") plus the bar depth. The load vs. deflection performance of the control beams was designed to envelope that of the specimens with spliced reinforcements.

If construction constraints require that a splice be installed at any location, a conservative design approach to account for this would be to assume that the cover on FRP grid reinforced concrete members is equal to the actual cover plus the depth of the FRP grid.
S-Series FRP Grid Reinforced Beams

Since the transverse bars in the FRP grids were located at 100 mm (≈4 in.) intervals, the splices for all test specimen were designed based upon the number of transverse bars which overlap or were embedded.

As indicated in the previous figure, FRP grid reinforcement splices were fabricated by overlapping one section of FRP grid with the adjacent section. The S-Series test specimens were a series of 0.30 m (≈12 in.) wide beams loaded monotonically to failure in four point flexure, with "Lap" splices located in the constant moment section of the beams (Figure 42). The shear span for the S-Series test was 1.2m (48"). The Q-Series and R-Series test specimens were a series of 0.20m (≈8 in.) wide beams loaded monotonically to failure in four point flexure, with "Lap" splices located in the constant moment section of the beams (Figure 42). The shear span for these two series was 810mm (32").
Figure 42 - Test Diagram

Since the transverse bars in the FRP grids were located at 100 mm (≈4 in.) intervals, the splices for all test specimen were designed based upon the number of transverse bars which overlap or were embedded.

Q-Series and R-Series Splice Test Specimens

The Q-Series and R-Series configurations tested are shown in Table 20.
<table>
<thead>
<tr>
<th>Bar Type</th>
<th>Series I.D.</th>
<th>Nominal Reinf. Depth (mm)</th>
<th>Beam Depth &quot;d&quot; (mm)</th>
<th>Total Area Reinf. (sq.mm.)</th>
<th>ρ (ρmin)</th>
<th>E (MPa (ksi))</th>
<th>AE (MN (kips))</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-H10 OH</td>
<td>10 (0.39)</td>
<td>81 (3.18)</td>
<td>156 (0.242)</td>
<td>0.0095 (0.002)</td>
<td>41,500 (6080)</td>
<td>3.3 (735)</td>
<td></td>
</tr>
<tr>
<td>2-C10 QC</td>
<td>8 (0.31)</td>
<td>82 (3.22)</td>
<td>100 (0.156)</td>
<td>0.0061 (0.001)</td>
<td>85,300 (12,400)</td>
<td>4.5 (1010)</td>
<td></td>
</tr>
<tr>
<td>2-H19 RH</td>
<td>20 (0.79)</td>
<td>177 (6.98)</td>
<td>600 (0.93)</td>
<td>0.0167 (0.002)</td>
<td>41,500 (6080)</td>
<td>25 (5650)</td>
<td></td>
</tr>
<tr>
<td>2-C19 RC</td>
<td>12.5 (0.49)</td>
<td>181 (7.13)</td>
<td>320 (0.496)</td>
<td>0.0087 (0.001)</td>
<td>85,300 (12,400)</td>
<td>41 (9100)</td>
<td></td>
</tr>
</tbody>
</table>

Notes: 1. All beams b = 200mm (8").
The Q-Series and R-Series portions of the program used the following samples:

<table>
<thead>
<tr>
<th>Beam I.D.</th>
<th>Bar Type</th>
<th># Bars</th>
<th>Splice Length</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>QH1</td>
<td>H10</td>
<td>2</td>
<td>0.10m (4&quot;)</td>
<td></td>
</tr>
<tr>
<td>QC1</td>
<td>C10</td>
<td>2</td>
<td>0.10m (4&quot;)</td>
<td></td>
</tr>
<tr>
<td>QC2</td>
<td>C10</td>
<td>2</td>
<td>0.20m (8&quot;)</td>
<td></td>
</tr>
<tr>
<td>RH1</td>
<td>H19</td>
<td>2</td>
<td>0.10m (4&quot;)</td>
<td></td>
</tr>
<tr>
<td>RH2</td>
<td>H19</td>
<td>2</td>
<td>0.25m (8&quot;)</td>
<td></td>
</tr>
<tr>
<td>RH3</td>
<td>H19</td>
<td>2</td>
<td>0.30m (12&quot;)</td>
<td></td>
</tr>
<tr>
<td>RH4</td>
<td>H19</td>
<td>2</td>
<td>0.30m (12&quot;)</td>
<td>0.15m trans. Grid spacing</td>
</tr>
<tr>
<td>RC1</td>
<td>C19</td>
<td>2</td>
<td>0.10m (4&quot;)</td>
<td></td>
</tr>
</tbody>
</table>

Notes: Lap splices were used for FRP grid reinforced beams. Transverse bars of the top grid were placed were inline with the transverse bars of the bottom grid.

Specimen Length = 1.8m (6 ft).

16mm (5/8 inch) cover on the "H" specimens is < 2dₖ,
16mm (5/8 inch) cover for the "C" specimens = 2dₖ
The S-Series portion of the test program used the following samples:

<table>
<thead>
<tr>
<th>Beam I.D.</th>
<th>Bar Type</th>
<th># Bars</th>
<th>Splice Length</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>SH1</td>
<td>H19</td>
<td>2</td>
<td>continuous bar</td>
<td>1&quot; Cover</td>
</tr>
<tr>
<td>SH2</td>
<td>H19</td>
<td>2</td>
<td>continuous bar</td>
<td>1&quot;+d_b Cover</td>
</tr>
<tr>
<td>SH3</td>
<td>H19</td>
<td>2</td>
<td>0.45m (18&quot;)</td>
<td></td>
</tr>
<tr>
<td>SH4</td>
<td>H19</td>
<td>2</td>
<td>0.65m (26&quot;)</td>
<td></td>
</tr>
<tr>
<td>SH5</td>
<td>H19</td>
<td>2</td>
<td>0.85m (34&quot;)</td>
<td></td>
</tr>
<tr>
<td>SC1</td>
<td>C19</td>
<td>3</td>
<td>continuous bar</td>
<td>1&quot; Cover</td>
</tr>
<tr>
<td>SC2</td>
<td>C19</td>
<td>3</td>
<td>continuous bar</td>
<td>1&quot;+d_b Cover</td>
</tr>
<tr>
<td>SC3</td>
<td>C19</td>
<td>3</td>
<td>0.45m (18&quot;)</td>
<td></td>
</tr>
<tr>
<td>SC4</td>
<td>C19</td>
<td>3</td>
<td>0.65m (26&quot;)</td>
<td></td>
</tr>
<tr>
<td>SC5</td>
<td>C19</td>
<td>3</td>
<td>0.85m (34&quot;)</td>
<td></td>
</tr>
</tbody>
</table>

Notes: Lap splices were used for FRP grid reinforced beams.
Transverse members were offset by 50mm (2 inches).
Specimen Length = 3.9m (13 ft).
25mm (1 inch) cover on the "H" specimens is < 2d_b
25mm (1 inch) cover for the "C" specimens = 2d_b
Steel Reinforced Beams

Steel Reinforced Beams were tested in order to serve as a control to the FRP reinforced test beams. All splices located in the constant moment section of the beams (Figure 42).

| Table 23: S-Series Steel Reinforcement Splice/Development Length Test Specimens |
|-----------------|-----------------|-----------------|-----------------|
| Beam I.D.       | Bar Type        | # Bars          | Splice Length   | Comments        |
| SS1             | #5              | 1               | continuous bar  |                 |
| SS2             | #5              | 1               | 0.30m (12")     | lap splice      |
| SS3             | #5              | 1               | 0.53m (21")     | lap splice      |

Note: Specimen Length = 3.9m (13 ft) unless noted.
Test Setup

The S-Series test beams were loaded monotonically to failure in four point symmetrical bending, using a 500kN (120 Kip) jack and 445kN (100 Kip) load cell as shown in the Figure 43 and Figure 44. The test beams were supported on a convex bearing plate at one end of the beam and a concave plate with roller at the other end of the beam.

