FLEXURE SHEAR RESPONSE IN FATIGUE OF FIBER REINFORCED CONCRETE BEAMS WITH FRP TENSILE REINFORCEMENT

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In Partial Fulfillment of the Requirements for the Degree Master of Science

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The undersigned, appointed by the Dean of the Graduate School, have examined the thesis entitled

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Master of Science

and hereby certify that in their opinion it is worthy of acceptance.

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Dr. P. Frank Pai
ACKNOWLEDGEMENTS

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As the days draw close to graduation, the author expresses his deepest gratitude to Jesus Christ, his parents, and his friends who encouraged him in his academic endeavors and achievements and who, all too often were told, “No, I really shouldn’t, I need to work on my thesis.” This work is dedicated to them with love.
ABSTRACT

The primary goal of this MODOT-sponsored investigation is to characterize the flexural shear response, fatigue behavior, and performance of fiber reinforced concrete flexural sections with fiber reinforced polymer tensile reinforcement for use in bridge deck applications. Results from a series of flexural fatigue tests, on plain and fiber reinforced concrete flexural specimens, are reported and discussed.

First, to establish the compressive response of concrete with differing fractions of polypropylene fibers by volume, individual compressive tests were performed on both lab-mix and ready mix concrete specimens. In the lab, concrete mixes with 0.0%, 0.5%, and 1.0% by volume, fibrillated polypropylene fibers were produced. Along with these, specimens from the various beam castings that were part of the Steel Free Bridge Deck project were also taken. These concrete ready-mix batches were either plain concrete or contained 0.5% fibers. Tests on both 6” standard cylinders and 4” cylinders were performed. The compression tests on the 6” cylinders were performed in standard open-loop testing apparatus and provided load and deflection data up to ultimate strength. The 4” cylinders were tested in closed-loop testing apparatus and provided post-peak load deflection data as well as pre-peak data.

After the compressive response of the fiber reinforced concrete was established, four point bending tests were performed on 36 lab cast beam specimens. The beam specimens had cross-sectional dimensions of 6”x9.5” (width x height). The span was 48” and the loading points were 8” apart (4” each side of the mid-point). The depth of reinforcement was 8”.

There were two castings of beams; one with plain concrete and one with 0.5% by volume polypropylene fibers, both using ready mix concrete. For each casting there were 18 beams; 6 with #8 GFRP, 6 with #4 GFRP, and 6 with #4 CFRP tensile reinforcement. For each set of 6 beams, two were tested under quasi-static loading until failure to
determine ultimate load capacity and ultimate deflection, two were tested at low fatigue, and two were tested at high fatigue. The low fatigue tests varied the applied loading cyclically from approximately 30%-60% of maximum load capacity and the high fatigue tests varied the loading from approximately 40%-80% of maximum load capacity. The fatigue cycles were continued until beam failure or 1.2 million cycles. A LabView® program was used to enable test-control and data-acquisition for the specialized type of flexural fatigue testing.

The fiber mixes performed similarly to the plain concrete control specimens except that the fiber reinforced beams failed in a more ductile manner. The rate of flexural stiffness degradation was slightly greater in the fiber reinforced specimens. The more rapid degradation may have been caused by lower strength concrete in the fiber reinforced beams instead of the addition of the fibers.
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<tr>
<td>a</td>
<td>depth of equivalent rectangular stress block = $\beta_1 c$</td>
</tr>
<tr>
<td>a</td>
<td>length of shear span</td>
</tr>
<tr>
<td>b</td>
<td>width of member</td>
</tr>
<tr>
<td>c</td>
<td>depth of neutral bending axis from the extreme concrete compression zone</td>
</tr>
<tr>
<td>d</td>
<td>depth to the centroid of flexural reinforcement from the extreme concrete compression zone</td>
</tr>
<tr>
<td>$f'_c$</td>
<td>28 day concrete compressive strength</td>
</tr>
<tr>
<td>$f_c$</td>
<td>concrete compressive strength</td>
</tr>
<tr>
<td>$f_u$</td>
<td>ultimate stress of reinforcement</td>
</tr>
<tr>
<td>$A_f$</td>
<td>area of FRP reinforcement</td>
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<tr>
<td>$A_{fb}$</td>
<td>area of FRP reinforcement required to provide of balanced flexural failure</td>
</tr>
<tr>
<td>$M_n$</td>
<td>nominal moment capacity</td>
</tr>
<tr>
<td>$M_s$</td>
<td>moment at service loads</td>
</tr>
<tr>
<td>$M_u$</td>
<td>ultimate moment capacity</td>
</tr>
<tr>
<td>$V_c$</td>
<td>concrete contribution the shear resistance</td>
</tr>
<tr>
<td>$V_s$</td>
<td>shear reinforcement (diagonal tension) contribution to the shear resistance</td>
</tr>
<tr>
<td>$\beta_1$</td>
<td>equivalent rectangular stress block factor</td>
</tr>
<tr>
<td>$\varepsilon_c$</td>
<td>concrete strain</td>
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<td>$\varepsilon_{fu}$</td>
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<td>-------------</td>
</tr>
<tr>
<td>( \phi_s )</td>
<td>curvature at service loads</td>
</tr>
<tr>
<td>( \phi_u )</td>
<td>curvature at ultimate strength</td>
</tr>
<tr>
<td>( \rho )</td>
<td>reinforcement ratio ([ = \text{(Area of reinforcement)} / (bxd)] )</td>
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<tr>
<td>( \rho_b )</td>
<td>balanced reinforcement ratio at ultimate strength</td>
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<th>Abbreviation</th>
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<tr>
<td>ACI</td>
<td>American Concrete Institute</td>
</tr>
<tr>
<td>AFRP</td>
<td>Aramid Fiber Reinforced Polymer</td>
</tr>
<tr>
<td>ASCE</td>
<td>American Society of Civil Engineers</td>
</tr>
<tr>
<td>CFRP</td>
<td>Carbon Fiber Reinforced Polymer</td>
</tr>
<tr>
<td>CSA</td>
<td>Canadian Standards Association</td>
</tr>
<tr>
<td>FRP</td>
<td>Fiber Reinforced Polymer</td>
</tr>
<tr>
<td>GFRP</td>
<td>Glass Fiber Reinforced Polymer</td>
</tr>
<tr>
<td>JSCE</td>
<td>Japan Society of Civil Engineers</td>
</tr>
<tr>
<td>MODOT</td>
<td>Missouri Department of Transportation</td>
</tr>
<tr>
<td>RC</td>
<td>Reinforced Concrete</td>
</tr>
<tr>
<td>UMC</td>
<td>University of Missouri – Columbia</td>
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<tr>
<td>UMR</td>
<td>University of Missouri - Rolla</td>
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CHAPTER 1 - INTRODUCTION

1.1 BACKGROUND INFORMATION

For many years concrete has been used as a preferred material in many structures including buildings, bridges, pavements, sewer and storm pipes, liquid holding tanks and others. As early as 200 B.C., the Romans discovered that by adding a particular volcanic ash from central Italy (pozzolanic ash) to their mortar mixtures of lime or gypsum and sand, that a hard, durable product was created. They became so skilled at the production of this concrete that some of their concrete structures still remain mostly intact 2,000 years later. Even at the time of the Romans, it was recognized that the great advantages of concrete were in its compressive strength and ability to be molded. Although concrete offered these advantages, it still presented the problem to designers that its tensile strength was very small compared to its compressive strength. The Roman architects overcame this problem by using concrete in the form of vaults and arches or in foundations; any place where large tensile stresses were not likely in the material. In the last century, designers again became fond of using concrete in structures. Advances in modern technology allowed the designers to add steel reinforcement to the concrete to overcome the tensile weakness inherent in concrete. But, with one problem solved another was created. The steel reinforcement, used to strengthen modern concrete structures, is subject to deterioration in corrosive environments resulting when bridge decks and other concrete structures are exposed to chlorides from de-icing salts or sea water.
For a time, epoxy coated steel rebar was the solution of choice to overcome the problem of chloride attack on the steel reinforcement. But in the 1980’s and 1990’s, it became more obvious that the epoxy coated rebar were not always performing as well as designers hoped because small nicks in the epoxy coating left the bars exposed to chlorides unless repaired. This led to the investigation of other alternatives to conventional steel reinforcing in concrete exposed to the chloride attack presented by de-icing salts and sea water.

Fiber reinforced polymer (FRP) rebar have been the subject of a significant amount of research in current years. Researchers have found that one of the major drawbacks to FRP reinforcement is their brittle failure at ultimate tensile strength. When FRP reinforcement is used as reinforcement in concrete, sudden failure of the reinforcement bars can lead to brittle structural failures.

This investigation of the flexural shear response of FRP reinforced concrete under fatigue loading has been performed as part of the Steel-Free Bridge Deck Project sponsored by the Missouri Department of Transportation (MODOT) and jointly executed at the University of Missouri – Columbia (UMC) and University of Missouri – Rolla (UMR) campuses. In this study, short, discrete, synthetic, fibrillated polypropylene fibers were added to the concrete matrix. The goal of the addition of fibers to the concrete mix was to decrease fatigue deterioration and to provide a more ductile ultimate strength failure.
1.2 SUMMARY OF FLEXURE SHEAR RESEARCH WORK DONE AT UMC

Three parts were included in the experimental investigation that forms the basis of this thesis. The first part involved the characterization of the compressive behavior of concrete with short, discrete, fibrillated, polypropylene fibers. The second part established the mechanical characteristics of the FRP tensile reinforcement. The final part involved four-point flexural shear bending tests performed on fiber reinforced concrete beams with FRP tensile reinforcement.

1.2.1 CHARACTERIZATION OF THE COMPRESSIVE BEHAVIOR OF CONCRETE WITH SHORT, DISCRETE, FIBRILLATED, POLYPROPYLENE FIBERS

Compressive tests were performed on both lab-mixed and ready mix concrete cylinders. In the lab, concrete mixes with 0.0%, 0.5%, and 1.0% by volume fibrillated polypropylene fibers were produced. Test cylinders from the various beam castings that were part of the Steel-Free Bridge Deck project were also taken. These concrete mixes were either plain concrete or contained 0.5% fibers. Compressive tests on both 6” standard cylinders and 4” cylinders were performed. The tests on the 6” cylinders were performed in standard open-loop testing apparatus that provided load and deflection data up to ultimate strength. The 4” cylinders were tested in closed-loop testing apparatus and provided post-peak load deflection data as well as pre-peak data.

1.2.2 FOUR POINT BENDING TESTS

Four-point bending tests were performed on 36 symmetric beam specimens. The beam specimens had cross-sectional dimensions of 6”x9.5” (width x height). The support
points were 48” apart and the top loading points were 8” apart (4” each side of the midpoint). The depth of reinforcement was 8” (see Figure 3.5-1).

There were two castings of beams; one with plain concrete and one with 0.5% by volume polypropylene fibers. For each casting there were 18 beams: six with #8 GFRP, six with #4 GFRP, and six with #4 CFRP tensile reinforcement. For each set of six beams, two were tested under quasi-static loading until failure to determine ultimate load capacity and ultimate deflection. Two were tested at a lower fatigue stress level, and two were tested at a higher fatigue stress level. The fatigue specimens were subjected to cyclic loading over a percentage range of the ultimate load capacity of the original beams. The lower level fatigue tests varied the loading cyclically from approximately 30%-60% of the original maximum load capacity and higher level fatigue tests varied the loading cyclically from approximately 40%-80% of the original maximum load capacity. The fatigue cycles were applied at a rate of five cycles per second. It should be noted that data, for all 36 beams that were tested, is not represented in this work. In addition, some beams failed due to sudden equipment malfunction and were excluded from the presented results.

1.3 Objectives of the Investigation

The objectives of this experimental investigation are to:

- Determine the flexural shear fatigue performance of concrete reinforced with FRP rebar.
• Investigate the possibility of providing a more ductile failure mechanism, for FRP reinforced concrete at ultimate strength, by the addition of polypropylene fibers.
• Investigate the effect of the polypropylene fibers in improving fatigue degradation of FRP reinforced concrete.

1.4 ORGANIZATION OF THIS THESIS REPORT

The work presented in this thesis includes the results of the laboratory investigation with an emphasis on …
- stiffness degradation of the beam specimens under fatigue loading
- ductility of the specimens at ultimate strength

A review of published research articles and data is presented in Chapter 2. Experimental preparations and procedures are outlined in Chapter 3. Concrete compression test results and discussion are presented on Chapter 4. FRP tension test results and discussion are presented in Chapter 5. Flexure shear test results are presented in Chapter 6. Conclusions, based on the experimental results and analysis, are outlined in Chapter 7 along with recommendations for future research.
CHAPTER 2 - REVIEW OF LITERATURE

2.1 INTRODUCTION

For over 100 years, concrete has been recognized for its high compressive strength, durability, and the ease with which it may be molded into almost any shape. It has also been equally understood that concrete possesses inherent drawbacks such as low tensile strength, low strain capacity and low ductility. The tensile strength of concrete is typically in the range of 10% of the compressive strength and strain at ultimate strength of concrete is typically considered to be 0.003. The most common method for overcoming the inherent “weaknesses” of plain concrete has been the addition of steel bars as reinforcement. Steel is most typically added in anticipated tension zones of the concrete to provide the lacking tensile strength. Current design philosophy also requires placement of steel in the concrete such that failure will take place via the tension yielding of steel (ductile) as opposed to the compression crushing of concrete (brittle).

Over the years however, it has become quite obvious that there is another problem presented by the addition of steel reinforcement to concrete. Steel in corrosive environments tends to corrode. The concrete matrix by itself is inherently a corrosive environment internally (alkali) and often reinforced concrete structures are constructed corrosive environments (i.e. exposed to seawater, de-icing salts).

As an alternative to corrosive steel, fiber reinforced polymer (FRP) rebar have been substituted as reinforcement in concrete. FRP rebar have several distinct advantages over conventional steel reinforcement including their non-corrosive nature,
high strength, and light weight. The main drawbacks of FRP include their brittle nature, low modulus of elasticity, and currently high relative price compared to steel.

As is typical with new materials, their acceptance into the main stream is generally slow because of lack of test data to substantiate their legitimate use and a lack of familiarity among the design community. For the last several decades the body of experimental data concerning the use of FRP reinforcement in structural concrete members has been building. Within the last ten years code authorities in Japan, Canada, the United Kingdom, and the United States of America have all issued guidelines for design of concrete structures using FRP as reinforcement thus paving the way for increased use by the engineering design community.

![Photographs of Concrete Distress Due to Steel Rebar Corrosion](image)

**Figure 2.1-1** Photographs of Concrete Distress Due to Steel Rebar Corrosion
2.2 **HISTORY OF FIBER REINFORCED POLYMER REBAR**

The development of FRP rebar can be traced to the expanded use of composites in the post World War II era (ACI, 2001). The lightweight, high-strength characteristics quickly made the material popular in the aerospace industry. In the 1950’s and 1960’s, the United States, the former Soviet Union, and the United Kingdom were undertaking research projects to more broadly implement the use of FRP’s.

With the expansion of the national highway system in the United States and the subsequent use of de-icing salts, corrosion of the reinforcing steel in pavements exposed to de-icing salts and marine water began to manifest itself as a problem. FRP reinforcement was not considered a viable alternative nor was it commercially available until the late 1970’s. The first solutions to the corrosion of pavement reinforcement were galvanized coatings, powder resin coatings, polymer-impregnated concrete, epoxy coatings, and GFRP rebar. For some time epoxy coated rebar was the solution of choice.

Technologically advanced products in the 1980’s, increased the demand for nonmetallic reinforcement. Due to its non-conductive and magnetically transparent characteristics, FRP reinforcement began to be used in concrete surrounding MRI equipment, seawall construction, substation reactor bases, airport runways, and electronics laboratories.

During the 1990’s, the deterioration of aging bridges in the United States and discovery of corrosion in some commonly used epoxy coated rebar again brought FRP reinforcement to the attention of the design and research communities as a possible solution to corrosion problems of reinforced pavements (ACI, 2001).
2.3 Design Philosophy Issues with FRP Reinforced Concrete

During the last decade governing code authorities in Japan, Canada, Europe, and the United States have issued design guidelines or recommendations for concrete reinforced with fiber reinforced polymer (FRP) rebar. The first generation design guidelines for FRP reinforced concrete (RC) structures were mainly provided in the form of modifications to existing steel RC codes of practice. Though the brittle linear-elastic behavior of FRP reinforcement was an influencing factor behind all of the existing design guidelines, the impact of the change of failure mode was not addressed in detail. (Pilakoutas, et. al, 2002).

2.3.1 Flexural and Ductility Design Issues

Conventionally reinforced concrete is a brittle-ductile system. The constituent materials consist of concrete (brittle) and steel rebar (ductile). The current design code requires that RC members be designed so that the steel is the “weak link” at the limit state of ultimate strength. This means that when the ultimate strength of an RC member is reached, the steel begins yielding and the member undergoes significant deformation, thereby providing obvious signs of imminent structural failure. The bulk of the current codes rely on the relatively predictable behavior of steel at the limit state of yielding, as opposed to the more unpredictable behavior of concrete at its limit state of ultimate compressive strength.