Figure 43 - Test Setup
A total of five deflection gauges were placed on each specimen; midspan, at load points, and midway between the supports and the load points. The deflections were continuously monitored during testing to verify that the beams were symmetrically loaded.

The Q-Series and R-Series used the same test configuration as the T-Series test beams, as shown in Figure 14.
Concrete Properties

The concrete used for the S-Series splice test specimens was specified to be produced to the Virginia DOT "Post and Rail" standard mix design.[58] This design specifies that the concrete shall have a maximum aggregate size of 13mm (1/2"), a slump ranging between 40mm (1-1/2") and 80mm (3"), and a minimum compressive strength after 28 days of 27.5 MPa (4000 psi). The concrete was obtained from a local ready mix concrete supplier.

Compression tests were conducted to determine the compressive strength of the concrete used in splice/development length test specimens. These tests were conducted following the provisions of ASTM C39-84.[3] The concrete strength was determined prior to the removal of the specimens from the forms and again at the commencement of testing. The results of the concrete compression tests are shown in Figure 45.
The concrete beam specimens for Q-Series and R-Series tests were cast at a local concrete batch plant using the New Hampshire Department of Transportation standard bridge mix design.[46] Test cylinders broken the day of the beam flexural tests indicated an average ultimate compressive strength of 4,800 psi.

The modulus of elasticity of the S-Series concrete was calculated based upon the provisions of ACI 318-89.

\[ E = 30,300 \text{ MPa (4400 ksi)} \]

Using the lowest modulus from each FRP type from Table 8, the ratio of the modulus of elasticity of the FRP reinforcements to that of the concrete is as follows:
<table>
<thead>
<tr>
<th>Bar Type</th>
<th>Modular Ratio (n)</th>
</tr>
</thead>
<tbody>
<tr>
<td>H-Type</td>
<td>1.4</td>
</tr>
<tr>
<td>C-Type</td>
<td>2.8</td>
</tr>
</tbody>
</table>

Similarly, the Modular Ratio (n) for the Q-Series and R-Series test concrete is as follows:

<table>
<thead>
<tr>
<th>Bar Type</th>
<th>Modular Ratio (n)</th>
</tr>
</thead>
<tbody>
<tr>
<td>H-Type</td>
<td>1.6</td>
</tr>
<tr>
<td>C-Type</td>
<td>3.3</td>
</tr>
</tbody>
</table>
Splice Test Results Summary

The following table presents the test results from the splice/development length testing.

Table 24: FRP Grid Splice/Development Length Test Results

<table>
<thead>
<tr>
<th>Beam I.D.</th>
<th>Bar Type</th>
<th>Splice Length mm (in.)</th>
<th>Max. Load kN (Kips)</th>
<th>Cal’d Load kN (Kips)</th>
<th>Ratio Test to Cal’d %</th>
<th>Bar Force kN (Kips)</th>
<th>% Ult Bar</th>
<th>Fail Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>SH1</td>
<td>H19</td>
<td>-</td>
<td>75.5 (17.0)</td>
<td>71.6 (16.1)</td>
<td>105%</td>
<td>148.3 (33.5)</td>
<td>73%</td>
<td>V</td>
</tr>
<tr>
<td>SH2</td>
<td>H19</td>
<td>-</td>
<td>58.7 (13.2)</td>
<td>58.7 (13.2)</td>
<td>100%</td>
<td>132.3 (29.7)</td>
<td>65%</td>
<td>V</td>
</tr>
<tr>
<td>SH3</td>
<td>H19</td>
<td>450 (18)</td>
<td>51.2 (11.5)</td>
<td>58.7 (13.2)</td>
<td>87%</td>
<td>100.9 (22.7)</td>
<td>49%</td>
<td>S</td>
</tr>
<tr>
<td>SH5</td>
<td>H19</td>
<td>850 (34)</td>
<td>54.4 (12.2)</td>
<td>58.7 (13.2)</td>
<td>92%</td>
<td>107.3 (24.1)</td>
<td>52%</td>
<td>V</td>
</tr>
<tr>
<td>SC1</td>
<td>C19</td>
<td>-</td>
<td>78.8 (17.7)</td>
<td>84.1 (18.9)</td>
<td>94%</td>
<td>103.3 (23.2)</td>
<td>51%</td>
<td>E</td>
</tr>
<tr>
<td>SC2</td>
<td>C19</td>
<td>-</td>
<td>85.4 (19.2)</td>
<td>78.2 (17.6)</td>
<td>109%</td>
<td>121.1 (27.2)</td>
<td>59%</td>
<td>V</td>
</tr>
<tr>
<td>SC3</td>
<td>C19</td>
<td>450 (18)</td>
<td>72.5 (16.3)</td>
<td>78.2 (17.6)</td>
<td>93%</td>
<td>95.0 (21.3)</td>
<td>47%</td>
<td>S</td>
</tr>
<tr>
<td>SC5</td>
<td>C19</td>
<td>850 (34)</td>
<td>79.7 (17.9)</td>
<td>78.2 (17.6)</td>
<td>102%</td>
<td>104.5 (23.5)</td>
<td>51%</td>
<td>S</td>
</tr>
</tbody>
</table>

Where:

S = Failure of Lap Splice.

V = Shear failure outside constant moment region.

E = Anchorage failure due to void in concrete.

Note that beams SC4 and SH4 are not included in this table. Both of these two beams had voids in the splice region of the beam which affected the test results.
Load vs. Deflection

The measured load vs. deflection results for the control beams are shown in Figure 46. Note that the FRP reinforced beams exhibited significantly higher ultimate loads than the steel reinforced beams.

![Load vs. Deflection Diagram](image)

Figure 46 - Load Vs. Deflection

The ultimate capacity of the steel reinforced beam was limited by yielding in the steel reinforcement. In contrast, the FRP reinforced concrete beams failed by diagonal shear-tension cracking.

Deflections were calculated using the ACI 318 equations.
derived by Branson and the CEB bilinear method. As shown in Figure 47 and Figure 48, the CEB bilinear method provides a better estimation of deflections than the ACI method.

Comparison between ACI 318 and CEB deflection predictions for SC series control beams:

![Graph showing load vs. deflection for control beam SC1](image)

**Figure 47 - Measured and Predicted Load Vs. Deflection For Control Beam SC1**

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DISCUSSION OF RESULTS

Due to 2-D nature of the FRP grid, there is a difference in depth to reinforcing from one side of the lap splice to the other. In order to account for this difference, there were two different grid reinforced control beams for each grid type. The first was fabricated with 25mm (1") cover, the second was fabricated with depth of cover equal to 25mm (1") plus the bar depth. As shown in
Figure 49, The load vs. deflection performance of the control beams enveloped those of the specimens with lap spliced reinforcements. Beam SC5, which had the longest length splice, initially performed as if it had two layers of reinforcement, and deflected slightly less than the control beams.

![Load vs. Deflection Graph for CS Series Test Beams](image)

Since the steel reinforced specimens were designed to be near the minimum reinforcing ratio, the predicted failure
mechanism was reinforcement yielding. In addition, the depth to span ratio for the test specimens was approximately 7, which is normally within the range of long beams, which generally precludes concrete shear as the failure mechanism.[37] The steel reinforced beams yielded as calculated, failure induced by yielding of the tension reinforcement. The FRP reinforced beams were designed to have the same reinforcement rigidity as the steel reinforced beams. Since the FRP reinforcements have a lower modulus of elasticity but a higher ultimate failure stress than the steel reinforcements, the FRP reinforced beams have significantly larger reinforcement ratios than the steel reinforced concrete beams. The larger reinforcement ratios, combined with the lower modulus of elasticity, resulted in concrete compression or diagonal shear governing capacity rather than reinforcement tension capacity. The FRP grid reinforced beams behaved as "intermediate length" beams while the steel reinforced beams with the same a/d ratio behaved as "long" beams.

The splices in the FRP bars failed by splitting failure with the plane of failure passed horizontally between the layers of grid. This failure mode is predicted based upon the requirements that l, be not less than the second equation given by ACI 12.7.2. for deformed welded wire mesh and by ACI 12.8 for plain welded wire mesh.
Development Length Recommendations

The development length for deformed steel welded wire mesh is given by ACI 318-89 as:

\[ l_{db} \geq 0.20 \frac{A}{S} \frac{f_y}{\sqrt{f'_c}} \quad \text{ACI 12.7.2} \]

Similarly, the development length for plain welded wire mesh is given by:

\[ l_{db} \geq 0.27 \frac{A}{S} \frac{f_y}{\sqrt{f'_c}} \quad \text{ACI 12.8} \]

Where:
- \( A \) = Area of an individual longitudinal reinforcing bar
- \( S \) = Spacing between longitudinal reinforcing bars
- \( f_y \) = yield stress of steel reinforcement
- \( f'_c \) = compressive strength of concrete

These two equations may be combined into the following form:

\[ l_{db} \geq \frac{A_{frp} f_{frp}}{\left( \frac{A_c}{grid} / \frac{S_{trans}}{vc} \right) \sqrt{vc}} \]

Where:
- \( A_{frp} \) = Area of longitudinal FRP reinforcing bar
- \( A_c/grid \) = Area of concrete enclosed by one pair of longitudinal and transverse bars (1 grid). 
- \( f_{frp} \) = design stress of reinforcement
- \( S_{trans} \) = Spacing between transverse frp bars of reinforcing grid.
- \( vc \) = allowable concrete shear stress
To prevent a splitting failure in the concrete, the force in the reinforcement bars must not exceed the force which can be transmitted by shear in the concrete between the longitudinal and transverse bars. Due to the smooth upper and lower surfaces and the width of the individual bars which make up the FRP grids, the area of concrete available to resist shearing forces is significantly less than the product of the transverse bar spacing times the longitudinal bar spacing. In order to determine the concrete shear area, the projected area of the FRP bars must be subtracting from the area calculated using the nominal transverse and longitudinal bar spacing.

For in-line grid, the formula for calculating the concrete shear area per grid is as follows:

\[ A_{c}/\text{grid} = (S_{\text{long}} - w_{\text{long}}) \times (S_{\text{trans}} - w_{\text{trans}}) \]

Taking the FRP bar width to be equal to the average FRP bar depth, for equal longitudinal and transverse bar spacing, this equation becomes:

\[ A_{c}/\text{grid} = (S - d_{\text{avg}})^2 \]

Multiplying by the number of grids overlapped at the critical section, the total available concrete shear area is as follows:
Total $A_c = N \times (S - d_{avg})^2$

(Where $N$ is the number of grids overlapped.)

In contrast, for offset grid with equal longitudinal and transverse bar spacings, the available concrete shear area is as follows:

$$A_c/\text{grid} = (S - d_{avg})^2 - (S - d_{avg}) \times d_{avg}$$

Note that ACI recommends that the transverse members from the two grid sections at a splice should be offset by at least 50 mm (2"). This provision has the effect of reducing the amount of concrete available between the FRP grid bars to resist the shear forces. Referring to Figure 50, note that when the transverse members are offset from each other, the concrete area available to resist the shear forces in the connection is reduced significantly. Approximately 1.5 offset grids have the same shear capacity as a single in-line grid.
For example, in a grid composed of H19 bars ($d_{avg} = 20\text{mm}$) with 100mm longitudinal and transverse bar spacing, the available concrete shear area is as follows:

**Offset Grid**: $A_t/\text{grid} = (100-20)^2 - (100-20) \times 20 = 4800 \text{ sq.mm.}$

**In-line Grid**: $A_t/\text{grid} = (100-20)^2 = 6400 \text{ sq.mm.}$

**Ratio of available shear areas**:

$$\text{Offset Grid/In-Line Grid} = \frac{4800}{6400} = 0.75$$
Compared to the theoretical area of each grid, the actual area concrete available to resist shear is significantly less.

Theoretical Grid Area = \( S^2 = 100^2 = 10,000 \) sq.mm.

Ratio of actual to theoretical areas:

\[
\text{Offset Grid/Theoretical} = \frac{4800}{10,000} = 0.48
\]
\[
\text{In-Line Grid/Theoretical} = \frac{6400}{10,000} = 0.64
\]

The difference between off-set and in-line grids is even more pronounced for larger bar sizes. Due to the reduced concrete shear area with offset grid, in-line transverse members are recommended.

**Allowable Shear Stress (\( v_c \)) in Splices**

Based upon research by Lloyd and Kesler[35], ACI has recommended the following values for \( v_c \) to prevent splitting failure of reinforcement splices:

- deformed wire mesh \( v_c = 5\sqrt{f'_c} \)
- plain welded wire mesh \( v_c = 3.7\sqrt{f'_c} \)

\( f'_c \) in psi

The experimental data obtained from splice/development length tests conducted on beams reinforced with FRP grids, shown in the following table, confirms that the ACI recommendations are conservative when applied to FRP grid.
<table>
<thead>
<tr>
<th>Beam I.D.</th>
<th>Bar Type</th>
<th>Splice Length # Grids</th>
<th>Grid Length mm (in.)</th>
<th>Bar Force kN (Kips)</th>
<th>% Ult Bar Force</th>
<th>f'c MPa (psi)</th>
<th>$A_v$/ Grid mm² (in²)</th>
<th>$v_u$ / $\sqrt{f'c}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>QH1 UNH</td>
<td>H10</td>
<td>1</td>
<td>100 (3.94)</td>
<td>28.3 (6.4)</td>
<td>53%</td>
<td>33.1 (4800)</td>
<td>8100 (12.6)</td>
<td>7.3</td>
</tr>
<tr>
<td>QC1 UNH</td>
<td>C10</td>
<td>1</td>
<td>100 (3.94)</td>
<td>22.9 (5.2)</td>
<td>32%</td>
<td>33.1 (4800)</td>
<td>8500 (13.2)</td>
<td>5.6</td>
</tr>
<tr>
<td>QC2 UNH</td>
<td>C10</td>
<td>2</td>
<td>100 (3.94)</td>
<td>38.6 (8.7)</td>
<td>53%</td>
<td>33.1 (4800)</td>
<td>8500 (13.2)</td>
<td>4.7</td>
</tr>
<tr>
<td>RH1 UNH</td>
<td>H19</td>
<td>1</td>
<td>100 (3.94)</td>
<td>40.0 (9.0)</td>
<td>19%</td>
<td>33.1 (4800)</td>
<td>6400 (9.9)</td>
<td>13.0</td>
</tr>
<tr>
<td>RC1 UNH</td>
<td>C19</td>
<td>2</td>
<td>100 (3.94)</td>
<td>44.8 (10.1)</td>
<td>22%</td>
<td>33.1 (4800)</td>
<td>7700 (11.9)</td>
<td>6.1</td>
</tr>
<tr>
<td>RH2 UNH</td>
<td>H19</td>
<td>2</td>
<td>100 (3.94)</td>
<td>53.0 (11.9)</td>
<td>26%</td>
<td>33.1 (4800)</td>
<td>6400 (9.9)</td>
<td>8.7</td>
</tr>
<tr>
<td>RH3 UNH</td>
<td>H19</td>
<td>2</td>
<td>150 (5.91)</td>
<td>62.6 (14.1)</td>
<td>31%</td>
<td>33.1 (4800)</td>
<td>10,400 (16.1)</td>
<td>6.8</td>
</tr>
<tr>
<td>RH4 UNH</td>
<td>H19</td>
<td>3</td>
<td>100 (3.94)</td>
<td>62.6 (14.1)</td>
<td>31%</td>
<td>33.1 (4800)</td>
<td>6400 (9.9)</td>
<td>6.3</td>
</tr>
<tr>
<td>SC3 FHWA</td>
<td>C19</td>
<td>4.5</td>
<td>100 (3.94)</td>
<td>95.0 (21.3)</td>
<td>47%</td>
<td>41.4 (6000)</td>
<td>6600 (10.2)</td>
<td>6.0</td>
</tr>
<tr>
<td>SH3 FHWA</td>
<td>H19</td>
<td>4.5</td>
<td>100 (3.94)</td>
<td>100.9 (22.7)</td>
<td>49%</td>
<td>41.4 (6000)</td>
<td>7800 (12.1)</td>
<td>5.5</td>
</tr>
<tr>
<td>SC5 FHWA</td>
<td>C19</td>
<td>8.5</td>
<td>100 (3.94)</td>
<td>104.5 (23.5)</td>
<td>51%</td>
<td>41.4 (6000)</td>
<td>6600 (10.2)</td>
<td>3.5</td>
</tr>
</tbody>
</table>