FRP reinforced concrete is a brittle-brittle system, with both the concrete and the FRP exhibiting brittle failure at ultimate strength. It may be noted that research is underway to produce an FRP rebar that mimics the stiffness and ductility of steel rebar.
(Harris, et. Al, 1998b), but currently, due to cost and availability, the most viable forms of FRP reinforcement are standard GFRP, CFRP, and AFRP bars (Glass, Carbon, and Aramid fiber bars respectively). Although FRP’s are generally capable of higher ultimate stress levels than conventional steel reinforcement, they have a linear elastic stress-strain response to failure. Thus there is no significant change in deflection prior to failure and the FRP bar will break suddenly. Since there is a lack of a ductile component (i.e. a material that deforms significantly at a certain level of stress without significant loss of strength), a ductile failure is difficult to achieve. The general trend in the new FRP codes is to favor failure by concrete crushing because concrete exhibits a degree of pseudo-ductility at its ultimate strength. The way in which flexural failure by concrete crushing is ensured is by requiring a minimum amount of flexural reinforcement (Pilakoutas et al., 2002). Due to the lack of available experimental data available for the direct determination of failure modes and behavior of FRP reinforced concrete, safety factors are currently conservative. However, as the body of experimental evidence increases, these factors will be refined.

2.3.2 Shear Capacity Design Issues

Unlike the flexural design of FRP reinforced concrete, the shear design of FRP RC does not require a major shift in design philosophy. There are, however, issues that must be recognized, that cause the shear design of FRP RC to differ from that of conventional RC.
1) The low modulus of elasticity of FRP flexural reinforcement causes the neutral axis to be higher in the beam cross-section. This leaves less uncracked concrete in the compression zone for shear resistance.

2) Due to the lower FRP axial rigidity, larger crack widths will decrease the aggregate interlock resistance mechanism of shear resistance. (ACI, 2001)

These are a couple of the major differences between the shear resistance mechanisms of FRP RC and conventional RC. They do not constitute a wholesale change in behavior, only a difference in degree of contribution.

### 2.3.3 Serviceability Design Issues

Due to the relatively low modulus of elasticity of FRP reinforcement (typically 20 – 50% of steel), there are also serviceability issues in the design of FRP reinforced concrete members that are notably different than conventional RC. Again, the low modulus of elasticity of FRP reinforcement presents itself as a major problem when designing with FRP reinforced concrete. This low modulus of elasticity can lead to excessive service load deflections and crack widths. For this reason, it is often typical for serviceability criteria to control the design of FRP RC members (ACI, 2001). In the ACI 2001 FRP design recommendation, the recommended design philosophy is that an FRP reinforced concrete member is designed based on its required strength and then checked for fatigue endurance, creep rupture endurance, and serviceability criteria. It is stated that this was chosen over the working stress design approach to ensure consistency with other ACI documents. Much of the basis for the promotion of limit states design in
conventional RC is based on the fact that this is usually the controlling factor in design. Therefore, if the designer accommodates the limit state first, they will probably not have to make significant changes to the previously calculated design values as they check serviceability criteria. It seems as though emphasis on one or the other in the design of FRP RC may, practically, be of no consequence. Both must be checked. Furthermore, if strength criteria are satisfied first, the likely consequence of the service checks will be that more reinforcement will be required for serviceability than for strength. This is of no concern for FRP RC structures (outside of additional cost), because for the limit state, there is a minimum imposed on the amount of reinforcement, not generally a maximum. So the likely chain of events will be that strength criteria will be met and then reinforcement ratios will be increased to accommodate serviceability criteria.
2.4. MATERIALS

2.4.1 FIBER REINFORCED POLYMER REBAR

Fiber reinforced polymer rebar are high strength, lightweight composite materials formed with long polymer fibers encased in resin. They are commonly used for reinforcement of structural concrete and strengthening of existing masonry. They are especially useful in situations where corrosion is a problem or where electromagnetic transparency is required. The typical types of fibers used in the production of these reinforcing bars include glass, carbon, aramid, and polyvinyl alcohol. These fibers are encased in the resin through a process called pultrusion, a continuous process that combines pulling and extrusion for manufacturing composite sections that have the same shape and cross section. The process consists of pulling a fiber material through a resin bath and then through a heated shaping die, where the resin is cured (Uomoto et al., 2002). FRP bars generally contain between 50 – 65% fibers by volume.

2.4.1.1 TENSILE BEHAVIOR

FRP’s are recognized for their high tensile strength capacity. Figure 2.4-1 shows a schematic representation of the tensile stress-strain response of grade 60 steel rebar and some typical types of FRP bars. It can be seen that the ultimate tensile capacity of FRP bars is higher than that of typical 60 grade reinforcing steel. Although the FRP bars possess higher strength, they have a lower modulus of elasticity and they also exhibit linear stress strain response up to failure and demonstrate no yield elongation. Therefore their failure mode is brittle and not ductile like typical steel reinforcing bars.
The relatively low modulus of elasticity of FRP reinforcement causes deflections of FRP RC members to be larger than for conventional steel reinforced concrete members. Therefore, serviceability criteria are, more often, the controlling element in the design of FRP RC members. Also, the brittle failure of FRP RC is of crucial concern when considering limit state design and ultimate ductility of a structure. Figure 2.4-2 displays some comparative material properties of common FRP reinforcement bars.
Table 2.4-1  Comparative Material Properties of Different Types of FRP Reinforcement

<table>
<thead>
<tr>
<th></th>
<th>Steel</th>
<th>GFRP</th>
<th>CFRP</th>
<th>AFRP</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nominal Yield Stress</td>
<td>40-75 (276-517)</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>ksi (MPa)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tensile Strength</td>
<td>70-100 (483-690)</td>
<td>70-230 (483-1600)</td>
<td>87-535 (600-3690)</td>
<td>250-368 (1720-2540)</td>
</tr>
<tr>
<td>ksi (MPa)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Elastic Modulus</td>
<td>29 (200)</td>
<td>5.1-7.4 (35-51)</td>
<td>15.9-84 (120-580)</td>
<td>6.0-18.2 (41-125)</td>
</tr>
<tr>
<td>x 10^3 ksi (GPa)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Yield Strain, %</td>
<td>1.4-2.5 N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Rupture Strain, %</td>
<td>6.0-12.0</td>
<td>1.2-3.1</td>
<td>0.5-1.7</td>
<td>1.9-4.4</td>
</tr>
<tr>
<td>Source:  (ACI, 2001)</td>
<td></td>
<td></td>
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</tr>
</tbody>
</table>

2.4.1.2  COMpressive Behavior

Because of the inherent difficulty of testing FRP bars in compression, the available test data are limited. Based on the lack of sufficient test data and the fact that concrete performs well in compression, the use of FRP bars as compression reinforcement is generally not recommended. Compressive strengths of 55%, 78%, and 20%, of the tensile strength, have been reported for GFRP, CFRP, and AFRP respectively. Compressive moduli are approximately 80% for GFRP, 85% for CFRP, and 100% for AFRP of the corresponding tensile modulus of elasticity (ACI, 2001). Since there are no standardized test procedures for determining the compressive characteristics of FRP bars, it still remains a challenge to implement the compressive resistance of these types of bars into design.
2.4.1.3 *Shear Behavior*

Most FRP bar composites are relatively weak in interlaminar shear where layers of unreinforced resin lie between the fibers (ACI, 2001). Due to the relatively low transverse rigidity and strength of the FRP bars, their ability to transfer shear by dowel action is greatly reduced (Razaqpur et al., 2004). Dowel action resistance to shear in a concrete beam is provided when longitudinal reinforcement bridges shear cracks and resists shear forces by transverse stresses incurred within the reinforcing bars.

2.4.1.4 *Bond Behavior*

For the concrete matrix to transfer stress to the FRP reinforcement, there must be sufficient bond capability of the surface of the bars. This bond stress transfer capability comes from…

1) Chemical bond.
   a. Adhesion between the interface of the FRP bars and the concrete.

2) Frictional resistance at the bar/concrete interface.

3) Mechanical interlock.
   a. This is made possible by any surface irregularity at the bar/concrete interface.

If special procedures such as braiding are performed during the manufacture of the FRP reinforcing bars, it has been shown that adequate bond can be ensured (JSCE, 1997). To increase the bond strength of FRP bars, another common practice is to adhere
a granular (sand) coating to the surface of the bars to increase the mechanical interlock mechanism of bond resistance.

Due to the wide variation in surface treatments provided by different manufacturers of FRP reinforcing bars, it is recommended that bond stress values be obtained from the manufacturer.

2.4.1.5 Time Dependent Behavior

In the early stages of FRP reinforcing bar development, premature failures were commonplace because of irregular tensile stress distributions in the FRP composite. These irregular stress distributions led to early static fatigue failures, which continued until the advent of the pultrusion process of manufacturing in the 1970’s (Kobayashi et al., 1987).

2.4.2 Fiber Reinforced Concrete

For nearly half a century, (FRC) fiber reinforced concrete has been increasingly used in structural applications. One of the main advantages of using FRC is increased toughness or ability of the concrete matrix, and therefore the entire structural member, to absorb energy. FRC also has superior resistance to fracture and impact loads than its plain concrete counterpart (Gopalaratnam et al., 1991). Hence, an FRC matrix has greater resistance to cracking and greater ductility than plain concrete. This is particularly useful for applications where impact, earthquake, or fatigue loads are of concern (Fanella and Naaman, 1985). Addition of fibers, to a plain concrete matrix, has
been shown to have little effect on its precracking behavior, but substantially improves its post cracking behavior through improved ductility and toughness (El-Shakra, 1995).

In compression, plain concrete and FRC exhibit very similar pre-peak responses. The largest behavioral differences come in the post-peak softening regime of the normalized stress-strain curve. This is due to the fact that when an FRC member, subjected to compression, begins to crack, the added fibers provide a tensile crack-closing stress transfer. These crack closing pressures are identical in nature to lateral confinement pressures on the member. El-Shakra (1995) and Gopalaratnam et al. (2005), surmised that an analysis of FRC using a lateral confinement analogy is reasonable. In fact, the equivalent lateral confinement pressure can be shown to equal the crack closing pressure when the straight segment of the split cylinder is viewed as a splitting crack along the length of the cylinder.

The crack closing stress $f_f$ (see Figure 4.1-3) can be obtained by finding the pullout force ($F$) of one fiber and the number of fibers crossing a plane. The pullout force is proportional to

$$F \propto \sqrt{f_{c}^* l \phi} \quad \{\text{Eqn. 2.4-1}\}$$

The number of fibers per unit area, $N$, crossing a plane is given by

$$N = (1.64 \times V_f) \div \left( \pi \times \phi^2 \right) \quad \{\text{Eqn. 2.4-2}\}$$

Hence, the crack closing stress hence can be obtained by

$$f_f = f_c = c \times \sqrt{f_{c}^* V_f \times (l / \phi)} \quad \{\text{Eqn. 2.4-3}\}$$

$$V_f = \text{fiber volume fraction}$$
$l = \text{fiber length}$

$\phi = \text{fiber equivalent diameter}$

$f'_c = 28 \text{ day concrete compressive strength}$

$f_r = \text{equivalent confinement stress}$

$c = \text{empirical constant}$

Studies show that lateral confinement produces a linear increase in the uniaxial compressive strength of concrete. Similarly, the addition of fibers produces an increase in compressive strength at the peak strain. The empirical formula obtained for composite compressive strength from the experiments performed by El-Shakra (1995) is given by

$$f^* = f'_c + 6f_r \quad \{\text{Eqn. 2.4-4}\}$$

and the corresponding peak strain is given by

$$\varepsilon^* = \varepsilon'_c + 8.82f_r(\varepsilon'_c + f'_c) \quad \{\text{Eqn. 2.4-5}\}$$

An equation for the effective confinement index ($f_i$) for steel fiber reinforced concrete was obtained.

$$f_i = (f^*)[1 - (1 - (\varepsilon_c/\varepsilon^*))]^a \quad \varepsilon \leq \varepsilon_c^* \quad \{\text{Eqn. 2.4-6}\}$$

$$f_i = (f^*)e^{-k(\varepsilon_c/\varepsilon^*)^{1.15}} \quad \varepsilon \geq \varepsilon_c^* \quad \{\text{Eqn. 2.4-7}\}$$

where

$$A = E_c\varepsilon_c/f_o$$

$$k = 0.17f'_ce^{-0.01f_r}$$
In 1985, Fanella and Naaman also performed a series of experiments to examine the effects of the addition of fibers to concrete mortar. They used 3 types of fibers: steel, glass, and polypropylene. Volume fractions of 1, 2, & 3% were added to three different matrices. The concrete was formed into 3”x6” cylinders, cured for 7 days at 70 degrees Fahrenheit and 100% relative humidity, and subjected to a compression test. The specimen was subjected to compression at a strain rate of 0.0110 strain/min. and the deformation of the specimen was measured with an LVDT. A summary of their results is presented subsequently.

**Compressive Strength**

The addition of glass or polypropylene fibers had very little effect on the compressive strength of the matrix. Steel fibers led to an improvement in compressive strength in the range of 0 to 15 percent.

**Toughness**

Toughness can be defined by the area under the stress-strain curve. The toughness index is defined as the ratio of the toughness of the FRC matrix to the plain concrete matrix. At a 2% volume fraction of fiber, the toughness index ranged from approximately 1.2 to 2.7 depending on the concrete mix and the type of fiber used.

**Post-Peak Stress-Strain Behavior**

An increase in the volume fraction of the fibers generally led to an increase in the slope of the descending branch of the stress-strain curve. Increasing the aspect ratio (ratio of the length of fiber to fiber diameter) of the steel fibers also led to the same phenomena.

**Analytical Stress-Stress Strain Relationship**
Fanella and Naaman modified an analytical expression, developed by Sargin (1971) and later modified by Wang et al. (1978), to describe the stress-strain relationship of the fiber reinforced concrete. They modified the constants in the equation to follow the characteristics of the FRC test specimens. The equation is as follows…

\[ Y = \frac{AX + BX^2}{1 + CX + DX^2} \]  \hspace{1cm} \text{Eqn. 2.4-8}

\( X \) = normalized strain
\( Y \) = normalized stress

\( A, B, C, D \) = constants to be determined from the boundary conditions of the curve

**Conclusions**

Addition of steel fibers to plain concrete increases the peak stress and the ductility of the concrete matrix. The addition of glass or polypropylene fibers only increases ductility significantly. This fact seems to suggest that the strength of the fiber has a direct correlation to the ability of the fibers to contribute to the compressive strength of the matrix. During the loading tests, the steel fibers failed by bond failure (i.e. pulling out of the matrix). The glass and polypropylene fibers failed in tension. The steel fiber failure by pullout seems to indicate that the bond development of the steel fibers was not sufficient to capture the full usefulness of the fibers. This could possibly be improved by using longer fibers or by using fibers that have better bond characteristics. This may lead to even further improvements in compressive strength and toughness. Fanella and Naaman had difficulty incorporating much more than a 2% fraction of polypropylene fibers in their mix without adversely reducing the density of the mix. This seemed to limit the usefulness of polypropylene fiber content at a maximum, beyond which, no more advantage was gained. If a method of incorporating more fibers without adversely
affecting the mix could be determined, the limit of usefulness of fiber might exhibit even higher compressive strength and toughness at fiber volume fractions greater than 2%.
2.5 **Ductility of FRP Reinforced Concrete Members**

For conventional (steel) reinforced concrete it has long been known that under-reinforcing the reinforced concrete (RC) flexural section allows the section to deform significantly. Under-reinforced members can deform significantly without much loss of load carrying capacity, when the reinforcing steel reaches its yield stress. This allows the member to avoid a brittle concrete crushing failure and to give sufficient warning that a structural failure is imminent. This ductile mechanism is possible because the reinforcing steel has a relatively large yield plateau in its stress-strain response. This phenomenon can be seen in Figure 2.5-1 at the portion of the curve labeled “yield, plastic”. FRP, on the other hand, has a linear-elastic stress-strain response until failure (Figure 2.5-2).

![Stress-Strain Curve for Typical Steel Bar](image)

**Figure 2.5-1** Stress-Strain Curve for Typical Steel Bar
Therefore, unlike conventional RC, there is no readily discernable change in flexural behavior that would indicate an imminent failure. For this reason, the Canadian Highway Bridge Design Code (Canadian Standards Association, 2000) requires that FRP reinforced flexural sections be checked during design to ensure that large curvature occurs before failure. This is referred to in the research literature as a check of deformability. Even though the FRP RC flexural members do not have as clearly a defined transition into the failure stage, they still typically undergo large deformations
prior to failure. Irregardless of whether there is a clear transition from the “stable” stage to the “failure” stage, if an FRP RC member is exhibiting relatively large tensile cracks and clearly visible and excessive deformation, warning will be taken that there are structural problems. Because of this, it seems reasonable that a check of deformability is a reasonable requirement to ensure sufficient ductility of an FRP RC member at ultimate load.

Due to the lack of ductility, of FRP RC flexural members, the Guide for the Design and Construction of Concrete Reinforced with FRP Bars (American Concrete Institute Committee 440, 2001) proposes that a larger safety factor should be used in design and that an even higher factor be used if the mode of failure is by brittle FRP tensile rupture. The ACI 440 document requires, that if the least conservative safety factor is used, failure must be by concrete crushing because concrete in compressive failure will exhibit a sort of pseudo-ductility. This concrete crushing failure is ensured by requiring that the area of FRP reinforcement be greater or equal to 1.4 times the balanced reinforcement ratio. This guideline would seem to be adequate insurance that sufficient reinforcement would be provided to avoid any chance of brittle failure of FRP in tension. But, to demonstrate the possible drawbacks of this philosophy, consider the case of a rectangular beam with a design concrete compressive strength $f'_c=5,000$ psi that is designed with the area of FRP reinforcement $(A_f)$ to provide 1.4 times the balanced reinforcement ratio $(\rho_{bal})$. It is not uncommon for concrete delivered to the construction site to have a significantly higher compressive strength than that used in design. This may be due to additional cementitious material added to the mix by the ready-mix plant, to assure design strength is reached, or due to rich mixes provided to reach the required
strength early in order to expedite construction. When the excessive initial strength is coupled with the continuing compressive strength increase of concrete after the initial 28 day period, the actual compressive strength of concrete in the field can be significantly higher than the specified minimum $f'_c$. In the example below, the area of reinforcement required for 1.4 times the area required for balanced failure is calculated. This is using a specified minimum $f'_c = 5,000$ psi. It is assumed that due to over-strength mix and continual concrete hardening, at some time in the future, the concrete compressive strength is $f_c = 8,000$ psi.

Given:

$f'_c = 5$ ksi

$b = 1$

$F_u = 100$ ksi [GFRP Reinforcement]

*Equilibrium of Forces*

$C = T$

$0.85f'_c b = A_f F_u$

*Strain compatibility*
\[
\frac{0.003}{c} = \frac{0.015}{d-c}
\]

Solving for “c”

\[c = \frac{d}{6}\]

\[a = \beta_1 c\]

\[\beta_1 = 0.80 \quad [\text{for } f'_c = 5 \text{ ksi}]\]

from equilibrium of forces

\[(0.85)(5)(0.80)(d/6) = A_{Fb} (100)\]

\[0.00567 = A_{Fb}/d = \rho_{bal}\]

To meet the criteria of 1.4 times \(\rho_{bal}\) the area of FRP reinforcement is

\[A_F = (1.4)(0.00567)(d) = 0.0079(d)\]

This is the area of reinforcement required by design. Then following with the previously stated assumption, the concrete arrives at the construction site with a stronger mix than specified and continues to harden until at some time in the future \(f_c = 8 \text{ ksi}\).

Equilibrium of Forces

\[\beta_1 = 0.65 \quad [\text{for } f'_c = 8 \text{ ksi}]\]

\[(0.85)(8)(0.80)(d/6) = A_{Fb} (100)\]

\[0.0074 = A_{Fb}/d = \rho_{bal}\]

At the \(f_c = 8 \text{ ksi}\) state, the area of reinforcement required for balanced failure is \(A_F = 0.0074(d)\) and the area provided by design is \(A_F = 0.0079(d)\). It can be seen that the
originally required 1.4AFB became only 1.07AFB. At this point the FRP reinforced flexural member is theoretically becoming very close to a 50% chance of failure by tensile FRP rupture at ultimate strength. As opposed to conventional RC, where any over-strength and continual hardening of the concrete is advantageous to assure a ductile failure, these same circumstances reduce the likelihood of a ductile failure in FRP reinforced beams. While the above example may not be the norm, it illustrates that the requirement of 1.4 times the balanced reinforcement ratio is only partially successful in producing the desired mode of failure. A possible saving grace to this potential lessening of the over-reinforcement ratio, is that strength requirements for FRP reinforced RC are generally less than serviceability reinforcement requirements. It will often be the case that serviceability reinforcement requirements will be even higher than the 1.4 times the balanced reinforcement ratio.

It becomes obvious that the classical definitions of ductility are not sufficient to describe deformations of FRP RC member at the limit state. Ductility in a steel RC beam has been defined as the ratio of ultimate deformation to the deformation at yield. Since there is no yield point for FRP reinforcement, this classic definition simply will not work.

Several methods have been proposed for the evaluation of ductility of FRP RC beams. They fall into two main categories: Deformation based methods and energy based methods.


2.5.1 Deformation Based Methods

Jaeger et al. (1997) and the Canadian Highway Bridge Design Code (Canadian Standards Association, 2000) assessed the deformability of sections reinforced with FRP by a “performance factor”. They defined a deformability factor (DF) as follows.

\[ DF = \text{Moment Factor} \times \text{Curvature Factor} \]

Moment Factor = (Moment at Ultimate)/(Moment at Concrete Strain of 0.001)

Curvature Factor = (Moment at Ultimate)/(Curvature at Concrete Strain of 0.001)

Or more simply

\[ DF = \frac{M_u \psi_u}{M_s \psi_s} \quad \text{(Eqn. 2.5-1)} \]

A direct application of this method showed that beams with a small percentage of FRP tensile reinforcement, which fail by tensile rupture, have larger deformability factors than do those beams failing in compression (Vijay et al., 1996). However, the compressive failure was more ductile and gradual than tensile rupture (Grace et al., 1998).

Newhook et al. (2002) proposed the use of the same ratio as presented above for the calculation of a deformability factor, but instead of finding the service load curvature (\(\psi_s\)) and moment (\(M_s\)) at a concrete strain of \(\varepsilon_c = 0.001\), they determined service values of curvature and moment at a rebar strain of \(\varepsilon_s = 2,000 \times 10^{-6}\). This was motivated by their suggestion that it was serviceability criteria that should motivate design of FRP flexural sections, most notable tensile crack width. This limiting rebar strain is, in effect, a limit on tensile crack width. They suggested a permissible value of \(DF \geq 4\) for all sections reinforced with FRP and presented a parametric study to show that \(DF \geq 4\) will always be satisfied as long as \(\rho_r < \rho_{fmax}\). Where \(\rho_{fmax}\) is limited by the empirical equation…
\[ \rho_{f,\text{max}} = 0.2 \frac{\beta_t f'_c}{f_{fs}} \]  \{Eqn. 2.5-2\}

Again, this method gives very high deformability factors for FRP RC members with very low reinforcement ratios even though the actual failure at ultimate strength, for these types of beams, is brittle. The focus of a deformability factor performance based standard is not necessarily that of providing a true ductile failure in the classical sense of the term, but instead assuring that a minimum amount of deformation occurs prior to ultimate failure. The actual failure mode may still be brittle, but sufficient deflection has already taken place and provided warning of significant structural problems and possible imminent failure.

### 2.5.2 Energy Based Methods

Using an energy based method may be defined as the ratio relating any two of the inelastic, elastic, and total energies. The problem is determining how much of the total energy is elastic and how much is inelastic (Grace et al., 1998). Grace et al. proposed that the “energy ratio” defined as the ratio of inelastic to total energy is the proper measure of ductility.

### 2.5.3 Conclusion

For so long, the definition of ductile failure has been relatively clearly understood with steel RC beams. With the onset of more frequent use of FRP reinforcement, the design community has been forced to reconsider their classical ideas of ductility. In the future what will be considered an acceptably ductile failure? Will it be the over-reinforced FRP RC beam that deflects 1” and fails in a relatively gradual manner, or will it be the under reinforced beam that deflects 2” and then fails suddenly.
It seems as though performance based standards are likely to replace the classical definitions of ductility when FRP reinforcement is used in concrete structures. As the body of experimental data increases, the design community and code authorities will be able to establish more reasonable guidelines for acceptable failure modes and ductility in FRP reinforced concrete structures.
2.6 Flexural Behavior of FRP Reinforced Concrete Members

Research has established that stress, strain, and ultimate flexural strength can be determined using the same equations whether the flexural reinforcement is steel or FRP bars (GangaRao and Vijay, 1997). More emphasis has been given to the determination of which mode of failure is preferred as opposed to the method of determining the behavior of concrete flexural sections reinforced with FRP reinforcement. Newhook et al. (2002) gave an overview of flexural behavior of FRP reinforced concrete sections that covered three modes of failure: concrete crushing, balanced, and tension failure of the FRP reinforcement.

2.6.1 Concrete Crushing Failure

Analysis of an over-reinforced concrete section, with FRP reinforcement, can be performed using the same equivalent stress block method and ultimate concrete strain of 0.003 that are used in conventionally reinforced concrete sections (Figure 2.6-1). Since the section is over-reinforced, concrete crushing failure is guaranteed.

Figure 2.6-1  Flexural Behavior of FRP Reinforced Concrete Sections (Concrete Crushing Failure Mode)
2.6.2 **Balanced Failure**

At balanced failure, the concrete reaches its ultimate strain value at the same time the FRP reinforcement reaches its rupture strain. The same equivalent stress block method may be used to determine the stress-strain behavior of the concrete in compression. The stress and strain in the FRP reinforcement are its ultimate stress and strain values.

2.6.3 **Tension Failure of the FRP Reinforcement**

When failure of an FRP reinforced concrete section is by tensile failure of the FRP reinforcement, a rectangular stress block cannot be used because the failure is not accompanied by yielding of the reinforcement followed by a large increase in curvature and crushing of the concrete (Newhook et al., 2002). Figure 2.6-2 represents the type of non-linear concrete stress-strain behavior that can be expected during FRP tensile failure in a concrete bending section.

![Figure 2.6-2](image)

**Figure 2.6-2** Flexural Behavior of FRP Reinforced Concrete Sections (FRP Tension Failure Mode)
An equation was proposed by MacGregor (1997) for the non-linear stress-strain compressive behavior of concrete in this situation.

\[ f_c = 1.8 f'_c \frac{\varepsilon / \varepsilon_0}{1 + (\varepsilon / \varepsilon_0)^2} \]  \hspace{1cm} \text{Eqn. 2.6-1}

where...

\[ \varepsilon_0 = 1.71 f'_c / E_c \]  \hspace{1cm} \text{Eqn. 2.6-2}

### 2.6.4 SERVICE BEHAVIOR

Service behavior of FRP reinforced concrete sections can be analyzed using classical mechanics and linear stress-strain distributions as shown in Figure 2.6-3.

**Figure 2.6-3** Flexural Behavior of FRP Reinforced Concrete Sections (Service Conditions)
2.7 Shear Behavior of FRP Reinforced Concrete Members

In reinforced concrete, after the formation of diagonal tension cracks, a member resists the applied shear stresses by means of a number of mechanisms:

1) The shear resistance of the uncracked compression zone
2) Aggregate interlock
3) Dowel action of the longitudinal reinforcement
4) Arching Action
5) Residual stresses across tensile cracks
6) Shear carried by shear reinforcement

(ACI-ASCE Committee 445, 1998)

In conventional reinforced concrete shear analysis, the first five mechanisms are lumped together as the concrete contribution to shear resistance ($V_c$). The last is known as the shear reinforcement (diagonal tension) resistance mechanism ($V_s$). The code models, for the calculation of shear capacity, recognize the total shear resistance as $V = V_c + V_s$. However, it is understood that for beams with shear span to effective depth ratio ($a/d$) less than 2.5, the shear strength can only be determined accurately using a strut and tie model (Razaqpur et al., 2004). For beams with $a/d$ greater than 2.5, also known as slender beams, Zsutty (1968) and Khuntia and Stojadinovic (2001) have confirmed the $V_c$ is a function of $a/d$, the longitudinal reinforcement axial rigidity ($E_s \rho_f$), and the concrete strength.

When using FRP for longitudinal flexural reinforcement, several of the above factors can be expected to have a smaller effect than when steel reinforcement is used.
(ACI, 2001). The Guide for the Design and Construction of Concrete Reinforced with FRP Bars recognized that due to the lower axial stiffness of FRP reinforcement, the depth of the neutral axis will be smaller than that for steel reinforced concrete. This smaller uncracked compression zone will contribute less to the shear resistance of the FRC RC member than the comparably larger uncracked compression zone of a similar steel RC member. Likewise the lower axial rigidity of the FRP reinforcement will contribute to larger crack widths in flexural sections, thereby minimizing the effects of aggregate interlock and residual stresses across tensile cracks (Razaqpur et al., 2004).

Current ACI recommendations (ACI, 2001) do not include the effect of the shear span to effective depth ratio (a/d). The Japan Society of Civil Engineers (JSCE, 1997) method neglects the shear span (a) and the Canadian Standard S806-02 (CSA, 2002) includes both the a/d effect and the effect of the reinforcement rigidity (\(E_f\rho_f\)). Razaqpur et al. (2004) state that these differences are partly due to contradictory findings from limited research data.

Gross et al. (2001) performed a study to determine the shear strength of concrete beams reinforced with GFRP bars. They tested thirty beams in which they varied the reinforcement ratio from 0.0011 to 0.0023. This equated to 2.10 to 4.32 times the balanced strain reinforcement ratio at ultimate (\(\rho_{fb}\)). They also tested one set of beams with normal strength concrete (\(f'_c \approx 5,000 \text{ psi}\)) and one set with high strength concrete (\(f'_c \approx 10,000 \text{ psi}\)). They found that…

1) The longitudinal reinforcement ratio has a small effect on concrete shear resistance. The increase was slight, not directly proportional.
2) The high strength concrete beams exhibited slightly lower relative shear strength than the normal strength beams. This is likely due to the neutral axis location being slightly higher in high strength beams.

3) The shear equation proposed by ACI (2001) significantly underestimates the shear strength of normal and high strength beams with longitudinal GFRP reinforcement. It also significantly overestimates the effect of the reinforcement ratio ($\rho_f$) on shear strength.

Razaqpur et al. (2004) performed a study where they tested seven (7) beams varying the shear span (a/d) and the reinforcement rigidity ($E_f\rho_f$). All of the beams were over-reinforced in flexure to assure shear failure.

Upon completion of the tests, the data was also compared and analyzed with previous experiments by Yost et al. (2001), Michaluk et al. (1998), Zhao and Maruyama (1995), Alkhradji et al. (2001), and Tureyen and Frosch (2002). The results of their study and the previous studies are summarized in Table 2.6-1. The data in Table 2.6-1 show the experimental failure load and then a ratio of the experimental failure load versus that predicted by the CSA, ACI, and JSCE models.

Razaqpur et al. (2004) also compared the data with major design recommendations.