The transverse grid spacing was equal to 100mm for all test specimens except beam SH3, which had a transverse grid spacing of 150mm. Note that the test specimens with the larger splice lengths had the lowest values for $v_u$. This is possibly an indication that splitting failure was precipitated by stress concentrations at the ends of the FRP bars. The length of the larger splices relative to the length of the constant moment section of the test beams also
appeared to influence capacity; the curvature of the beam promoting splitting failure. Test specimen RH1 exhibited significantly greater concrete shearing stresses than the other specimens. This greater resistance to splitting failure might be due to aggregate interlock in the failure plane.

Excluding test specimen RH1, the average value for $v_{cs}$ was found to be 6.0 with a sample standard deviation of 1.4. A conservative estimate for $v_c$ would be:

$$v_c = (v_{cs})_{AVG} - \text{sample Std.Dev}(v_{cs}) = (6.0 - 1.4)\sqrt{f'c} = 4.6\sqrt{f'c}$$

Which falls between the values used by ACI 12.8 and ACI 12.7.2.

The proposed design equation for the development length of FRP grid is as follows:

$$l_{db} \geq \frac{A_{frp} f_{frp}}{\left(\frac{A_c}{grid} \right. \left. \frac{S_{trans}}{\sqrt{f'c}} \right)}$$

or

$$l_{db} \geq 0.22 \frac{A_{frp} f_{frp}}{\left(\frac{A_c}{grid} \frac{S_{trans}}{\sqrt{f'c}} \right)}$$
Splice Design Considerations

The full tension capacity of the FRP bars cannot readily be fully developed by lap splices. This is not necessarily a problem since the FRP bars are not normally designed to be fully stressed. Due to deflection restrictions and limits in concrete stresses, the stresses in FRP reinforcements typically have been limited to 25% to 40% of ultimate.\[19][20][23][32][41][42][52][55][57]\ Since ACI 318-89 allows the required development length to be multiplied by \((A_{reqd}/A_{provd})\), the actual development length would be reduced to 40% of that required to develop the full capacity of the FRP reinforcement.

For example, the development length of a H19 FRP bar stressed to 25% of ultimate, using 34.5 MPa (5000 psi) concrete, would be calculated as follows:

\[
A_{frp} = 0.465 \text{ in}^2 \\
F_{ul} = 99,000 \text{ psi} \\
f_{frp} = 0.25(99,000) = 24,750 \text{ psi} \\
f'_{c} = 5000 \text{ psi} \\
S = 100 \text{mm} = 4.0 \text{ in} \\
A_{c} = 6400 \text{ sq.mm.} = 9.92 \text{ in}^2
\]

\[
l_{db} = 0.22 \frac{(0.465)24,750}{\sqrt{\frac{9.92}{4.0} \times 5000}} = 14.4 \text{ inches}
\]
The development length should be rounded up to the next nearest whole # of grids, therefore \( l_{db} = 16 \) in. (4 grids @ 4 in.).

The required splice length is then calculated from the development length. Since ACI 318-89 Section 12.19 requires that the basic development length be multiplied by 1.5 when splicing smooth welded wire fabric and Section 12.18 requires that a factor of 1.3 when splicing deformed wire fabric, a factor of 1.5 should be used for FRP grid. Note that The 1.5 factor is applied to the calculated value for \( l_{db} \), not the rounded value.

Therefore the required splice length would be calculated as:

\[
\ell_s = 1.5 \times l_{db} = 1.5(15.1) = 22.7 \text{ in.} \quad \text{use 24 in}
\]
(even # of grids)
Summary of Splice Test Results - FRP Grid

The results from these tests indicate that the ACI 318 requirements for deformed wire mesh (Section 12.7) and smooth wire mesh (Section 12.8) envelope those for FRP grids. At low loads, the FRP grids behave like deformed wire mesh, but at loads approaching 50% of ultimate the behavior is closer to that of the smooth wire mesh.

The "Lap" splices used with the FRP grid performed satisfactorily, provided there were no voids in the concrete in the vicinity of the splice.

With one exception, the ACI provisions for smooth welded wire mesh may be applied to FRP grid. The exception to the ACI provisions is as follows: The transverse grid members should not be offset from each other. As shown in the following picture and in Figure 50, offsetting the grids increases the possibility of voids in the concrete and reduces the shear capacity of the concrete in the vicinity of the splice, with a resulting increase in development length requirements. Tests on splices using in-line grids resulted in more compact connections than those which used offset grids.

Note that the offsetting of the transverse bars also makes concrete placement extremely difficult, tending to promote the presence of voids. The presence of voids renders the concrete in the splice region ineffective in resisting the required shear forces.
Photograph 9 - Splice Failure Due To Reduced Concrete Shear Capacity
CHAPTER V
SUMMARY AND CONCLUSIONS

5.0 Summary of Work

The following work was completed during the course of this research:

5.1 FRP Material and Mechanical Properties

The material and mechanical properties of the FRP reinforcements were obtained from experiments conducted on the full cross section of the individual FRP bars. Since these tests were conducted upon full bars, the modulus of elasticity, ultimate stress and ultimate strains were all able to be obtained for the various FRP bars.

These tests indicated that ultimate stress and ultimate strain decreased with increasing bars size.

Tests were conducted to determine the actual volume fractions of the individual components comprising the FRP bars. Calculated material and mechanical properties, based upon derived equations and the results of the volume fraction component tests, correspond to those obtained experimentally and also to the values provided by the manufacturer of the FRP grids.
5.2 Design Considerations

The design moment capacity of a member is taken as the minimum of that obtained from the reinforcement failure, concrete compression failure, and the shear capacity equations. Reinforcement failure will govern only in situations where the reinforcing ratio is very low. In these instances, deflection limitations may ultimately control the design.

Equations were presented for calculating the neutral axis in FRP reinforced concrete. There were two different equations for calculating the neutral axis of a reinforced concrete section, one for when the tension reinforcement fails, and the other for when the tension reinforcement remains elastic. These equations are based upon the equivalent rectangular stress block, and assume that plane sections remain plane.

The shear capacity of FRP reinforced beams was predicted by the equation first proposed by Zsutty.[63] This equation provides a more accurate prediction of shear capacity than the ACI equations, for moderate and long beams. Among other items, the Zsutty equation takes into account the shear-span to depth ratio.

In addition, methods were presented to determine if anchorage failure needs to be addressed. Typically, anchorage failure will not govern the capacity of a FRP reinforced member. However, if the area of concrete
resisting shear ($A_s$) is reduced, due to the presence of voids or improperly placed reinforcements, anchorage failure may result.