**ACI Recommendations**

According to ACI Committee 440 (2001) the concrete contribution to shear stress can be evaluated using
\[ V_c = \frac{\rho_f E_f}{90 \beta_i f'_c} \left( \frac{\sqrt{f'_c b_a d}}{6} \right) \]  \{Eqn. 2.7-1\}

It was noted according to this equation \( V_c \) is not a function of \( a/d \), is directly proportional to \( E_f \) and is inversely proportional to the square root of \( f'_c \). This equation also assumes that plain concrete has no shear strength. The experimental data however shows that \( V_c \) is a function of \( a/d \) and is not directly proportional to \( E_f \). The inverse relation to the square root of \( f'_c \) indicates that the shear strength decreases as the concrete strength increases. They state that this is counterintuitive and not supported by empirical evidence, but as stated previously by Gross et al. (2001), high strength concrete beams reinforced with GFRP bars exhibited a relatively lower shear capacity due to the fact the neutral axis is higher in these beams.
Table 2.6-1  Comparison of Experimental Shear Failure Loads with Code/Guideline Predictions

<table>
<thead>
<tr>
<th>Beam</th>
<th>$P_{\text{exp}}$ (kN)</th>
<th>$P_{\text{exp}}/P_{u,\text{CSA}}$</th>
<th>$P_{\text{exp}}/P_{u,\text{ACI}}$</th>
<th>$P_{\text{exp}}/P_{u,\text{JSCE}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>36.11</td>
<td>1.30</td>
<td>6.56</td>
<td>1.42</td>
</tr>
<tr>
<td>2</td>
<td>46.95</td>
<td>1.25</td>
<td>4.58</td>
<td>1.40</td>
</tr>
<tr>
<td>3</td>
<td>47.23</td>
<td>1.25</td>
<td>3.43</td>
<td>1.22</td>
</tr>
<tr>
<td>4</td>
<td>42.71</td>
<td>1.01</td>
<td>2.22</td>
<td>1.33</td>
</tr>
<tr>
<td>5a</td>
<td>96.18</td>
<td>2.41</td>
<td>2.02</td>
<td>3.00</td>
</tr>
<tr>
<td>6</td>
<td>49.66</td>
<td>1.47</td>
<td>4.27</td>
<td>1.47</td>
</tr>
<tr>
<td>7</td>
<td>38.45</td>
<td>1.28</td>
<td>3.49</td>
<td>1.20</td>
</tr>
<tr>
<td>8</td>
<td>38.10</td>
<td>1.24</td>
<td>3.94</td>
<td>1.18</td>
</tr>
<tr>
<td>9</td>
<td>31.50</td>
<td>1.23</td>
<td>3.29</td>
<td>1.17</td>
</tr>
<tr>
<td>10</td>
<td>44.40</td>
<td>1.28</td>
<td>3.10</td>
<td>1.22</td>
</tr>
<tr>
<td>11</td>
<td>45.30</td>
<td>1.04</td>
<td>2.38</td>
<td>0.99</td>
</tr>
<tr>
<td>12</td>
<td>45.10</td>
<td>1.09</td>
<td>2.30</td>
<td>1.04</td>
</tr>
<tr>
<td>13</td>
<td>42.20</td>
<td>1.10</td>
<td>2.20</td>
<td>1.05</td>
</tr>
<tr>
<td>14</td>
<td>37.30</td>
<td>0.79</td>
<td>2.90</td>
<td>0.57</td>
</tr>
<tr>
<td>15</td>
<td>79.05</td>
<td>1.09</td>
<td>5.24</td>
<td>0.88</td>
</tr>
<tr>
<td>16</td>
<td>45.00</td>
<td>1.30</td>
<td>2.11</td>
<td>1.41</td>
</tr>
<tr>
<td>17</td>
<td>46.00</td>
<td>1.06</td>
<td>1.26</td>
<td>1.14</td>
</tr>
<tr>
<td>18</td>
<td>40.50</td>
<td>1.03</td>
<td>1.27</td>
<td>1.11</td>
</tr>
<tr>
<td>19</td>
<td>106.80</td>
<td>1.52</td>
<td>2.82</td>
<td>1.75</td>
</tr>
<tr>
<td>20</td>
<td>72.20</td>
<td>1.42</td>
<td>5.53</td>
<td>1.67</td>
</tr>
<tr>
<td>21</td>
<td>80.10</td>
<td>1.31</td>
<td>3.53</td>
<td>1.54</td>
</tr>
<tr>
<td>22</td>
<td>108.10</td>
<td>1.19</td>
<td>4.73</td>
<td>1.35</td>
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<tr>
<td>23</td>
<td>94.80</td>
<td>1.07</td>
<td>4.48</td>
<td>1.21</td>
</tr>
<tr>
<td>24</td>
<td>114.80</td>
<td>1.20</td>
<td>4.33</td>
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<td>25</td>
<td>137.00</td>
<td>1.19</td>
<td>3.00</td>
<td>1.36</td>
</tr>
<tr>
<td>26</td>
<td>152.60</td>
<td>1.36</td>
<td>3.61</td>
<td>1.55</td>
</tr>
<tr>
<td>27</td>
<td>177.00</td>
<td>1.47</td>
<td>3.34</td>
<td>1.67</td>
</tr>
<tr>
<td>Mean</td>
<td></td>
<td>1.21</td>
<td>3.06</td>
<td>1.27</td>
</tr>
<tr>
<td>Standard deviation</td>
<td>0.17</td>
<td>1.42</td>
<td>0.27</td>
<td></td>
</tr>
</tbody>
</table>

aSince the $a/d$ ratio for this beam is less than 2.5, it is not included in the mean and standard deviation.
CSA Recommendations

According to the Canadian Standards Association (2002) the concrete contribution to shear resistance can be calculated as

\[ V_c = 0.035\lambda \phi_c \left( f'_c \rho f E_f \frac{V_f}{M_f} d \right)^{1/3} b_w d \] \hspace{1cm} \{Eqn. 2.7-2\}

where

\[ 0.1\lambda \phi_c \sqrt{f'_c b_w d} \leq V_c \leq 0.2\lambda \phi_c \sqrt{f'_c b_w d} \] \hspace{1cm} \{Eqn. 2.7-3\}

This equation takes into account the effects a/d, E_dρ_f, and f'_c.

JSCE Recommendations

JSCE (1997) recommends the following for concrete contribution to shear resistance.

\[ V_c = \beta_d \beta_p \beta_n f_{vd} b_w d / \gamma_b \] \hspace{1cm} \{Eqn. 2.7-4\}

\[ \beta_p = \left( \frac{100 \rho_f E_f / E_s}{1.5} \right)^{1/3} \leq 1.5 \] \hspace{1cm} \{Eqn. 2.7-5\}

\[ \beta_d = \left( \frac{1000}{d} \right)^{1/4} \leq 1.5 \] \hspace{1cm} \{Eqn. 2.7-6\}

\[ f_{vd} = 0.2 f_{mcd}^{1/3} \leq 0.72 (MPa) \] \hspace{1cm} \{Eqn. 2.7-7\}

The JSCE equation is similar to the Canadian equation. It does not however take into account the effect of the shear span.

Razaqpur et al. (2004) concluded that:
1) The concrete contribution to the shear strength of beams ($V_c$) is a function of concrete strength, the rigidity of the longitudinal reinforcement ($E_d\rho_f$), and the shear span ratio ($a/d$).

2) For beams with shear span/depth ratio greater than 2.5, $V_c$ varies almost linearly with the cubic root of the product of the above three parameters.

3) The Canadian and Japanese methods are in close agreement with the experimental data for beams with $a/d > 2.5$.

4) The ACI recommendation is highly conservative and its predicted values differ significantly from experimental results.

5) Contrary to physical evidence both the ACI and JSCE methods assume that plain concrete has no shear strength.

6) All three methods yield conservative results for beams with $a/d < 2.5$. More research is needed to arrive at appropriate methods for the estimation of shear capacity of beams with $a/d < 2.5$. Possibly by the use of strut and tie models.

It is worth noting here that the beams examined by Razaqpur et al. (2004) varied in concrete strength from $f'_c = 3,495$ psi (24.1 MPa) to $f'_c = 5,874$ psi (40.5 MPa) except for the two beams from the Michaluk et al. (1998) study which had an $f'_c = 9,572$ psi (66.0 MPa). In Table 2.6-1 these beams are represented as beams number 14 and 15. It is for these beams that the CSA and JSCE methods overestimated the concrete strength (beam 14 for CSA and both beams for JSCE). This overestimation may give reason to more carefully examine the relationship of increasing the shear resistance of FRP reinforced beams linearly with the cubic root of concrete strength for high strength
concrete. This tends to correspond with the findings previously mentioned by Gross et al. (2001).

The research seems to indicate the Canadian and Japanese methods most closely model the shear behavior of normal strength concrete beams with FRP reinforcement. The ACI recommendation is overly conservative. The sources of its conservatism could very well come from the assumed linear relationship it uses concerning the axial rigidity of the reinforcement and the neglect of the shear span (a/d) relationship. Both the ACI and JSCE recommendations are quite conservative in assuming plain concrete has no shear resistance. The CSA and JSCE guidelines may look into further research on the effectiveness of their equations for high strength concrete reinforced with FRP. The recommendations may be non-conservative for these types of beams.
2.8 Fatigue Behavior of FRP Reinforced Concrete Members

Over the last 30 years, a large portion of the experimental data, concerning the fatigue of FRP materials, has been obtained from stand alone tests performed on FRP materials suitable for aerospace applications (ACI 2001). CFRP is considered to be the least prone to fatigue failure. CFRP composites have been reported to have a 5 – 8% degradation of initial static strength in a plot of stress vs. logarithmic of the number of cycles to failure. AFRP composites have been reported to have a degradation of 5 – 6% and GRRP composites have been reported to show approximately 10% degradation with respect to initial static strength per decade of logarithmic life.

Kumar and GangaRao (1998) performed fatigue tests on concrete bridge deck specimens reinforced with GFRP bars. The experimental program consisted of fatigue testing of four deck specimens with varying stringer stiffness, composite and non-composite casting, and transverse post-tensioning. The specimens were subjected to a cyclic loading at the center of one hertz. They had the following conclusions.

1) The deck degradation rate in GFRP reinforced decks compared well with that of similar steel reinforced decks.
   a. In steel reinforced decks, there was a gradual degradation until 80% of their fatigue life, thereafter, a non-linear variation was observed until failure.
   b. In GFRP reinforced decks, there was a linear degradation even after 2,000,000 cycles, so 2,000,000 cycles can conservatively be taken as 80% of their fatigue life.
2) Transverse post-tensioning limited the degradation by a factor of five for two similar slabs. However, decreasing the stringer span also gave similar limitation in degradation.

3) Fatigue failure in concrete decks is influenced by crack formation at the bottom of the deck, so extreme fiber tensile stresses in the deck should be limited.
CHAPTER 3 - EXPERIMENTAL PROGRAM

3.1 SCOPE AND MOTIVATIONS

The scope of the test program was to examine the flexure shear response of concrete members, both with and without additional polypropylene fibers as part of the concrete matrix, reinforced with different types of fiber reinforced polymer (FRP) bars. Not only was the static response of the beams examined, but the response of the beams due to fatigue loading was also examined. As a precursor to the primary testing program, two preliminary experimental programs were used to establish the behavior or the constituent materials. Concrete compression tests were conducted to determine the compressive behavior of the concrete with and without polypropylene fibers. In addition, tension tests were conducted on the FRP bars to determine their mechanical properties.

As mentioned previously in this report, a strong motivation for this research was the search for non-corrosive reinforcement that can be viably used in concrete structures. The fatigue testing segment derives its motivation from the desire to simulate bridge deck loading. While a significant deal of research has been done on the fatigue properties of FRP bars in stand-alone tests, relatively little information is available on the fatigue behavior of FRP as concrete flexural reinforcement. Another main objective of the testing program was to identify possible improvements in the ductility of FRP reinforced concrete flexural members, most notably through the addition of fibers.
3.2 CONCRETE SPECIMEN MIX DETAILS

3.2.1 LAB MIXED CONCRETE SPECIMENS

For the characterization and comparison of the compressive behavior of plain and polypropylene fiber reinforced concrete, several batches of concrete compression specimens were fabricated in the lab. The specimens were combined in a stationary, electrically powered mixer in 0.5 cubic yard batches. The mix design proportions are shown in Tables 3.2-1, 2, & 3.

<table>
<thead>
<tr>
<th>Constituent Material</th>
<th>Constituent / yd$^3$ of concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement (Type I)</td>
<td>594 lbs.</td>
</tr>
<tr>
<td>Fly ash (Class C)</td>
<td>106 lbs.</td>
</tr>
<tr>
<td>Water</td>
<td>310 lbs.</td>
</tr>
<tr>
<td>Water to Cementitious materials ratio ($w/(c+f)$)</td>
<td>0.44</td>
</tr>
<tr>
<td>Limestone Aggregates (3/4” max. – Bethany Falls)</td>
<td>1714 lbs.</td>
</tr>
<tr>
<td>Sand (Missouri river)</td>
<td>1186 lbs.</td>
</tr>
<tr>
<td>Air Entraining Agent (Daravair 1000)</td>
<td>0.8 oz/100 lb. cement</td>
</tr>
<tr>
<td>Super-plasticizer (AdvaCast 530)</td>
<td>16 oz.</td>
</tr>
<tr>
<td>Polypropylene Fibers (Fibermesh by Synthetic Industries)</td>
<td>0.0%</td>
</tr>
<tr>
<td>Unit Weight</td>
<td>144.8 pcf</td>
</tr>
<tr>
<td>Air Content</td>
<td>4.2%</td>
</tr>
<tr>
<td>Slump</td>
<td>2”</td>
</tr>
</tbody>
</table>
The fibers were added to the aggregates and pre-mixed prior to the inclusion of the water and cement to ensure sufficient dispersal of the fibers. While placing the concrete, no significant “balling” of the fibers was noticed, and the fibers seemed to be evenly distributed. After mixing the concrete, both 6” diameter standard and 4” diameter cylinders were prepared and cured in a 100% relative humidity curing chamber until the time of testing.

<table>
<thead>
<tr>
<th>Constituent Material</th>
<th>Constituent / yd$^3$ of concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement (Type I)</td>
<td>594 lbs.</td>
</tr>
<tr>
<td>Fly ash (Class C)</td>
<td>106 lbs.</td>
</tr>
<tr>
<td>Water</td>
<td>310 lbs.</td>
</tr>
<tr>
<td>Water to Cementitious materials ratio ($w/(c+f)$)</td>
<td>0.44</td>
</tr>
<tr>
<td>Limestone Aggregates (3/4” max. – Bethany Falls)</td>
<td>1714 lbs.</td>
</tr>
<tr>
<td>Sand</td>
<td>1186 lbs.</td>
</tr>
<tr>
<td>Air Entraining Agent (Daravair 1000)</td>
<td>0.94 oz/100 lb. cement</td>
</tr>
<tr>
<td>Super-plasticizer (AdvaCast 530)</td>
<td>33 oz.</td>
</tr>
<tr>
<td>Polypropylene Fibers (Fibermesh by Synthetic Industries)</td>
<td>0.5%</td>
</tr>
<tr>
<td>Unit Weight</td>
<td>137.0 pcf</td>
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<tr>
<td>Air Content</td>
<td>8.0%</td>
</tr>
<tr>
<td>Slump</td>
<td>3.75”</td>
</tr>
</tbody>
</table>

The fibers were added to the aggregates and pre-mixed prior to the inclusion of the water and cement to ensure sufficient dispersal of the fibers. While placing the concrete, no significant “balling” of the fibers was noticed, and the fibers seemed to be evenly distributed. After mixing the concrete, both 6” diameter standard and 4” diameter cylinders were prepared and cured in a 100% relative humidity curing chamber until the time of testing.
Table 3.2-3  1.0% Fiber Fraction Mix Proportions and Properties (prepared in lab)

<table>
<thead>
<tr>
<th>Constituent Material</th>
<th>Constituent / yd$^3$ of concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement (Type I)</td>
<td>584 lbs.</td>
</tr>
<tr>
<td>Fly ash (Class C)</td>
<td>103 lbs.</td>
</tr>
<tr>
<td>Water</td>
<td>304 lbs.</td>
</tr>
<tr>
<td>Water to Cementitious materials ratio $w/(c+f)$</td>
<td>0.44</td>
</tr>
<tr>
<td>Limestone Aggregates</td>
<td>1687 lbs.</td>
</tr>
<tr>
<td>(3/4” max. – Bethany Falls)</td>
<td></td>
</tr>
<tr>
<td>Sand</td>
<td>1173 lbs.</td>
</tr>
<tr>
<td>(Missouri river)</td>
<td></td>
</tr>
<tr>
<td>Air Entraining Agent</td>
<td>1.15 oz/100 lb. cement</td>
</tr>
<tr>
<td>(Daravair 1000)</td>
<td></td>
</tr>
<tr>
<td>Super-plasticizer</td>
<td>66 oz.</td>
</tr>
<tr>
<td>(AdvaCast 530)</td>
<td></td>
</tr>
<tr>
<td>Polypropylene Fibers</td>
<td>1.0%</td>
</tr>
<tr>
<td>(Fibermesh by Synthetic Industries)</td>
<td></td>
</tr>
<tr>
<td>Unit Weight</td>
<td>137.2 pcf</td>
</tr>
<tr>
<td>Air Content</td>
<td>5.5%</td>
</tr>
<tr>
<td>Slump</td>
<td>1”</td>
</tr>
</tbody>
</table>

3.2.2 READY-MIX CONCRETE SPECIMENS

Ready-mix concrete was used for the fabrication of the beam test specimens and also for the fabrication of the full-scale deck slab tests that were part of the MoDOT sponsored Steel Free Bridge Deck Project. In addition to the compression specimens prepared in the lab, compression cylinders (both 6” and 4” diameter) were taken from the ready-mix concrete provided for the fabrication of the beam test specimens and the deck slab specimens. There were 6 different castings of the beam specimens: four for the
flexural bond testing investigation of the Steel Free Bridge Deck Project and two for the flexure shear testing investigation (covered by this thesis). There were also three separate castings of the full-scale bridge slab specimens. The full-scale deck slab specimens consisted of one plain concrete slab and two 0.5% fiber fraction slabs. The typical mix design proportions and properties for both the plain and 0.5% Ready Mix concrete batches are given in Tables 3.2-4 & 5.

**Table 3.2-4** Plain Concrete Mix Proportions and Properties (Ready Mix)

<table>
<thead>
<tr>
<th>Constituent Material</th>
<th>Constituent / yd$^3$ of concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement (Type I)</td>
<td>622 lbs.</td>
</tr>
<tr>
<td>Fly ash (Class C)</td>
<td>148 lbs.</td>
</tr>
<tr>
<td>Water</td>
<td>243 lbs.</td>
</tr>
<tr>
<td>Water to Cementitious materials ratio (w/(c+f))</td>
<td>.32</td>
</tr>
<tr>
<td>Limestone Aggregates (3/4” max. – Bethany Falls)</td>
<td>1735 lbs.</td>
</tr>
<tr>
<td>Sand (Missouri river)</td>
<td>1329 lbs.</td>
</tr>
<tr>
<td>Air Entraining Agent (Daravair 1000)</td>
<td>3.4 oz/100 lb. cement</td>
</tr>
<tr>
<td>Super-plasticizer (AdvaCast 530)</td>
<td>50 oz.</td>
</tr>
<tr>
<td>Polypropylene Fibers (Fibermesh by Synthetic Industries)</td>
<td>0.0%</td>
</tr>
<tr>
<td>Unit Weight</td>
<td>142 pcf</td>
</tr>
<tr>
<td>Air Content</td>
<td>6.1%</td>
</tr>
<tr>
<td>Slump</td>
<td>3.25”</td>
</tr>
</tbody>
</table>
**Table 3.2-5** 0.5% Fiber Fraction Concrete Mix Proportions and Properties (Ready Mix)

<table>
<thead>
<tr>
<th>Constituent Material</th>
<th>Constituent / yd³ of concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement (Type I)</td>
<td>622 lbs.</td>
</tr>
<tr>
<td>Fly ash (Class C)</td>
<td>148 lbs.</td>
</tr>
<tr>
<td>Water</td>
<td>243 lbs.</td>
</tr>
<tr>
<td>Water to Cementitious materials ratio ((w/(c+f)))</td>
<td>.32</td>
</tr>
<tr>
<td>Limestone Aggregates (3/4” max. – Bethany Falls)</td>
<td>1735 lbs.</td>
</tr>
<tr>
<td>Sand</td>
<td>1329 lbs.</td>
</tr>
<tr>
<td>Air Entraining Agent (Daravair 1000)</td>
<td>3.4 oz/100 lb. cement</td>
</tr>
<tr>
<td>Super-plasticizer (Advacast 530)</td>
<td>183 oz.</td>
</tr>
<tr>
<td>Polypropylene Fibers (Fibermesh by Synthetic Industries)</td>
<td>0.5%</td>
</tr>
<tr>
<td>Unit Weight</td>
<td>133 pcf</td>
</tr>
<tr>
<td>Air Content</td>
<td>8.7%</td>
</tr>
<tr>
<td>Slump</td>
<td>2.75”</td>
</tr>
</tbody>
</table>
Table 3.2-6  Concrete Test Batch Summary

<table>
<thead>
<tr>
<th>Batch</th>
<th>Fiber Fraction</th>
<th>Batch Type</th>
<th>Use</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.0%</td>
<td>Lab Prepared</td>
<td>Compression Response Characterization</td>
</tr>
<tr>
<td>2</td>
<td>0.5%</td>
<td>Lab Prepared</td>
<td>Compression Response Characterization</td>
</tr>
<tr>
<td>3</td>
<td>1.0%</td>
<td>Lab Prepared</td>
<td>Compression Response Characterization</td>
</tr>
<tr>
<td>4</td>
<td>0.0%</td>
<td>Ready-Mix</td>
<td>Flexural Bond Testing</td>
</tr>
<tr>
<td>5</td>
<td>0.5%</td>
<td>Ready-Mix</td>
<td>Flexural Bond Testing</td>
</tr>
<tr>
<td>6</td>
<td>0.0%</td>
<td>Ready-Mix</td>
<td>Flexural Bond Testing</td>
</tr>
<tr>
<td>7</td>
<td>0.5%</td>
<td>Ready-Mix</td>
<td>Flexural Bond Testing</td>
</tr>
<tr>
<td>8</td>
<td>0.0%</td>
<td>Ready-Mix</td>
<td>Flexure Shear Testing</td>
</tr>
<tr>
<td>9</td>
<td>0.5%</td>
<td>Ready-Mix</td>
<td>Flexure Shear Testing</td>
</tr>
<tr>
<td>10</td>
<td>0.0%</td>
<td>Ready-Mix</td>
<td>Full Scale Slab Testing</td>
</tr>
<tr>
<td>11</td>
<td>0.5%</td>
<td>Ready-Mix</td>
<td>Full Scale Slab Testing</td>
</tr>
<tr>
<td>12</td>
<td>0.5%</td>
<td>Ready-Mix</td>
<td>Full Scale Slab Testing</td>
</tr>
</tbody>
</table>

3.3 CONCRETE COMPRESSION TEST

3.3.1 CONCRETE COMPRESSION TEST PROGRAM

Compression tests were performed on twelve different batches of concrete total. The outline of these batches, used for compression tests are shown in Table 3.2-6.

Uniaxial compression tests were performed between 7 days to 156 days on both 4” and 6” diameter concrete cylinders to characterize the compressive response over a period of time. This allowed analysis of both strength and the time increase of strength of the concrete specimens.
Two types of compression tests were performed on the cylinders obtained during these experiments: closed loop and open loop.

**Open Loop Compression Test**

The open loop compression test is the procedure most commonly used to determine the compressive strength of concrete cylinders for typical construction concrete strength monitoring. A photograph of the open loop compression test set-up is shown in Figure 3.3-1. The tests were conducted with a hydraulic Forney compression machine with 200 kip capacity.

In the open loop test the rate of loading is set by the controller and the response of the specimen is recorded by the instrumentation and the data acquisition devices. The cylinders are fitted with aluminum compression rings securely attached to the sides of the cylinders via compression screws (see Figure 3.3-1). Three LVDT (Linear Variable Displacement Transducers) are mounted to the compression rings to detect the axial deflection of the specimens. The LVDT’s are evenly spaced at 120° from each other and their signals are averaged to obtain a more accurate reflection of the core response of the cylinder even if deflection becomes unbalanced across the section. The load signal is sent to the data acquisition devices from the controller of the compression machine. The load and displacement signals are collected with a CPU employing the use of a National Instruments DAQ (Data AcQuisition) card and LabView programming software. The set-up provides the ability to define the stress-strain response of the compression
cylinders up to ultimate load. These tests are typically marked by sudden failure of the specimens at or shortly after peak strength, so a post-peak response is generally not available from this testing procedure. However, some of the fiber concrete specimens were able to maintain enough post-peak strength to provide significant post-peak data even under open loop testing.
Closed Loop Compression Test

One of the main advantages, of the closed loop compression test compared to the open loop compression test, is the ability of the closed loop test to provide post-peak compression data. The closed loop test differs from the open loop test by continually monitoring the response of the test specimen and adjusting the controller to facilitate a more controlled machine response. During the closed loop compression tests, the specimens are continually monitored via a circumferential extensometer that measures the circumferential expansion of the compression specimens under uniaxial loading due to Poisson’s effect. The tests are set to load the specimen at a uniform circumferential strain rate. This allows the equipment to adjust the loading rate (most importantly at post-peak) so that a stress-strain response can be obtained for both the pre and post-peak regimes.

Figure 3.3-2  Overall Closed Loop Compression Test Set-Up
Figures 3.3-2 and 3.3-3 show photographs of the closed loop compression test set-up. The closed loop tests are performed with an MTS 110 kip hydraulic actuator and controller. The instrumentation is quite similar to the open loop tests mentioned previously. The same type of LVDT set-up is used. In addition to the axial deflection, the circumferential expansion was also measured. Again the data is recorded via the use
of a CPU employing the use of a National Instruments DAQ (Data AcQuisition) card and LabView programming software. This test setup provides data for pre and post-peak stress-strain response of the concrete compression cylinders.

### 3.4 FRP Tension Test

#### 3.4.1 FRP Tension Test Program

Nine FRP (fiber reinforced polymer) rebar tension test specimens were received from Hughes Brothers Inc. There were three of each type of bar used for the tension test: #8 GFRP, #4 GFRP, and #4 CFRP (GFRP = Glass FRP and CFRP = Carbon FRP). These specimens were prepared at the ends with steel tube grips to prevent crushing of the FRP rebar since this type of rebar is relatively weak under lateral compression. The bars were set in the steel tube grips using expansive cement.

NOTE: The size designation follows closely with standard steel rebar size designation where the size of a rebar is given as the diameter of the bar in the number of eighths of an inch (i.e. A #4 rebar has a diameter of four eighths of an inch or 1/2” diameter).

#### 3.4.2 FRP Tension Test Procedure, Instrumentation, & Data Acquisition

The specimens were placed in a 300 kip hydraulic load frame via the use of wedge grips. The tension test specimens were loaded to failure under constant displacement control. An LVDT was used to measure the elongation of the FRP bars. An internal load cell, in the hydraulic load frame, was used to measure the applied load.
The data was recorded via the use of a CPU employing the use of a National Instruments DAQ card and LabView programming software. Load and displacement data were collected for the tests from the beginning of the test up to tensile failure.

**Figure 3.4-1**  FRP Tension Test Set-Up
3.5 Beam Flexural Fatigue Test

3.5.1 Beam Flexure Shear Test Program

Thirty-six beams were tested in a 4-point bending configuration as shown in Figures 3.5-1 and 3.5-2.

Figure 3.5-1  Beam Flexure Shear Test Set-Up
The beams were all 6” wide by 9.5” tall with a reinforcement depth (d) of 8” and were tested according to the schedule shown in Table 3.5-1. Half of the beams were plain concrete and half included 0.5% by volume polypropylene fibrillated fibers in the

**Figure 3.5-2**  Photograph of Beam Flexure Shear Test Set-Up

**Table 3.5-1**  Beam Test Schedule

<table>
<thead>
<tr>
<th>Rebar</th>
<th>Fiber volume Fraction = 0.5%</th>
<th>Fiber volume Fraction = 0.0%</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Static</td>
<td>High Fatigue</td>
</tr>
<tr>
<td>#4 GFRP</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>#8 GFRP</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>#4 CFRP</td>
<td>2</td>
<td>2</td>
</tr>
</tbody>
</table>
concrete mix. There were three different types of beams identified by the type of rebar used. Two of the beams, for each type of beam, were first tested under quasi-static, displacement controlled loading to determine the initial strength and stiffness properties of the beams. Then two different types of fatigue tests were performed on the remaining specimens. The fatigue tests were load controlled tests that continued for 1.2 million cycles. Two beams were tested in low fatigue and two beams were tested at high fatigue. During the low fatigue tests, the specimens were loaded cyclically from 30% to 60% of the ultimate strength of that type of beam as determined by the previous static tests. The value of 60% was chosen because under current load and resistance factor design criteria for steel reinforced concrete, the design service loads are typically close to 60% of the final ultimate design strength. During the high fatigue tests, the specimens were loaded cyclically from 40% to 80% of their ultimate strength. In both the high and low fatigue tests, the load was applied at a rate of five hertz. These tests were performed to determine what, if any, fatigue capacity the specimens may have had at high levels of repetitive loading. None of the beams tested at 80% of their ultimate capacity survived the 1.2 million cycle fatigue test. It should be noted that data for every beam is not presented in the results section. Some of the beams failed due to equipment malfunction and therefore produced no useful data.

It is worth noting here, that during a different phase of the overall project, of which this research is a smaller part, a bridge deck slab was designed using the current ACI 440 Guide for the Design and Construction of Concrete Reinforced with FRP Bars. This guide allows for design using limit states principles. The members are designed for required strength and then checked for fatigue and creep rupture endurance and
serviceability criteria. In most cases the fatigue, creep, and serviceability criteria will control the design of the members. The design of the bridge slab was no exception. The nominal strength of the slab was over three times the design service load in order to accommodate the serviceability criteria. This would imply that service loads would probably not be anywhere close to the 60% ultimate strength level in an FRP reinforced concrete member. Rather, the service loads would probably be closer to 30% of ultimate strength, so it will be important to bear in mind that the fatigue results of this experiment are based on a loading scheme that applied loads that are in the range of double the expected design service loads that would be applied to the members.

There were three general types of beams tested in the experimental process and each type was tested with and without the addition of 0.5%, by volume, fibrillated polypropylene fibers. The types of beams were differentiated based on the type and amount of FRP flexural reinforcement included. The reinforcement ratios and number of bars for each type of beam are indicated in Table 3.5-2. All reinforcement ratios are reported in terms of the balanced reinforcement ratio.

<table>
<thead>
<tr>
<th>Beam Type</th>
<th>Reinforcement Ratio ($\rho$)</th>
<th>Number of Bars</th>
</tr>
</thead>
<tbody>
<tr>
<td>#4 GFRP</td>
<td>$1.61 \rho_{\text{bal}}$</td>
<td>2</td>
</tr>
<tr>
<td>#8 GFRP</td>
<td>$2.99 \rho_{\text{bal}}$</td>
<td>1</td>
</tr>
<tr>
<td>#4 CFRP</td>
<td>$1.96 \rho_{\text{bal}}$</td>
<td>2</td>
</tr>
</tbody>
</table>

Table 3.5-2  Beam Reinforcement Data
3.5.2 Beam Flexure Shear Test Procedure, Instrumentation, & Data Acquisition

The beam flexure shear tests were closed loop, load controlled tests. The tests were controlled by an MTS 458.20 microconsole and the loading was applied via an MTS 311.11 load frame with a 220 kip actuator.

During the beam tests, load, rebar end slip, mid-span deflection, and ram deflection data were recorded. Figure 3.5-2 shows the setup of the equipment used for acquiring the data. All data were recorded via CPU and data acquisition software at regular intervals. MTS hydraulic ram deflection was recorded through the integral linear voltage displacement transducer (LVDT) in the MTS ram. Rebar end slip was recorded through two LVDT mounted at the ends of the rebar which protruded 2” outside of the beam specimens. Mid-span deflection was also monitored with an LVDT. Finally, load was also measured with the use of an MTS 100 metric ton load cell.

An additional LVDT for measurement of the mid-span deflection, beside the ram LVDT, was required to avoid adding deflections in the ram itself and in the loading fixtures to values of mid-span deflection. The separate mid-span LVDT, placed directly in contact with the test specimen, allowed the displacement to be measured without including the effects of the other factors previously mentioned.
4.1 COMPRESSION TESTING AND CHARACTERISTICS

It has long been known that the compressive strength and ductility of many materials are increased under conditions of triaxial compression. This can be easily seen in a relatively weak material such as soil. The addition of lateral confining stresses allows the soil to withstand much greater axial compressive stresses before failure. Although it is not as easily detected with the normal human senses, a similar phenomenon occurs in concrete. As early as 1928, Richart, Brandtzaeg, and Brown (Richart, et. al, 1928) developed a relationship between the 28 day compressive strength of concrete and concrete strength under applied triaxial stress. They proposed the following equation to define the compressive strength of concrete under applied triaxial stress.

\[
f'_{cc} = f'_c + 4.1f_l
\]

Where

- \( f'_{cc} \) = axial compressive strength of confined specimen
- \( f'_c \) = uniaxial compressive strength of unconfined specimen
- \( f_l \) = lateral confining pressure
- \( 4.1 \) = empirical constant

Since then, others have performed similar tests to establish a correlation between applied lateral confinement stresses and an increase in the peak compressive strength.
The proposals have generally followed the same basic equation with some modification to the empirical constant proposed by Richart et al.

Besides increasing the compressive resistance of concrete, confinement of the concrete compression specimen provides a way to make the formerly brittle concrete matrix behave in a more ductile manner. If the concrete can be made to have sufficient ductility, in the case of FRP reinforced concrete, there is now a structural member with one brittle (FRP) and one ductile (concrete) component and the system can be designed so that the ductile component begins to significantly deform at its limit state while the non-ductile component remains at a stress level significantly below its ultimate strength.

In practice, concrete is often confined by reinforcing spirals or hoops, but a somewhat similar effect can be achieved by the addition of fibers into the concrete matrix. Shah, Fafitas, and Arnold (Shah, et al, 1983) proposed a model to define the behavior or concrete confined by ties or spirals. The model defines concrete behavior in the pre-peak and post-peak regimes. In the pre-peak regime the strength of the concrete is defined as

\[
f = f_0 \left[1 - \left(1 - \frac{\varepsilon}{\varepsilon_0}\right)^A\right] \quad \varepsilon_{\text{cr}} \leq \varepsilon_0 \quad \text{[psi]} \quad \{\text{Eqn. 4.1-2}\}
\]

with…

\[
f_0 = f'_c + \left(1.15 + \frac{3048}{f'_c}\right) f_r \quad \text{[psi]} \quad \{\text{Eqn. 4.1-3}\}
\]

\[
f_r = \frac{2 A_s f_y}{d} \left(\frac{1}{s} - \frac{1}{1.25d}\right) \quad \text{[psi]} \quad \{\text{Eqn. 4.1-4}\}
\]

\[
\varepsilon_{\text{cr}} = 1.027 \times 10^{-7} f'_c + 0.0296 \left(\frac{f_r}{f'_c}\right) + 0.00195 \quad \{\text{Eqn. 4.1-5}\}
\]
\[ A = \frac{E_c \varepsilon_o}{f_o} \]  \hspace{1cm} \{\text{Eqn. 4.1-6}\}

where

\[ f_o = \text{strength of confined concrete} \quad [\text{psi}] \]
\[ f'_c = \text{strength of unconfined concrete} \quad [\text{psi}] \]
\[ f_r = \text{confinement stress or index} \quad [\text{psi}] \]
\[ \varepsilon = \text{concrete strain} \]
\[ \varepsilon_o = \text{concrete peak strain} \]
\[ A = \text{empirical constant} \]

In the post-peak regime the concrete strength is defined as…

\[ f = f_o e^{-k(\varepsilon-\varepsilon_o)^{0.15}} \quad [\varepsilon \geq \varepsilon_o] \quad [\text{psi}] \]  \hspace{1cm} \{\text{Eqn. 4.1-7}\}

with…

\[ k = 0.17 f'_c e^{-0.01f_r} \]  \hspace{1cm} \{\text{Eqn. 4.1-8}\}
In this experimental regimen, an attempt was made to determine the amount of “confinement” contributed by the specific polypropylene fibers used in these experimental FRC beams. The goal was to replace the confinement stress \( f_c \) proposed by Shah et al. with a corresponding “confinement” stress based on the fiber fraction by volume introduced into the concrete matrix.
Three different fiber fraction volumes were used for this evaluation (0.0%, 0.5%, and 1.0%). Concrete was prepared in the lab and 4” diameter by 8” tall cylinders were prepared each with similar concrete mixes but different amounts of fiber included. The fibers were added to the aggregates and pre-mixed prior to the inclusion of the water and cement to insure sufficient dispersal of the fibers. While placing the concrete, no significant “balling” of the fibers was noticed. After the cylinders were prepared they
were placed in a 100% humidity curing room to allow proper curing until the
compression tests were performed.