Deflections were calculated using the ACI 318 equations derived by Branson and the CEB bilinear method. As shown in Figure 47 and Figure 48 (Chapter IV), the CEB bilinear method provides a better estimation of deflections in FRP reinforced concrete beams compared to the ACI method. Due to the low modulus of elasticity for the FRP reinforcement, there is almost no transition from the uncracked to the fully cracked section properties.

5.3 Summary of Experimental Test Results

Summary of Flexural Testing of FRP Reinforced Beams

Table 26 contains a summary of the results from the flexural testing of FRP reinforced concrete beam. The first series of tests was designed to show that FRP reinforced concrete beams demonstrate a bending behavior similar to that of beams with steel reinforcement. There was no evidence of relative slip between the transverse members of the reinforcement and the concrete, indicating that the transverse bars adequately transferred the tensile forces in the reinforcements to the concrete. Each of the concrete beams reinforced with FRP responded consistently with respect to the phases that characterize the brittle failure sequence of over-reinforced concrete beams.
After the successful completion of the first series of tests, the cyclical test portion of the program was initiated and completed. This portion of the research program has shown that FRP reinforced concrete beams demonstrate a cyclical flexural loading behavior similar to that of beams with steel reinforcement. The FRP reinforced beams do not accumulate excessive damage with each load/unload cycle.

### Table 26: Summary of Test Results

<table>
<thead>
<tr>
<th>Beam I.D.</th>
<th>ρ</th>
<th>a/d</th>
<th>Predict Max. Load (kips)</th>
<th>Actual Max. Load (kips)</th>
<th>Ratio Act/Predict %</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>YH1</td>
<td>0.0027</td>
<td>2.8</td>
<td>14.1</td>
<td>14.0</td>
<td>99 %</td>
<td>Shear</td>
</tr>
<tr>
<td>ZH1</td>
<td>0.0029</td>
<td>4.4</td>
<td>11.7</td>
<td>11.7</td>
<td>100 %</td>
<td>Comp.</td>
</tr>
<tr>
<td>QC1</td>
<td>0.0061</td>
<td>7.5</td>
<td>4.8</td>
<td>4.7</td>
<td>98 %</td>
<td>Shear</td>
</tr>
<tr>
<td>RC1</td>
<td>0.0087</td>
<td>4.5</td>
<td>14.1</td>
<td>15.0</td>
<td>106 %</td>
<td>Shear</td>
</tr>
<tr>
<td>SC1</td>
<td>0.0092</td>
<td>7.1</td>
<td>18.9</td>
<td>19.2</td>
<td>102 %</td>
<td>Shear</td>
</tr>
<tr>
<td>TH1</td>
<td>0.0095</td>
<td>7.5</td>
<td>4.1</td>
<td>4.2</td>
<td>103 %</td>
<td>Comp.</td>
</tr>
<tr>
<td>SH1</td>
<td>0.0117</td>
<td>7.3</td>
<td>16.1</td>
<td>17.0</td>
<td>106 %</td>
<td>Comp.</td>
</tr>
<tr>
<td>TH4</td>
<td>0.0143</td>
<td>7.5</td>
<td>4.6</td>
<td>5.1</td>
<td>110 %</td>
<td>Comp.</td>
</tr>
<tr>
<td>RH1</td>
<td>0.0167</td>
<td>3.4</td>
<td>15.5</td>
<td>16.0</td>
<td>103 %</td>
<td>Shear</td>
</tr>
</tbody>
</table>
Summary of Flexural Testing of FRP Reinforced Bridge Deck Slabs

The next portion of the test program involved the investigation of the flexural behavior of FRP reinforced bridge deck slabs. These full scale test slabs were designed to be direct replacements for concrete bridge deck slabs reinforced with steel bars. All FRP grid reinforced slabs significantly exceeded the design loads. The FRP reinforced slabs performed satisfactorily when subjected to multiple load/unload cycles. The FRP grid reinforced slabs tested during the limited scale cyclical test portion of this program did not exhibit stiffness degradation. This testing showed that FRP grid reinforced slabs can be designed to satisfy AASHTO serviceability requirements. The results of the bridge deck tests are also shown in Table 18.

Splice/Development Length Requirements

The research presented in this portion of the test program was directed towards determining development length and splice requirements for FRP grids used in the reinforcement of concrete structures. Based upon tests conducted at the University of New Hampshire and the FHWA TFHRC and research into the background information contained in the ACI Code, a design formula for calculating the development length requirements for FRP grids was derived.
This equation, presented in on page 139, Chapter IV, is based upon the area of concrete available to resist shear at the critical section of the grid. The proposed value for allowable shear is enveloped by the ACI design equations for deformed welded wire mesh and the design equations for smooth welded wire mesh.

The "Lap" splices used with the FRP grid performed satisfactorily, provided there were no voids in the concrete in the vicinity of the splice.

With one exception, the ACI provisions for welded wire mesh may be applied to FRP grid. The exception to the ACI provisions is as follows: The transverse grid members should not be offset from each other. As shown Figure 50 (Chapter 4), offsetting the grids increases the possibility of voids in the concrete and reduces the shear capacity of the concrete in the vicinity of the splice, with a resulting increase in development length requirements. Tests on splices using in-line grids yielded more compact connections than those which used offset grids.

5.4 Conclusion

FRP grids are a potentially viable replacement for steel as a reinforcement for concrete. The flexural behavior of FRP reinforced structures can be predicted such that ACI and AASHTO serviceability requirements are satisfied. The main problems that will influence the
acceptance of the use of FRP as a reinforcement for concrete is the lack of "Tension Reinforcement Yielding" and the low modulus of elasticity exhibited by FRP reinforcements. The low modulus of elasticity will result in larger deflections than in a concrete member reinforced with an equal strength steel reinforcement. The brittle behavior and low modulus of elasticity of FRP reinforcements requires that concrete shear, deflections, and concrete compression be taken into greater consideration in the design of structures. However, since the FRP materials behaves in a predictable manner, and is not susceptible to corrosion, the use of FRP as a concrete reinforcement will continue to expand.

5.5 Future Work

There are several significant areas in which no research has yet be completed: Long-term loading performance, shear capacity, and fatigue testing of FRP reinforced concrete structures all require further investigation. Strength reduction factors for use with FRP reinforced concrete must be determined. In addition, additional work is required to increase the modulus of elasticity of the FRP in order to reduce deflections and decrease the reinforcement cross sectional area.
LIST OF REFERENCES


[43] NEFCOM Corporation, "NEFMAC - Technical Leaflet 1, New Fiber Composite Material for Reinforcing Concrete", Tokyo, Japan, 1988


[54] Ivan A. Rubinsky and Andrew Rubinsky, "A Preliminary Investigation of the Use of Fibre-Glass for Prestressed Concrete", Magazine of Concrete Research, September 1954.


APPENDIX - SAMPLE CALCULATIONS

A. Splice/Development Length Beam Analysis

The moment and shear diagrams for the splice/development length tests are shown in Figure 51. The splices were located in the constant moment section of the test beams.

Figure 51 - Shear and Moment Diagram
BEAM QC

Reinforcement:
2 C10 FRP bars
E_{frp} = 13000 ksi
f_{wh} = 210 ksi
Total A_{frp} = 0.156 in²

Cross Section:
b = 8"  h = 4"  d = 3.22"
shear span = 24"

Concrete Properties:
f'c = 4800 psi
E_c = 3900 ksi  β_t = 0.81  ε_{cu} = 0.003

CALCULATIONS:

n = E_s/E_c = 13000/3900 = 3.3

ρ = A_{frp} / (bd) = 0.156 / (8 * 3.22) = 0.0061

Flexural Capacity: Concrete Compression Failure

kd = 0.76",  β_t kd = 0.81(0.76) = 0.62"

M_n = 0.85f'_c(β_t kd)b(d-β_t kd/2)
     = 0.85*4.8*0.62*8*(3.22-(0.62/2)) = 58.9 in-kip

I_c = b(kd)^3/3 + N_a(d - kd)^2
I_c = 8(0.76)^3/3 + 3.3*0.156*(3.22-0.76)^2 = 4.29 in⁴

Shear Capacity:

V_c = 2bd(f'c)^{1/2} = 2 * 8 * 3.22 * (4800)^{1/2} = 3570 lb.

V_c = 59bd(f'_cρd/a)^{1/3} = 59*8*3.22(4800*0.0061*3.22/24)^{1/3}
     = 2400 lb = 2.4 kip

M_n = Shear Span * V_c = 24 in * 2.4 kip
     = 57.6 in-kip < 58.9 in-kip

Shear failure is likely to govern.

P_{căl} = 4.8 kips  P_{measured} = 4.7 kips
BEAM QH

Reinforcement:
2 H10 FRP bars
\( E_{frp} = 6080 \text{ ksi} \)
\( f_{uk} = 100 \text{ ksi} \)
Total \( A_{frp} = 0.242 \text{ in}^2 \)

Cross Section:
b = 8"  h = 4"  d = 3.18"
shear span = 24"

Concrete Properties:
\( f'c = 4800 \text{ psi} \)
\( E_c = 3900 \text{ ksi} \)
\( \beta_1 = 0.81 \)
\( \varepsilon_{cu} = 0.003 \)

CALCULATIONS:

\( n = E_s/E_c = 6080/3900 = 1.6 \)

\( \rho = \frac{A_{frp}}{(bd)} = 0.242/ (8 * 3.18) = 0.0095 \)

Flexural Capacity: Concrete Compression Failure

\[ kd = 0.65", \quad \beta_1 kd = 0.81(0.65) = 0.53" \]

\[ Mn = 0.85f'c(\beta_1 kd)b(d - \beta_1 kd/2) \]
\[ = 0.85*4.8*0.53*8*(3.18-(0.53/2)) = 51.1 \text{ in-kip} \]

\[ Icr = b(kd)^3/3 + Na_t(d - kd)^4 \]
\[ Icr = 8(0.65)^3/3 + 1.6*0.242*(3.18-0.65)^2 = 3.29 \text{ in}^4 \]

Shear Capacity:
\[ Vc = 2bd(f'c)^{1/2} = 2 * 8 * 3.22 (4800)^{1/2} = 3570 \text{ lb.} \]

\[ Vc = 59bd(f'c\rho d/a)^{1/3} = 59*8*3.18(4800*0.0095*3.18/24)^{1/3} = 2780 \text{ lb} = 2.78 \text{ kip} \]

\[ Mn = \text{Shear Span} * Vc = 24 \text{ in} * 2.78 \text{ kip} = 66.7 \text{ in-kip} \]

Compression Failure is most likely to occur.

\[ P_{calc} = 4.3 \text{ kips} \quad P_{measured} = 4.5 \text{ kips} \]
BEAM RC

Reinforcement:
2 C19 FRP bars
$E_{frp} = 12300$ ksi
$f_{uh} = 185$ ksi
Total $A_{frp} = 0.496$ in

Cross Section:
b = 8" h = 8" d = 7.13"
shear span = 32"

Concrete Properties:
f'c = 4800 psi
$E_c = 3900$ ksi
$\beta_1 = 0.81$ $\epsilon_{cu} = 0.003$

CALCULATIONS:
n = $E_s/E_c = 12300/3900 = 3.2$

$\rho = A_{frp}/(bd) = 0.496 / (8 \times 7.13) = 0.0087$

Flexural Capacity: Concrete Compression Failure

$kd = 1.91", \quad \beta_1kd = 0.81(1.91) = 1.55"$

$M_n = 0.85f'_c(\beta_1kd)b(d-\beta_1kd/2) = 0.85 \times 4.8 \times 1.55 \times 8 \times (7.13-(1.55/2))$

$M_n = 321$ in-kip

$I_{cr} = b(kd)^3/3 + N_a(d - kd)^2$

$I_{cr} = 8(1.91)^3/3 + 3.2 \times 0.496 \times (7.13-1.91)^2 = 61.8$ in

Shear Capacity:

$V_c = 2bd(f'_c)^{1/2} = 2 \times 8 \times 7.13 \times (4800)^{1/2} = 7900$ lb.

$V_c = 59bd(f'_c\rho d/a)^{1/3} = 59 \times 8 \times 7.13(4800 \times 0.0087 \times 7.13/32)^{1/3}$

$= 7080$ lb = 7.08 kip

$M_n = \text{Shear Span} \times V_c = 32 \text{ in} \times 7.08$ kip

$= 226$ in-kip < 321 in-kip

Shear failure is most likely to govern.

$P_{calc} = 14.1$ kips $P_{measured} = 15.0$ kips
BEAM RH

Reinforcement:
2 H19 FRP bars
$E_{frp} = 6080$ ksi
$f_{uh} = 99$ ksi
Total $A_{frp} = 0.930$ in$^2$

Cross Section:
b = 8" h = 4" d = 6.98"
shear span = 32"

Concrete Properties:
$f'c = 4800$ psi
$E_c = 3900$ ksi $\beta_1 = 0.81 \; \epsilon_{cu} = 0.003$

CALCULATIONS:
n = $E_s/E_c = 6080/3900 = 1.6$

$\rho = A_{frp}/(bd) = 0.93/(8 \times 6.98) = 0.0167$

Flexural Capacity: Concrete Compression Failure

$kd = 1.82"$, $\beta_1 kd = 0.81(1.82) = 1.47"$

$M_n = 0.85f'_c(\beta_1 kd)b(d - \beta_1 kd/2) = 0.85 \times 4.8 \times 1.47 \times 8 \times (6.98 - (1.47/2))$

$M_n = 300$ in-kip

$I_{cr} = b(kd)^3/3 + nA_s(d - kd)^4$

$I_{cr} = 8(1.82)^3/3 + 1.6 \times 0.93 \times (6.98 - 1.82)^2 = 55.7$ in$^4$

Shear Capacity:

$V_c = 2bd(f'_c)^{1/2} = 2 \times 8 \times 6.98 \times (4800)^{1/2} = 7740$ lb = 7.74 kip

$V_c = 59bd(f'_c\rho d/a)^{1/3} = 59 \times 8 \times 6.98 \times (4800 \times 0.0167 \times 6.98/32)^{1/3}$

$= 8550$ lb = 8.85 kip

$M_n = \text{Shear Span} \times V_c = 32 \text{ in} \times 7.74 \text{ kip} = 248$ in-kip

Shear Failure is most likely to occur.
$P_{calc} = 15.5$ kips $P_{measured} = 16.0$ kips
BEAM 88

Reinforcement:
1 #5 Grade 60 steel bar

\[ E_s = 29000 \text{ ksi} \]
\[ f_y = 60 \text{ ksi} \]
\[ A_{bar} = 0.31 \text{ in}^2 \]

Cross Section:
\[ b = 12'' \quad d = 6.69'' \quad a = 48'' \]

Concrete Properties:
\[ f'c = 6000 \text{ psi} \quad E_c = 4400 \text{ ksi} \]
\[ \beta_1 = 0.75 \quad \epsilon_{cu} = 0.003 \]