To determine the concrete compressive response throughout the pre and post-peak
regimes of the stress vs. strain curve, quasi-static closed loop compression tests were
conducted on the 4”x8” cylinders. The compression tests were carried out with an MTS
hydraulic actuator and controller. The strain rate for the test was regulated via the use of
a circumferential extensometer and was based on the rate of circumferential expansion of
the cylinder. The axial displacement of the specimens was measured through the use of
three linear variable displacement transducers (LVDT). A photograph of the
compression test setup can be seen in Figure 4.1-1 and 4.1-2.

![FRC Matrix Cross-Section](image)

**Figure 4.1-3** The “Confinement Analogy” Model

The stress strain response of the test specimens was compared to some theoretical
stress-strain values obtained by plotting some curves based on the previously discussed
confined concrete model by Shah et al. The premise of this comparison is taken from the
“confinement analogy” developed by Gopalaratnam and El-Shakra (El-Shakra, 1995). The confinement analogy treats the internal fiber forces in the concrete matrix as “confinement” to the concrete matrix. It can be shown, that the internal crack closing stresses ($F_\tau$) through a cross-section of a concrete matrix (Figure 4.1-3) are equal to an equivalent confinement pressure ($f_c$). The confinement analogy uses the Shah et al. model but modifies the confinement stress to correspond to the resistance provided by the fibers instead of stirrups or ties. The confinement analogy was based on a large volume of data in which steel fibers were used. The experimental data presented in the El-Shakra research matched quite closely to the theoretical prediction. The fibers used in this current experimental program however, were polypropylene and their material properties vary greatly from those of steel. Therefore it is necessary to examine the behavior of the polypropylene fibers in this experiment to determine if the confinement analogy is a viable method of predicting the behavior of the polypropylene FRC.

Figure 4.1-4 shows the typical normalized uniaxial stress vs. strain response of the concrete specimens created for this experiment. The stress values were normalized with respect to the ultimate stress of the specimens. As expected, the behavior of the specimens is quite similar up to peak, but in the post-peak regime there is an obvious difference in the level of stress that the FRC specimens were able to sustain compared to the plain concrete specimens. The fiber reinforced specimens exhibited greater toughness and ductility than the plain concrete. Of significance is the lack of improvement in the post-peak strength of the 1.0% fiber fraction concrete compared to those with 0.5% fiber fraction. This may lead us to believe that a significant advantage will not be gained by the addition of much more than 0.5% by volume fiber fraction when employing the use of
fibrillated polypropylene fibers. Due to this, and the fact that as the percentage of fiber fraction goes up the workability goes down, it seems as though there should be no need to increase the fiber fraction to greater levels than 0.5%. There is most definitely a more exact optimum fiber content but, that goes beyond the current level of experimentation performed here. For now it will suffice to say that a clear advantage was not obtained by the addition of fibers past the 0.5% fiber fraction by volume level.

**Figure 4.1-4** Normalized Uniaxial Compression Stress vs. Strain Response of Concrete with Different Percentage of Fiber Fraction by Volume

NOTE: The actual compressive strengths of the concrete cylinders used for this graph are as follows: 0.0%FF – 7,300 psi; 0.5%FF – 3,400 psi; 1.0%FF – 6800 psi
In Figures 4.1-5 and 4.1-6, the stress-strain data from the 0.0% and 0.5% fiber fraction by volume concrete compression test specimens are compared to the “confinement analogy” prediction. The comparison of the 1.0% specimens was not shown due to its similarity to the 0.5% specimens. Again all of the values are normalized with respect to peak stress.

**Figure 4.1-5** Normalized Uniaxial Compression Stress vs. Strain Response of 0.0% Fiber Fraction by Volume Concrete Compared to “Confinement Analogy” Results

**NOTE:** The theoretical stress-strain curves are normalized with respect to the “0 psi Lateral Confinement” curve to show the difference in the relative maximum stress predicted with more confinement stress.

The actual compressive strength of the plain concrete used for this graph was 7,300 psi.
From the compression test data shown in Figures 4.1-5 and 4.1-6, several observations can be made. The graphs of stress vs. strain demonstrate that a clear increase in post-peak ductility is achieved when fibers are added to the concrete matrix, especially at higher strains (greater than 0.004). The toughness of the concrete matrix is significantly increased by the addition of fibers as well.

**Figure 4.1-6** Normalized Uniaxial Compression Stress vs. Strain Response of 0.5% Fiber Fraction by Volume Concrete Compared to “Confinement Analogy” Results

NOTE: The theoretical stress-strain curves are normalized with respect to the “0 psi Lateral Confinement” curve to show the difference in the relative maximum stress predicted with more confinement stress.

The actual compressive strength of the plain concrete used for this graph was 7,300 psi.
Comparing the experimental data to the “confinement analogy”, it can be seen that the analogy slightly overestimates compressive strength immediately post-peak and underestimates strength at higher strains (> 0.005). It seems reasonable to compare the behavior of the 0.5% fiber fraction concrete to an equivalent lateral confinement of 50 psi for purposes of this investigation.

Figure 4.1-7  Close Up of Uniaxial Compression Test Setup
The difference in the post peak behavior of the compression specimens can also be seen in the appearance of the failed specimens. The appearance is easily distinguished from the typical open loop compression test specimens. Figure 4.1-7 shows a photograph on the open-loop compression test setup and Figure 4.1-8 shows two failed 6” diameter specimens. One of the specimens was of plain concrete and the other had 0.5% fiber content. The plain concrete specimen shows a typical break pattern with the side completely separating from the original specimen. The fiber concrete however remained intact even through a relatively large displacement.

Another item worthy of noting is the behavior of the concrete compression specimens subjected to open-loop compression tests. In the open-loop compression tests, the plain concrete specimens broke in a brittle manner with almost complete loss of
strength immediately after peak load. This type of failure is common with plain concrete. Conversely, the FRC specimens did not experience a brittle rupture, but continued to maintain partial strength after peak loading. This is significant, in part, because of the manner in which the load is applied in an open-loop vs. a closed-loop compression test. In a lab, it is possible to obtain post-peak strength even in plain concrete because, in a closed-loop test, the applied loading is strain controlled. In a structure, under natural loading (i.e. wind, gravity, seismic), the forces are applied in a load control manner. This does not allow for stable fracture in softening materials. Wind and seismic events will not limit themselves just because peak stress has been reached in the concrete. For this reason, the open-loop compression tests may apply a loading that is more similar to the loading experienced in service. The ability of the FRC to maintain partial strength in the open-loop compression test indicates it will be more able to resist overloading that is more similar to service conditions.
CHAPTER 5 - FRP TENSION TEST RESULTS AND DISCUSSION

5.1 FRP CHARACTERISTICS AND TENSION TEST RESULTS

Two categories of FRC reinforced concrete member failure were of interest for this experimental investigation, brittle and ductile. It is desirable for any civil structure to be designed so that if failure is imminent, it will not be sudden or catastrophic. The brittle nature of typical FRP tension reinforcement failure can directly lead to the aforementioned sudden failure. For this reason the linear elastic behavior of FRP bars, from initial loading all the way to failure, is of prime importance to these evaluations.

FRP reinforcement has several distinct advantages over conventional steel reinforcement. Some of the more significant advantages common to all FRP’s are…

1) They are impervious to chloride ion attack.
2) They have tensile strength greater than steel.
3) They are approximately 1/4 to 1/5 the weight of steel rebar.

The ability of FRP’s to resist corrosion by chloride ions gives the FRP reinforcement the distinct advantage of greater longevity in circumstances where reinforced concrete will encounter chlorides such as in areas close to sea water or in pavements that have deicing salts applied during wintry weather.

The high tensile strength is also useful and the lightweight nature of the FRP’s allows for ease of construction as compared to conventional steel rebar. The mechanical properties of the rebar used in this experimental investigation are shown in Tables 5.1-1
and Table 5.1-2. The values in Table 5.1-1 are taken from the manufacturer’s (Hughes Brothers Inc.) provided data and the values in Table 5.1-2 are taken from tensile tests performed at the University of Missouri – Columbia Civil Engineering lab.

### Table 5.1-1  Manufacturer’s Mechanical Properties of FRP Bars

<table>
<thead>
<tr>
<th>Bar Size / Type</th>
<th>Cross-Sectional Area (in²)</th>
<th>Nominal Diameter (in)</th>
<th>Tensile Strength (ksi)</th>
<th>Tensile Modulus of Elasticity (psi x10⁶)</th>
<th>Ultimate Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>#4 CFRP</td>
<td>0.1679</td>
<td>0.47</td>
<td>300</td>
<td>18.0</td>
<td>0.017</td>
</tr>
<tr>
<td>#4 GFRP</td>
<td>0.2245</td>
<td>0.5</td>
<td>100</td>
<td>5.92</td>
<td>0.017</td>
</tr>
<tr>
<td>#8 GFRP</td>
<td>0.8337</td>
<td>1.0</td>
<td>80</td>
<td>5.92</td>
<td>0.14</td>
</tr>
</tbody>
</table>

### Table 5.1-2  Mechanical Properties of FRP Bars

<table>
<thead>
<tr>
<th></th>
<th>Modulus (psi)</th>
<th>Max Stress (ksi)</th>
<th>Max Strain (μstr)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Individual</td>
<td>Average</td>
<td>Individual</td>
</tr>
<tr>
<td>#4 CFRP</td>
<td>17,080,000</td>
<td>19,100,000</td>
<td>276,400</td>
</tr>
<tr>
<td></td>
<td>21,510,000</td>
<td></td>
<td>280,500</td>
</tr>
<tr>
<td></td>
<td>18,720,000</td>
<td></td>
<td>301,500</td>
</tr>
<tr>
<td>#4 GFRP</td>
<td>4,410,000</td>
<td>4,230,000</td>
<td>79,900</td>
</tr>
<tr>
<td></td>
<td>4,140,000</td>
<td></td>
<td>78,500</td>
</tr>
<tr>
<td></td>
<td>4,130,000</td>
<td></td>
<td>81,000</td>
</tr>
<tr>
<td>#8 GFRP</td>
<td>5,550,000</td>
<td>4,920,000</td>
<td>65,500</td>
</tr>
<tr>
<td></td>
<td>5,100,000</td>
<td></td>
<td>66,500</td>
</tr>
<tr>
<td></td>
<td>4,100,000</td>
<td></td>
<td>65,000</td>
</tr>
</tbody>
</table>
The laboratory test values of the GFRP bars showed a significant deviation from the values provided by the manufacturer. For the #4 GFRP bars, the average tensile strength and modulus were 20.2% and 18.7% less than listed by the manufacturer respectively. For the #8 GFRP bars, the average tensile strength and modulus were 17.9% and 5.4% less than listed by the manufacturer respectively. For the #4 CFRP bars, the elastic modulus was 6.1% higher than the manufacturer’s listed minimum value but the tensile strength was 4.6% less than listed by the manufacturer.

While FRP reinforcement has several distinct advantages over conventional steel reinforcement, there are also several disadvantages. These include…

1) Relatively low tensile modulus of elasticity
2) Brittle failure mode (see Figures 5.1-1 & 5.1-2)
3) Relatively low shear strength
4) For GFRP, the alkali concrete matrix is corrosive to the individual glass fibers.

The low modulus of elasticity can lead to excessive deflections and crack widths of concrete members reinforced with FRP. As mentioned previously, under the current ACI 440 design philosophy, the design of FRP reinforced concrete is usually governed by serviceability criteria. This is due, in large part, to the relatively low elastic modulus of FRP.
The brittle nature of FRP reinforcement is important when considering the failure mode of concrete structures reinforced with it. Unlike its steel rebar counterpart, FRP rebar remains linear elastic until failure (Figure 5.1-3). A conventional steel rebar has a linear elastic range followed by a yield plateau that progresses into a strain hardening regime (Figure 5.1-4). For years, engineers have taken advantage of the high strength, modulus, and ductility of steel reinforcement to design steel reinforced concrete with a mode of failure, in flexure, that is typically ductile. This is possible because only
one of the constituent materials is brittle. Since the steel is ductile, the system is designed so that, at ultimate strength, the concrete stress will remain far below its maximum capacity and the ductile steel rebar will begin to yield, producing a large member deflection. Reinforced concrete designed in this manner is typically referred to as under-reinforced. In fact, the current ACI 318 building code (ACI, 2002) places a maximum limit on the amount of flexural reinforcement to insure that the yielding of steel will
occur prior to concrete crushing when the strength limit state of the member is surpassed. In the concrete/FRP system both materials are brittle and therefore the standard reinforced concrete design method is not sufficient to produce a ductile structural member.

![Typical Stress-Strain Curve for GFRP rebar.](image)

**Figure 5.1-3** Typical Stress-Strain Curve for GFRP rebar.

ACI 440 Guide for the Design and Construction of Concrete Reinforced with FRP Bars (ACI, 2000) recommends that, for FRP RC design, instead of designing for the failure (yielding) of the rebar, the system should be “over-reinforced” so that the concrete will crush before the rebar will reach its rupture point. According to ACI 440, this can be
accomplished by requiring that the reinforcement ratio is greater than 1.4 times the balanced reinforcement ratio. Although the choice of concrete crushing is definitely a better choice than brittle failure of the rebar and immediate collapse of the structure, it still lacks the same level of ductility the engineering community has come to enjoy with steel reinforcement. Currently this lack of ductility at failure is countered in design by employing larger safety factors than have been used in conventionally reinforced concrete.

![Typical Stress-Strain Curve for 60 Grade Steel Rebar](image)

**Figure 5.1-4**  Typical Stress-Strain Curve for 60 Grade Steel Rebar
CHAPTER 6 - FLEXURE SHEAR TEST RESULTS AND DISCUSSION

6.1 INTRODUCTION

The results presented in this chapter are from the flexure shear beam tests described in Chapter 3. Representative data is presented alongside the discussion of the experimental testing results in this chapter. Complete experimental test data can be found at the end of this report in Appendices A through C.

There are two items that are critical to keep in mind when considering the results presented in this chapter:

1) The load resistance mechanism, for the beam size and testing configuration used in this experimental investigation, is dominantly a shear resistance mechanism. Flexural resistance of the beam specimens, to the applied loading, is secondary to the shear resistance.

2) Due to difficulties in mixing the additional fibers in the ready-mix truck at the testing site, there was an unusually high air content (8-10%) for the concrete mix with 0.5% polypropylene fibers. This additional air content caused the compressive strength of the fiber mix to be significantly lower (approximately 25%) than the plain concrete mix even though the mix designs were almost identical otherwise.
6.2 NOMENCLATURE

In the following discussion graphs and figures, the test specimens are identified in the following manner.

**F 8 G 1**

- Specimen Number (first or second of single type)
- Type of Reinforcement (G = GFRP & C = CFRP)
- Size of Reinforcement (#8 Bar or #4 Bar)
- Type of Concrete Mix (F = Fiber concrete or N = No fiber / plain)

OR……..

**F 8 G ST**

- Specimen Type (ST = static or LF = low fatigue)
- Type of Reinforcement (G = GFRP & C = CFRP)
- Size of Reinforcement (#8 Bar or #4 Bar)
- Type of Concrete Mix (F = Fiber concrete or N = No fiber / plain)

6.3 MODES OF FAILURE

Prior to the main discussion of the resultant behavior of the test beams, it will be useful to have a brief discussion of the typical modes of shear failure observed in reinforced concrete beams.
The shear failure mechanism in reinforced concrete beams is characterized by inclined or diagonal cracks. Generally, the shear crack initiates just above the longitudinal tension reinforcement in the location of a previously formed flexural crack (El-Shakra, 1995).