CALCULATIONS:
\[ n = \frac{E_s}{E_c} = \frac{29000}{4400} = 6.6 \]
\[ \rho = \frac{A_{bar}}{(bd)} = \frac{0.31}{(12 \times 6.69)} = 0.0039 \]
\[ \rho_b = 0.85 \beta_1 \left( \frac{f'c}{f_y} \right) \left( \frac{\epsilon_{cu}}{\epsilon_{cu} + (F_y/E_s)} \right) \]
\[ \rho_b = 0.85 \times 0.75 \times (6/60) \times \left( \frac{0.003}{\left(0.003+(60/29000)\right)} \right) = 0.0377 \]
\[ \rho < \rho_b \quad \text{tension reinforcement yields} \]

\[ k = (2 \rho n + (\rho n)^2)^{1/2} - \rho n \]
\[ = (2 \ast 0.0039 \ast 6.6) + (0.0039 \ast 6.6)^2 \]
\[ = 0.203 \]

\[ kd = 0.203 \times 6.69'' = 1.36'' \]

\[ I_{cr} = b (kd)^3/3 + n A_y (d - kd)^2 \]
\[ I_{cr} = 12 (1.36)^3/3 + 6.6 \times 0.31 \times (6.69 - 1.36)^2 = 68.2 \text{ in}^4 \]

Flexural Capacity:
\[ M_n = A_y f_y (d - (0.59 A_y f_y/b f'c)) \]
\[ M_n = 0.31 \times 60,000 \times (6.69 - (0.59 \times 0.31 \times 60,000)/(12 \times 6000)) \]
\[ = 115,000 \text{ in}-\text{lb} \]

Shear Capacity:
\[ V_c = 2 bd (f'c)^{1/2} = 2 \times 12 \times 6.69 (6000)^{1/2} = 12,400 \text{ lb.} \]
\[ V_c = 59 bd (f'c \rho d/a)^{1/3} = 59 \times 12 \times 6.69 (6000 \times 0.0039 \times 6.69/48)^{1/3} \]
\[ = 7020 \text{ lb} \]
\[ M_n = \text{Shear Span} \times V_c = 48 \text{ in} \times 7020 \text{ lb.} \]
\[ = 337,000 \text{ in}-\text{lb.} > 115,000 \]

Tension capacity of reinforcement governs capacity
\[ P_{cal} = 4.8 \text{ kips} \quad P_{measured} = 5.0 \text{ kips} \]
BEAM 8C

Reinforcement:
3 C19 FRP bars

\[ E_{frp} = 12400 \text{ ksi} \]
\[ f_{ub} = 185 \text{ ksi} \]
Total \( A_{frp} = 0.744 \text{ in}^2 \)

Cross Section:
\( b = 12" \quad d = 6.76" \quad a = 48" \)

Concrete Properties:
\( f'c = 6000 \text{ psi} \quad E_c = 4400 \text{ ksi} \)
\( \beta_1 = 0.75 \quad \epsilon_{cu} = 0.003 \)

CALCULATIONS:
\[ n = \frac{E_s}{E_c} = \frac{12400}{4400} = 2.8 \]
\[ \rho = \frac{A_{frp}}{(bd)} = 0.744 / (12 \times 6.76) = 0.0092 \]

Flexural Capacity: Concrete Compression Failure

\[ kd = 1.74", \quad \beta kd = 0.75(1.74) = 1.31" \]

\[ Mn = 0.85f'(\beta kd)(d - \beta kd/2) = 0.85 \times 6000 \times 1.31 \times 12 \times (6.76 - (1.31/2)) \]

\[ Mn = 490,000 \text{ in-lb} \]

\[ I_{cr} = b(kd)^3/3 + nA_v(d - kd)^2 \]

\[ I_{cr} = 12(1.74)^3/3 + 2.8 \times 0.744 \times (6.76 - 1.74)^2 = 73.6 \text{ in}^4 \]

Shear Capacity:

\[ V_c = 2bd(f'c/2)^{1/2} = 2 \times 12 \times 6.76 (6000)^{1/2} = 12,600 \text{ lb.} \]

\[ V_c = 59bd(f'c \rho d/a)^{1/3} = 59 \times 12 \times 6.76 \times (6000 \times 0.0092 \times 6.76/48)^{1/3} = 9480 \text{ lb} \]

\[ Mn = \text{Shear Span} \times V_c = 48 \text{ in} \times 9480 \text{ lb.} = 455,000 \text{ in-lb.} \]

Shear failure is likely to govern.

\[ P_{arc} = 18.9 \text{ kips} \quad P_{measured} = 19.2 \text{ kips} \]
Anchorage Calculations for Beam SC

\[
S = 3.94 \text{ in} \\
\delta = 6.76 \text{ in} \\
k_d = 1.74 \text{ in} \\
j_d = 5.89 \text{ in} \\
N = 3 \text{ Bars} \\
d_{avg} = 0.49 \text{ in} \\
A_c = (S-d_{avg})^2 = 11.88 \\
f'c = 6000 \text{ psi} \\
3.5\sqrt{f'c} = 271 \\
P = 19.2 \text{ kips}
\]

Therefore \( V = 9450 \text{ lb} \) (Calculated Capacity)

\[
v = \frac{9450 \times 3.94}{(5.89 \times 3 \times 11.88)} = 177 < 271 \text{ OK}
\]

If concrete area reduced in half due to improperly placed grids at end of beam (For example, beam SC1).

\[
A_c = 5.9 \\
v = 354 > 271 \text{ NG} \\
\text{will have anchorage failure.}
\]
BEAM SH

Reinforcement:
2 H19 FRP bars

\[ E_{frp} = 6080 \text{ ksi} \]
\[ f_{ult} = 100 \text{ ksi} \]
Total \( A_{frp} = 0.930 \text{ in}^2 \)

Cross Section:
\[ b = 12" \]
\[ d = 6.61" \]
\[ a = 48" \]

Concrete Properties:
\[ f'c = 6000 \text{ psi} \]
\[ E_c = 4400 \text{ ksi} \]
\[ \beta_1 = 0.75 \]
\[ \epsilon_{cu} = 0.003 \]

CALCULATIONS:
\[ n = \frac{E_s}{E_c} = \frac{6080}{4400} = 1.4 \]
\[ \rho = \frac{A_{frp}}{(bd)} = 0.93 / (12 \times 6.61) = 0.0117 \]

Flexural Capacity: Concrete Compression Failure

\[ kd = 1.39", \quad \beta_1 kd = 0.75(1.39) = 1.04" \]

\[ Mn = 0.85f'_c(\beta_1 kd)b(d-\beta_1 kd/2) \]
\[ = 0.85 \times 6000 \times 1.04 \times 12 \times (6.61 - (1.04/2)) \]
\[ Mn = 387,000 \text{ in-lb} \]

\[ Icr = b(kd)^3/3 + nA_x(d - kd)^2 \]
\[ Icr = 12(1.39)^3/3 + 1.4 \times 0.93 \times (6.61 - 1.39)^2 = 46.2 \text{ in}^4 \]

Shear Capacity:
\[ V_c = 2bd(f'_c)^{1/2} = 2 \times 12 \times 6.61 \times (6000)^{1/2} = 12,300 \text{ lb.} \]

\[ V_c = 59bd(f'_c \rho d/a)^{1/3} = 59 \times 12 \times 6.61 \times (6000 \times 0.0117 \times 6.61/48)^{1/3} = 9970 \text{ lb} \]
\[ Mn = \text{Shear Span} \times V_c = 48 \text{ in} \times 9970 \text{ lb.} \]
\[ = 479,000 \text{ in-lb} > 387,000 \text{ in-lb} \]

Compression Failure most likely to occur.

\[ P_{cal} = 16.1 \text{ kips} \]
\[ P_{measured} = 17.0 \text{ kips} \]
yB. Minimum Reinforcement Design

BEAM Y1

Reinforcement:
2 H10 FRP bars
$E_{\text{frp}} = 6080 \text{ ksi}$
$f_{\text{ult}} = 110 \text{ ksi}$
Total $A_{\text{frp}} = 0.242 \text{ in}^2$

Cross Section:
b = 8" h = 12" d = 11.06"
shear span = 31"