6.3.1 Diagonal Tension Failure

In this type of shear failure, the shear crack extends along a diagonal along the entire depth of the beam. The failure is generally sudden and brittle. This type of failure is common when the shear span to depth ratio is approximately 2.5.

6.3.2 Shear Tension Failure

This type of failure occurs when the shear crack extends along the tension reinforcement. Subsequent debonding and bond splitting of the tension reinforcement occurs at the level of the tension reinforcement. The initiation of the failure is the shear crack, but the subsequent bond splitting and loss of dowel resistance is responsible for the ensuing failure. This type of failure is generally brittle and explosive (El-Shakra, 1995).

6.3.3 Shear Compression Failure

When beams have a shear span of less than 2.5 and there is adequate bond anchorage of the tensile reinforcement, this type of failure becomes more common. In this case the shear crack grows in a stable manner and the beam begins to resist the loading by arching action. The tensile reinforcement acts as a “tie” and the concrete above the cracked section acts as a compression “strut”. If the uncracked section becomes too small and the applied load exceeds the compressive strength of the concrete, a compression failure of the section will occur. This type of failure is generally less explosive than the other types of shear failure, but still considered brittle.
6.4 Static Flexure Tests of Plain Concrete Beams

To establish the initial stiffness, ultimate strength, and failure characteristics of the plain concrete test beam specimens, two, of each type of beam (#4 GFRP, #8 GFRP, & #4CFRP), were tested to failure under quasi-static loading (see Appendix-A for complete results). Load, mid-span deflection, and rebar end slip were all recorded during the static tests. Figures 6.4-1 – 6.4-13 show graphs of load vs. mid-span deflection, rebar end slip vs. mid-span deflection, and corresponding photographs of the failed specimens. Both the plain and FRC concrete beam specimen test results are shown on the same graphs for comparison.

Figure 6.4-1  Static Load vs. Mid-Span Deflection Response of Beams Reinforced with One (1) #8 GFRP Rebar
Figure 6.4-2  Static Rebar End-Slip vs. Mid-Span Deflection Response of Beams Reinforced with One (1) #8 GFRP Rebar

Figure 6.4-3  Photograph of Failed Static Bending Specimen with One (1) #8 GFRP Rebar and No Fibers
Figure 6.4-4  Photograph of Failed Static Bending Specimen with One (1) #8 GFRP Rebar and 0.5% Fiber Fraction

Figure 6.4-5  Static Load vs. Mid-Span Deflection Response of Beams Reinforced with Two (2) #4 GFRP Rebar
**Figure 6.4-6**  Static Rebar End-Slip vs. Mid-Span Deflection Response of Beams Reinforced with Two (2) #4 GFRP Rebar

**Figure 6.4-7**  Photograph of Failed Static Bending Specimen with Two (2) #4 GFRP Rebar and No Fibers
**Figure 6.4-8** Close Up Photograph of Shear Compression Failure of Failed Static Bending Specimen with Two (2) #4 GFRP Rebar and No Fibers

**Figure 6.4-9** Photograph of Failed Static Bending Specimen with Two (2) #4 GFRP Rebar and 0.5% Fiber Fraction
Figure 6.4-10  Static Load vs. Mid-Span Deflection Response of Beams Reinforced with Two (2) #4 CFRP Rebar

Figure 6.4-11  Photographs of Failed Static Bending Specimen with Two (2) #4 CFRP Rebar and No Fibers
Figure 6.4-12  Static Rebar End-Slip vs. Mid-Span Deflection Response of Beams Reinforced with Two (2) #4 CFRP Rebar

Figure 6.4-13  Photograph of Failed Static Bending Specimen with Two (2) #4 CFRP Rebar and 0.5% Fiber Fraction
For the plain concrete beams, the mode of failure varied, depending mostly on the reinforcement ratio. For the #8 GFRP and #4 CFRP beams with $\rho = 2.99\%$ and $\rho = 1.96\%$ respectively, the beam failure was due to shear tension failure (see Table 6.4-1). For the #4 GFRP beams with $\rho = 1.61\%$, the beam failure was due first to shear compression in the uncracked upper region of the beam and then later shear tension with bond splitting action followed.

In Section 11.1.2 of the ACI 440 Document, it states that when concrete cover is less than or equal to two bar diameters a splitting failure can occur; if the concrete cover exceeds two bar diameters, a pullout failure is more likely (ACI, 2001). In the case of the #8 GFRP beams, the concrete cover was one (1) bar diameter, so it is not surprising that a shear tension failure leading to bond splitting was experienced (Table 6.5-1). On the other hand, in the case of the #4 CFRP beams the concrete cover was 2.5 bar diameters, so according to the current ACI 440 recommendations, a pullout type of failure would seem to be more likely under the previously mentioned criteria. In the following paragraphs, the reasons for the shear tension and ensuing bond splitting failure of the #4 CFRP specimens will be presented.

Generally, splitting failure of concrete is also prevented by providing sufficient development length for the rebar so that the bond stress does not become excessive. The recommended development length (from ACI 440, Eqn. 11-6) for bars where failure by splitting failure controls is

$$l_d = K_2 \frac{d^2 f_{tu}}{\sqrt{f'c}} \quad \{\text{Eqns. 6.4-8}\}$$

Where…

93
K2 = 1/(16.2) [empirical constant]

*Note: the above K2 value is based on an average of experimental values reported for reference in ACI 440, Section 11.1.*

\[ f'c = 5,000 \text{ psi} \]

Using this equation gives recommended development lengths of \( l_d (\#8 \text{ GFRP}) = 70" \), \( l_d (\#4 \text{ CFRP}) = 65" \) and \( l_d (\#4 \text{ GFRP}) = 20" \). In the case of both the #8 GFRP and the #4 CFRP beams, the provided development length of 20” is far below the recommended. The insufficient development length causes the bond stress to become higher than the concrete matrix can withstand and helps to promote a bond splitting failure as was experienced in the tests. In the case of the #4GFRP beams, the recommended development length was provided and the tensile reinforcement was sufficiently anchored to cause the beam to exhibit obvious signs of arching action and shear compression failure. This would tend to imply that the provision of sufficient development length for the reinforcement will help to minimize bond splitting failure as we would expect. This was confirmed by Hutton (1997) when he documented concrete bond splitting failures for FRP bars with small development lengths.

The factors that may combine to induce bond splitting along the flexural reinforcement in the shear span of beams are:

1) Circumferential tensile stresses generated in the vicinity of each flexural crack.

2) Circumferential or transverse tensile stresses induced by wedging action of the deformations and by compressed concrete at the ribs when large bond forces
need to be transferred. (note: In the case of typical FRP reinforcement, the contribution of compressed concrete at the ribs will be significantly smaller than in deformed steel bars, because of the surface geometry differences between steel and FRP bars.)

3) Transverse tensile stresses resulting from dowel action of the flexural reinforcement. (Park & Paulay, 1975).

Gergely noted that dowel forces reduce the bond strength if no confining pressure is present, resulting in larger slips for a given load (Gergely, 1969).

High bond stresses combined with large dowel forces in the shear span of the beams are the likely causes of the shear tension failures, with bond splitting, that were experienced.
6.5 Static Flexure Tests of Fiber Reinforced Concrete Beams

In figures 6.4-1 through 6.4-13, the load vs. mid-span deflection and end-slip vs. mid-span deflection responses of the FRC beams are plotted. They accompany the similar plots for the plain concrete specimens.

In the initial stages (linear elastic material response) the behavior of the two different types of beams is quite similar as was expected. The behavior began to differ as the peak and post-peak zones were entered.

For the #8 GFRP beams, the peak strengths were quite similar even though the concrete strength of the FRP beams was only 74% (4450/6030) of the plain concrete beams. In the post-peak zone, the FRC beams exhibited a much more ductile response than the plain concrete beams. While the plain concrete beams failed in a brittle manner immediately after peak strength, the FRC beams were still holding approximately 1/3 of the peak load at twice the mid-span deflection of the peak load. It can easily be seen that the fibers acted to limit the shear tension failures so that a brittle failure was not experienced. As noted previously, Gergely’s research indicated that dowel forces reduce the bond strength if no confining pressure is present, resulting in larger slips for a given load (Gergely, 1969). This helps to explain why the FRC beams tended to fail in a more ductile fashion. Even though the onset of failure tended to be from shear tension with ensuing debonding of the reinforcement, the cracks were limited by the “confinement” provided by the internal fibers.

The #4 CFRP beams marked a much more significant improvement of peak strength, due to the addition of fibers, than the #8 GFRP beams. The FRC beams were able to carry approximately 20% more load than their plain concrete counterparts. Again
considering the relative compressive strengths of the concrete, this is significant. The additional peak strength was incurred because even after initial shear tension and bond split cracking, the beams were able to continue resisting even higher applied loads because the cracking was somewhat contained.

The #4 GFRP beams with FRC also exhibited higher peak strength than their plain concrete counterparts but for a different reason than the #4 CFRP FRC beams. The #4 GFRP FRC beam mode of failure was shear compression, the same as the plain concrete specimens. The previously discussed advantages of the internal “confinement” of the concrete can be seen in action in this case. The effective confinement of the compression concrete by the polypropylene fibers allowed the FRC beams to continue resisting higher loads than the plain concrete specimens.

The following Table 6.5-1 outlines the types of failure that were exhibited in the static beam tests.

The FRC beams exhibited a more ductile failure than the plain concrete beams. The #4 GFRP beams with $\rho = 1.61\rho_{bal}$ also exhibited a ductile failure as predicted by the ACI 440 document. For the beams without fibers and higher reinforcement ratios, shear tension with bond splitting was the common failure mechanism. The beams with fibers provided some restraint to the shear tension and bond splitting cracks and prevented sudden failure of the specimens. Instead of sudden failure, upon initiation of shear tension cracking, the rebar began to slip providing a much less violent failure.
Table 6.5-1  Mode of Failure for Static Flexure Shear Beams

<table>
<thead>
<tr>
<th>BEAM</th>
<th>CAUSE OF FAILURE</th>
<th>FAILURE NOTES</th>
</tr>
</thead>
<tbody>
<tr>
<td>N-8G-ST-01</td>
<td>Shear Tension</td>
<td>Brittle Failure</td>
</tr>
<tr>
<td>N-4G-ST-01</td>
<td>Shear Compression</td>
<td>Ductile Failure</td>
</tr>
<tr>
<td>N-4G-ST-02</td>
<td>Shear Compression</td>
<td>Ductile Failure</td>
</tr>
<tr>
<td>N-4C-ST-01</td>
<td>Shear Tension</td>
<td>Brittle Failure</td>
</tr>
<tr>
<td>N-4C-ST-02</td>
<td>Shear Tension</td>
<td>Brittle Failure</td>
</tr>
<tr>
<td>N-8G-LF-01</td>
<td>Shear Tension</td>
<td>Brittle Failure</td>
</tr>
<tr>
<td>N-8G-LF-02</td>
<td>Shear Tension</td>
<td>Brittle Failure</td>
</tr>
<tr>
<td>N-4G-LF-01</td>
<td>Shear Tension</td>
<td>Brittle Failure</td>
</tr>
<tr>
<td>N-4G-LF-02</td>
<td>Shear Tension</td>
<td>Brittle Failure</td>
</tr>
<tr>
<td>N-4C-LF-01</td>
<td>Shear Tension</td>
<td>Brittle Failure</td>
</tr>
<tr>
<td>N-4C-LF-02</td>
<td>Shear Tension</td>
<td>Brittle Failure</td>
</tr>
<tr>
<td>F-8G-ST-01</td>
<td>Shear Tension</td>
<td>Bond Splitting induced Slip of Rebar to begin Ductile Failure</td>
</tr>
<tr>
<td>F-8G-ST-02</td>
<td>Shear Tension</td>
<td>Bond Splitting induced Slip of Rebar to begin Ductile Failure</td>
</tr>
<tr>
<td>F-4G-ST-01</td>
<td>Shear Compression</td>
<td>Bond Splitting induced Slip of Rebar to begin Ductile Failure</td>
</tr>
<tr>
<td>F-4G-ST-02</td>
<td>Shear Compression</td>
<td>Bond Splitting induced Slip of Rebar to begin Ductile Failure</td>
</tr>
<tr>
<td>F-4C-ST-01</td>
<td>End Slip</td>
<td>Ductile Failure</td>
</tr>
<tr>
<td>F-4C-ST-02</td>
<td>End Slip</td>
<td>Ductile Failure</td>
</tr>
<tr>
<td>F-8G-LF-01</td>
<td>Shear Tension</td>
<td>Ductile Failure</td>
</tr>
<tr>
<td>F-8G-LF-02</td>
<td>Shear Tension</td>
<td>Ductile Failure</td>
</tr>
<tr>
<td>F-4G-LF-01</td>
<td>Shear Compression</td>
<td>Ductile Failure</td>
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<tr>
<td>F-4G-LF-02</td>
<td>Shear Compression</td>
<td>Ductile Failure</td>
</tr>
<tr>
<td>F-4C-LF-01</td>
<td>End Slip</td>
<td>Ductile Failure</td>
</tr>
<tr>
<td>F-4C-LF-02</td>
<td>End Slip</td>
<td>Ductile Failure</td>
</tr>
</tbody>
</table>
6.6 Fatigue Tests of Concrete Beam Specimens

The beam fatigue tests were continued for 1.2 million cycles. The tests loaded the beams cyclically from 30% to 60% of the ultimate load capacity of the beams. When using typical LRFD design methodology, the factor of safety for a typical steel reinforced concrete flexural member ranges from approximately 1.6 – 1.8. A 1.6 safety factor would give the maximum service load expected as $1/1.6 = 62.5\%$ of ultimate load. This would imply that our testing would be similar to having 1.2 million occurrences of the MODOT design truck crossing our structure if steel reinforced concrete was used. For FRP flexural members, the 60% of ultimate strength fatigue load will be significantly greater than the maximum design service loads. The reason is because, as stated previously, the service, creep, and fatigue criteria generally control the design of FRP reinforced flexural members. Therefore the 60% ultimate strength fatigue load will correspond more closely with double the design service load. This implies, also, that any degradation seen as a result of the fatigue loads applied in this program will be much more severe than those experienced in actual bridge deck slabs or concrete beams designed to meet serviceability criteria.

*NOTE: Previously, in this thesis, both low fatigue and high fatigue testing programs were mentioned. None of the high fatigue specimens survived the 1.2 million cycle high fatigue test regimen. All data, from here on, refers to low fatigue tests.*

Figures 6.6-1 – 6.6-3 show the percent loss of flexural stiffness, from initial stiffness, for the test specimens (see Appendix-B for complete data). The data is shown for all three types of beams, with and without fibers.
At the onset of the testing program, it was hoped that the addition of 0.5%, by volume, fibrillated polypropylene fibers into the concrete matrix, would provide a mechanism by which the flexural stiffness of the test beams would not degrade as quickly as non-fiber concrete beams under fatigue. The addition of fibers into concrete is generally known to minimize crack growth by bridging cracks that have formed and somewhat limiting their width, thereby retarding crack propagation. Concrete compression tests show that fiber reinforced concrete shows little difference in strength, compared to plain concrete, up to peak strength, but in the post peak regime the fiber...
reinforced concrete (FRC) exhibits a much greater ability to maintain strength, a higher toughness, and greater ductility than plain concrete.

Figure 6.6-2  Percent Degradation of Stiffness (from maximum) of Beams Reinforced with Two (2) #4 GFRP Rebar

It was a combination of the above factors that was hoped would cause the FRC beams to maintain their flexural stiffness better than the plain concrete beams. This is of special importance in the use of FRP reinforcement, due to its low modulus of elasticity (comparative to steel, $E_{GFRP} \approx (20\%)E_S$, $E_{CFRP} \approx 60\%E_S$), cracking is a more significant problem than with conventional steel reinforcement.
The test results show (Figures 6.6-1 – 6.6-3) that, almost without exception, the FRC beams experienced a higher rate of flexural stiffness degradation than the plain concrete specimens. This is likely not due to the action of the fibers as much as it is the problem of a poorer quality concrete in the FRC beams due to problems caused in the mixing process because of the addition of high quantities of fiber.

In fact, in flexural tests performed by Wang (2005), when polypropylene fibers were added to the concrete matrix, the bond stiffness degradation for CFRP reinforced specimens was up to 35% less and up to 25% less in GFRP reinforced specimens. Similar results were documented by Gopalaratnam et al. (2004). In tensile fatigue tests, the inclusion of polypropylene fiber noticeably decreased residual slip (Gopalaratnam et
al., 2006). This reinforces the conclusion that the addition of polypropylene fibers is not the cause of the increased degradation. If anything, these results should be used as a note of precaution that careful mixing procedures should be followed when adding higher amounts of fibers to concrete mixes.