Concrete Properties:
f'c = 5530 psi
$E_c = 4200 \text{ ksi}$ $\beta_1 = 0.77$
$\epsilon_{cu} = 0.003$

CALCULATIONS:
$n = E_s/E_c = 6080/4200 = 1.4$

$\rho = A_{\text{frp}}/(bd) = 0.242/(8 * 11.06) = 0.0027$

Flexural Capacity: Tension Reinforcement Failure
$kd = 0.921", \quad \beta_1kd = 0.77(0.921) = 0.71"$
$M_n = 0.242*110*(11.06-(0.71/2)) = 285$
$M_n = 285 \text{ in-kip}$

Shear Capacity:
$V_c = 2bd(f'c)^{1/2} = 2 * 8 * 11.06 (5530)^{1/2} = 13,200 \text{ lb} = 13.2\text{kip}$

$V_c = 59bd(f'c\rho d/a)^{1/3} = 59*8*11.06(5530*0.0027*11.06/31)^{1/3}$
$= 9100 \text{ lb} = 9.1 \text{ kip}$

$V_c = 8*11.06[0.8+(100*0.0027)](5530)^{1/2} = 7,040 \text{ lb} = 7.04\text{kip}$
$M_n = \text{Shear Span} * V_c = 31 \text{ in} * 7.04 \text{ kip} = 218 \text{ in-kip}$

Shear Failure is most likely to occur.

$P_{\text{cal}} = 14.1 \text{ kips}$
$P_{\text{measured}} = 14.0 \text{ kips}$
C. Bridge Deck Analysis

Bridge Deck Cross Section

Cross Section:
b = 48"
h = 8.5"

Concrete Properties:
f'c = 5000 psi
Ec = 4000 ksi
β = 0.80
εcu = 0.003

shear span = 48" - (25"/2) = 35.5"
Reinforcement:
10 #5 Grade 60 steel bars
$E_s = 29000$ ksi
$f_y = 60$ ksi
$A_{bar} = 0.31$ in$^2$

Cross Section:
$b = 48"$
$d = 7.19"$

Calculations:

$n = \frac{E_s}{E_c} = \frac{29000}{4000} = 7.3$

$\rho = \frac{A_{bar}}{(bd)} = \frac{3.10}{(48 \times 7.19)} = 0.0090$

$\rho_b = 0.85 \beta_i \left( \frac{f_c}{f_y} \right) \left[ \frac{\epsilon_{cu}}{\epsilon_{cr} + (f_y/E_s)} \right]$

$\rho_b = 0.85 \times 0.80 \times (5/60) \times (0.003 / (0.003 + (60/29000))) = 0.0335$

$\rho < \rho_b$, tension reinforcement yields prior to concrete crushing

$k = (2\rho n + (\rho n)^2)^{1/2} - \rho n$

$k = ((2 \times 0.009 \times 7.3) + (0.009 \times 7.3)^2)^{1/2} - (0.009 \times 7.3) = 0.303$

$kd = 0.303 \times 7.19" = 2.18"$

$Icr = b(kd)^3/3 + nA_t(d - kd)^2$

$Icr = 48(2.18)^3/3 + 7.3 \times 3.10 \times (7.19 - 2.18)^2 = 734$ in$^4$

Flexural Capacity:

$M_n = A_t f_y (d - (0.59 A_s f_c'/b f_c'))$

$M_n = 3.10 \times 60 \times (7.19 - (0.59 \times 3.10 \times 60 / (48 \times 5)))$

$M_n = 1250$ in-kip

Shear Capacity:

$V_c = 2bd(f_c')^{1/2} = 2 \times 48 \times 7.19 \times (5000)^{1/2} = 48,800$ lb.

$V_c = 59bd(f_c' \rho d/a)^{1/3} = 59 \times 48 \times 7.19 \times (5000 \times 0.009 \times 7.19 / 37.5)^{1/3}$

$V_c = 41,800$ lb

$M_n = \text{Shear Span} \times V_c = 35.5 \text{ in} \times 41.8 \text{ kip}$

$M_n = 1484 \text{ in-kip} > 1250$ in-kip

Tension capacity of reinforcement governs capacity
DECK DC

Reinforcement:
12 C22 FRP bars
\( E_{fp} = 12300 \text{ ksi} \)
\( f_{\text{up}} = 178 \text{ ksi} \)
Total \( A_{\text{fp}} = 3.912 \text{ in}^2 \)

Cross Section:
\( b = 48'' \)
\( d = 7.09'' \)

CALCULATIONS:
\[ n = \frac{E_s}{E_c} = \frac{12300}{4000} = 3.1 \]
\[ \rho = \frac{A_{\text{fp}}}{(bd)} = \frac{3.912}{(48 \times 7.09)} = 0.0115 \]

Flexural Capacity: Concrete Compression Failure
\( k_d = 2.10'', \quad \beta_k d = 0.80(2.10) = 1.68'' \)
\[ M_n = 0.85f'_{\text{c}}(\beta_k d)(d - \beta_k d/2) = 0.85 \times 5 \times 1.68 \times 48 \times (7.09 - (1.68/2)) \]
\[ M_n = 2140 \text{ in-kip} \]

\[ I_{cr} = b(k_d)^{3/2} + nA_{\text{f}}(d - k_d)^2 \]
\[ I_{cr} = 48(2.10)^{3/2} + 3.1 \times (3.912)(7.09 - 2.10)^2 = 450 \text{ in}^4 \]

Shear Capacity:
\[ V_c = 2bd(f'c)^{1/2} = 2 \times 48 \times 7.09 \times (5000)^{1/2} = 48,100 \text{ lb.} \]
\[ V_c = 59bd(f'c\rho d/a)^{1/3} = 59 \times 48 \times 7.09 \times (5000 \times 0.0115 \times 7.09 / 37.5)^{1/3} \]
\[ = 44,500 \text{ lb} = 44.5 \text{ kip} \]
\[ M_n = \text{Shear Span} \times V_c = 35.5 \text{ in} \times 44.5 \text{ kip} \]
\[ = 1580 \text{ in-kip} < 2140 \text{ in-kip} \]
Shear failure is likely to govern.
Reinforcement:
12 H22 FRP bars

E_{frp} = 5985 ksi
f_{sh} = 92 ksi
Total A_{frp} = 7.44 in^2
Cross Section:
b = 48"
d = 6.99"

CALCULATIONS:
\[ n = \frac{E_s}{E_c} = \frac{5985}{4000} = 1.5 \]
\[ \rho = \frac{A_{frp}}{(bd)} = 7.44 / (48 \times 6.99) = 0.0222 \]

Flexural Capacity: Concrete Compression Failure
\[ kd = 2.02", \quad \beta_1 kd = 0.80(2.02) = 1.62" \]
\[ Mn = 0.85f'_c(\beta_1 kd)b(d - \beta_1 kd/2) = 0.85 \times 5 \times 1.62 \times 48 \times (6.99 - (1.62/2)) \]
\[ Mn = 2040 \text{ in-kip} \]

Icr = b(kd)^3/3 + nA_y(d - kd)^2
\[ Icr = 48(2.02)^3/3 + 1.5 \times 7.44 \times (6.99 - 2.02)^2 = 407 \text{ in}^4 \]

Shear Capacity:
\[ V_c = 2bd(f'_c)^{1/2} = 2 \times 48 \times 6.99 \times (5000)^{1/2} = 47,400 \text{ lb.} \]
\[ V_c = 59bd(f'_c \rho d/a)^{1/3} = 59 \times 48 \times 6.99 \times (5000 \times 0.0222 \times 6.99 / 37.5)^{1/3} \]
\[ = 54,300 \text{ lb} = 54.3 \text{ kip} \]

Mn = Shear Span \times V_c = 35.5 \text{ in} \times 47.4 \text{ kip}
\[ = 1680 \text{ in-kip} \]

Shear Failure is most equally likely to occur.