General stiffness degradation trends include a high initial loss in stiffness after initial cracking of the beam and during the first 200,000 cycles. The typical percentage loss of stiffness during the first 200,000 cycles was approximately 45%. Thereafter, the stiffness of the specimens tended to stabilize with losses generally 5-10% more over the next one-million cycles.

In Figure 6.6-1, the plain concrete fatigue specimen exhibited a very small decrease in stiffness over approximately the first 100,000 cycles. Then there is an abrupt loss in stiffness of almost 40% of the initial stiffness. In this circumstance, and for other sudden reductions in stiffness shown on the stiffness vs. cycles graphs, the sudden drops in stiffness are due to the formation of new shear cracks in the specimens.

Considering the relatively extreme loading on the test specimens for this investigation, the fatigue stiffness degradation was relatively minor. An in service interstate bridge could expect to see $1.0 \times 10^7$ loading cycles during it’s expected life span but at half of the load levels as tested here. There were no detectable signs that the reinforcement bars suffered degradation during these experiments. The greater portion of the degradation was obviously in the concrete. Large crack widths and shear action on an extremely reduced compression section were the main contributors to the degradation of these test specimens. Flexural members, adequately designed to resist the applied shear
forces and also designed with careful consideration of serviceability criteria, are expected to see a much smaller degradation of stiffness than the beams tested here.
6.7 POST-FATIGUE STATIC FLEXURAL TESTS

If the beam fatigue specimens survived the 1.2 million fatigue loading cycle, the specimens were subjected to quasi-static tests to failure in the same manner as the virgin specimens discussed at the beginning of this chapter. Below is a table of comparisons, of several factors, between beams tested to static failure without fatigue testing and those having undergone fatigue testing. See Appendix-C for complete post-fatigue static flexural test data.

**Table 6.7-1** Comparison of Data for Pre and Post-Fatigue Static Flexural Specimens

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Virgin Static Test</th>
<th>Post Fatigue Static Test</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Max. Load (lb)</td>
<td>E (lb/in)</td>
</tr>
<tr>
<td>#4 GFRP (plain)</td>
<td>17600</td>
<td>27300</td>
</tr>
<tr>
<td>#4 GFRP (0.5% fiber)</td>
<td>19400</td>
<td>44500</td>
</tr>
<tr>
<td>#8 GFRP (plain)</td>
<td>21500</td>
<td>63000</td>
</tr>
<tr>
<td>#8 GFRP (0.5% fiber)</td>
<td>19900</td>
<td>66600</td>
</tr>
<tr>
<td>#4 CFRP (plain)</td>
<td>18300</td>
<td>91300</td>
</tr>
<tr>
<td>#4 CFRP (0.5% fiber)</td>
<td>21700</td>
<td>81000</td>
</tr>
<tr>
<td>( f'_c ) (plain)</td>
<td></td>
<td>6030</td>
</tr>
<tr>
<td>( f'_c ) (0.5% fiber)</td>
<td></td>
<td>4450</td>
</tr>
</tbody>
</table>

*NOTE: Beam stiffness values were taken from the range of load vs. deflection data from approximately 25 – 75% of ultimate load.*

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Figures 6.7-1 – 6.7-10 show graphs of load vs. mid-span deflection, rebar end slip vs. mid-span deflection, and corresponding photographs of the failed specimens. Both the plain and FRC concrete are shown on the same graphs for comparison.

**Figure 6.7-1**  Post-Fatigue Static Load vs. Mid-Span Deflection Response of Beams Reinforced with One (1) #8 GFRP Rebar
**Figure 6.7-2**  Post-Fatigue Static Rebar End-Slip vs. Mid-Span Deflection Response of Beams Reinforced with One (1) #8 GFRP Rebar

**Figure 6.7-3**  Photograph of Failed Post-Fatigue Static Bending Specimen with One (1) #8 GFRP Rebar and 0.5% Fiber Fraction
Figure 6.7-4  Post-Fatigue Static Load vs. Mid-Span Deflection Response of Beams Reinforced with Two (2) #4 GFRP Rebar

Figure 6.7-5  Photograph of Failed Post-Fatigue Static Bending Specimen with Two #4GFRP Rebar and No Fibers
Figure 6.7-6  Post-Fatigue Static Rebar End-Slip vs. Mid-Span Deflection Response of Beams Reinforced with Two (2) #4 GFRP Rebar

Figure 6.7-7  Photograph of Failed Post-Fatigue Static Bending Specimen with Two) #4 GFRP Rebar and 0.5% Fiber Fraction
Figure 6.7-8  Post-Fatigue Static Load vs. Mid-Span Deflection Response of Beams Reinforced with Two (2) #4 CFRP Rebar
Figure 6.7-9  Photographs of Failed Post-Fatigue Static Bending Specimen with Two (2) #4 CFRP Rebar and No Fibers
Figure 6.7-10  Post-Fatigue Static Rebar End-Slip vs. Mid-Span Deflection Response of Beams Reinforced with Two (2) #4 CFRP Rebar

Figure 6.7-11  Photograph of Failed Post-Fatigue Static Bending Specimen with Two (2) #4 GFRP Rebar and 0.5% Fiber Fraction
The peak strength of the beams, post-fatigue, was generally higher than the pre-fatigue peak strength. Two exceptions are the #4 GFRP FRC beams and the #4 CFRP FRC beams.

The #4 GFRP FRC beams failed during the fatigue testing program. Since concrete shear capacity is proportionally related to concrete compressive strength, it is possible that the plain concrete #4 GFRP beams did not fail in shear because they had stronger concrete (approximately 25% stronger) than the FRC beams.

The #4 CFRP FRC beams had a lower post-fatigue peak strength but did not fail during the fatigue cycles. Again, it seems likely that brittle shear failure was at least a partial contributor to this phenomenon. The propagation of cracks during the fatigue cycles during fatigue again works to limit viable, shear-resisting concrete and a brittle post-fatigue static failure was experienced.

Generally the peak strength of FRP reinforced beams did not decrease because of flexural fatigue. Preliminary failures and decreases in peak strength seem to be more due to deep beam shear induced factors.

Except in the case of the preliminary failure of the #4 GFRP FRC beams, the static test flexural stiffness was higher post-fatigue than in the virgin specimens. It is important to note that the post-fatigue static test data does not account for any residual deformation that accumulated during the fatigue cycles. So, this relative higher stiffness does not negate the data, from the slow cycle fatigue tests, which indicates a steady loss in flexural stiffness throughout the fatigue life of the beam. The increased stiffness in the post-fatigue static tests is relative to a new baseline zero deflection and therefore only a relative increase in stiffness and not a global one.
Other than the already noted exceptions, the behavior of the beams pre and post-fatigue are quite similar. If the beams are limited to an appropriate span to depth ratio, it would ensure flexural rather than shear behavior, it seems likely that the early failures experienced would be eliminated in this situation.
6.8 SHEAR STRENGTH

The shear strength of the test specimens will here be compared to the predictive shear strength values provided by several code governing agencies and also to a method of predicting shear capacities formulated by El-Shakra (1995). Since the failure mechanism was predominantly shear failure, the shear capacity of the specimens is taken as half of the applied load at ultimate strength.

Table 6.8-1  Maximum Shear Capacities of Experimental Test Specimens

<table>
<thead>
<tr>
<th>Specimens</th>
<th>ρ/ρ_{bal}</th>
<th>Virgin Static Test</th>
<th>Post Fatigue Static Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>#4 GFRP (plain)</td>
<td>1.61</td>
<td>17600</td>
<td>8800</td>
</tr>
<tr>
<td>#4 GFRP (0.5% fiber)</td>
<td>1.61</td>
<td>19400</td>
<td>9700</td>
</tr>
<tr>
<td>#8 GFRP (plain)</td>
<td>2.99</td>
<td>21500</td>
<td>10800</td>
</tr>
<tr>
<td>#8 GFRP (0.5% fiber)</td>
<td>2.99</td>
<td>19900</td>
<td>10000</td>
</tr>
<tr>
<td>#4 CFRP (plain)</td>
<td>1.96</td>
<td>18300</td>
<td>9100</td>
</tr>
<tr>
<td>#4 CFRP (0.5% fiber)</td>
<td>1.96</td>
<td>21700</td>
<td>10900</td>
</tr>
<tr>
<td>f’c (plain)</td>
<td></td>
<td>6030 psi</td>
<td></td>
</tr>
<tr>
<td>f’c (0.5% fiber)</td>
<td></td>
<td>4450 psi</td>
<td></td>
</tr>
</tbody>
</table>

* - These specimens failed during fatigue testing.

ACI Method

According to the ACI 440 Document (ACI 2001), the shear capacity of an FRP reinforced concrete beam can be predicted by the following equation...

\[
V_c = \frac{\rho_f E_f}{90 \beta_1 f'c} \left( \sqrt{\frac{f'c b_w d}{6}} \right) \leq \sqrt{\frac{f'c b_w d}{6}} \]  

{Eqn. 6.7-1}
where…

\[ \rho_f = \text{reinforcement ratio of the FRP} \]

\[ E_f = \text{Modulus of elasticity of FRP} \]

\[ \beta_1 = \text{Equivalent stress block depth modification factor} \]

\[ f'_c = 28 \text{ day compressive strength} \]

\[ b_w = \text{beam width} \]

\[ d = \text{depth to centroid of reinforcement} \]

**Canadian Method**

According to the Canadian Standard (2002) the nominal shear capacity of an FRP reinforced concrete beam can be predicted by the following equation…

\[
V_c = 0.035 \lambda \phi_c \left( f'_c \rho_f E_f \frac{V_c}{M_f} \right)^{1/3} b_w d
\]

{Eqn. 6.7-2}

where…

\[
0.1 \lambda \phi_c \sqrt{f' c b_w d} \leq V_c \leq 0.2 \lambda \phi_c \sqrt{f' c b_w d}
\]

\[ \lambda = \begin{cases} 1.0 & \text{for normal weight concrete} \\ 0.85 & \text{for semi-light weight concrete} \\ 0.75 & \text{for light weight concrete} \end{cases} \]

\[ \phi_c = \text{concrete resistance factor} \]

\[ (= 1.0 \text{ for purposes of predicting nominal strength}) \]

\[
\frac{V_f}{M_f} d = \frac{d}{a}
\]

{Eqn. 6.7-3}

After simplification, this leaves…
JSCE Method

The JSCE (1997) authority states that the nominal shear capacity of an FRP reinforced concrete beam can be predicted as follows...

\[ V_c = 0.035 \left( f'_{cd} \rho_f E_f \frac{d}{a} \right)^{\frac{1}{3}} b_n d \]  
\{Eqn. 6.7-4\}

\[ V_c = \beta_d \beta_p \beta_n f_{vud} b_n d \]  
\{Eqn. 6.7-5\}

where...

\[ \beta_d = \left( \frac{1000}{d} \right)^{\frac{1}{4}} \leq 1.5 \]  
\{Eqn. 6.7-6\}

\[ \beta_p = 3 \sqrt{\frac{100 \rho_f E_f}{E_s}} \leq 1.5 \]  
\{Eqn. 6.7-7\}

\[ \beta_n = 1.0 \quad \text{(for sections without axial force resultant)} \]

\[ f_{vud} = 0.2 f'_{cd}^{\frac{1}{3}} \leq 0.72 \text{ MPa} \]  
\{Eqn. 6.7-8\}

\[ f'_{cd} = \text{design compressive strength} \]

El-Shakra Method

El-Shakra (1995) proposed that the nominal shear capacity of an FRC concrete beam can be predicted as follows...

\[ V_c = v_u (b)(d) \]  
\{Eqn. 6.7-9\}

Where...

\[ (NOTE: \quad \text{El-Shakra calibrated the empirical coefficients for this equation using conventional steel reinforcement and steel fibers.} \]

\[ \text{The equation is used here without modification.}) \]

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\[
v_u = 0.023 f'c \left( \frac{d}{a} \right) j_o^2 + 1.18 \sqrt{f'c} \left( \frac{d}{a} \right)^{\frac{3}{2}} j_o + 5.17 V_f \frac{1}{\phi} \sqrt{f'c} \left( \frac{d}{a} \right)^{\frac{3}{2}} j_o + 69.79 \rho \sqrt{f'c} \left( \frac{d}{a} \right)^{\frac{3}{2}} j_o + 20.85 \sqrt{\rho f'c} \left( \frac{d}{a} \right)^{\frac{3}{2}} j_o
\]

\{Eqn. 6.7-10\}

\( f'c \) = 28 day concrete compressive strength (psi)

\( d \) = depth of reinforcement (in)

\( a \) = shear span (in)

\( j_o \) = internal moment arm (in)

\( V_f \) = fiber volume fraction

\( 1/\phi \) = fiber aspect ratio

\( \rho \) = reinforcement ratio

Table 6.8-2 shows a comparison of actual experimental shear capacities with those predicted by the above described methods.
Table 6.8-2  Comparison of Predicted Shear Capacities with Experimental Shear Capacities

<table>
<thead>
<tr>
<th>Beam Type</th>
<th>28 Day Compressive Strength (f'c)</th>
<th>Reinf. Ratio (ρ)</th>
<th>Method</th>
<th>Predicted Shear Capacity (lb)</th>
<th>Experimental Shear Capacity (lb)</th>
<th>Percent Deviation</th>
</tr>
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<tbody>
<tr>
<td></td>
<td>(psi) (MPa)</td>
<td></td>
<td>(lb)</td>
<td>(N)</td>
<td>(lb) (N)</td>
<td></td>
</tr>
<tr>
<td>Plain (#4 GFRP)</td>
<td>6030 41.58 .0094</td>
<td>ACI</td>
<td>1023 4550</td>
<td>8800 39100</td>
<td>-88%</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Canadian</td>
<td>4519 20104</td>
<td>-49%</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>JSCE</td>
<td>4173 18563</td>
<td>-52%</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>El-Shakra</td>
<td>7633 33953</td>
<td>-13%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.5% Fiber (#4 GFRP)</td>
<td>4450 30.68 .0094</td>
<td>ACI</td>
<td>1076 4796</td>
<td>9700 43300</td>
<td>-89%</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Canadian</td>
<td>4084 18167</td>
<td>-58%</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>JSCE</td>
<td>3771 16774</td>
<td>-65%</td>
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<tr>
<td></td>
<td></td>
<td>El-Shakra</td>
<td>9504 42276</td>
<td>-2%</td>
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<tr>
<td>Plain (#8 GFRP)</td>
<td>6030 41.58 .0174</td>
<td>ACI</td>
<td>1711 7610</td>
<td>10800 47800</td>
<td>-84%</td>
<td></td>
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<tr>
<td></td>
<td></td>
<td>Canadian</td>
<td>5549 24684</td>
<td>-48%</td>
<td></td>
<td></td>
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<tr>
<td></td>
<td></td>
<td>JSCE</td>
<td>5124 22792</td>
<td>-52%</td>
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<td>El-Shakra</td>
<td>8916 39660</td>
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<tr>
<td>0.5% Fiber (#8 GFRP)</td>
<td>4450 30.68 .0174</td>
<td>ACI</td>
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<td>10000 44300</td>
<td>-80%</td>
<td></td>
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<td></td>
<td></td>
<td>Canadian</td>
<td>5014 22305</td>
<td>-50%</td>
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<tr>
<td></td>
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<td>JSCE</td>
<td>4630 20595</td>
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<tr>
<td></td>
<td></td>
<td>El-Shakra</td>
<td>10768 47898</td>
<td>+8%</td>
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<tr>
<td>Plain (#4 CFRP)</td>
<td>6030 41.58 .007</td>
<td>ACI</td>
<td>688 3061</td>
<td>9100 40700</td>
<td>-92%</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Canadian</td>
<td>4096 18222</td>
<td>-55%</td>
<td></td>
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</tr>
<tr>
<td></td>
<td></td>
<td>JSCE</td>
<td>3782 16825</td>
<td>-57%</td>
<td></td>
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<tr>
<td></td>
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<td>El-Shakra</td>
<td>7022 31235</td>
<td>-23%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.5% Fiber (#4 CFRP)</td>
<td>4450 30.68 .007</td>
<td>ACI</td>
<td>801 3564</td>
<td>10900 48300</td>
<td>-93%</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Canadian</td>
<td>3702 16466</td>
<td>-66%</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>JSCE</td>
<td>3418 15204</td>
<td>-69%</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>El-Shakra</td>
<td>8866 39438</td>
<td>-18%</td>
<td></td>
<td></td>
</tr>
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</table>

All the predictive values provided by the governing code agencies are significantly below the experimental values. Since the beam specimens in this testing regimen are considered to be “deep” beams, meaning they have a span to depth ratio of...
less than or equal to 2.5, the prescribed shear equations, given by the respective code authorities, are of limited value. These types of beams will begin to undergo an arching type resistance mechanism and will behave more like a strut and tie than a beam resisting applied loads through typical flexure shear mechanisms. Since the arching type of resistance relies more heavily on the compressive strength of the concrete than the shear strength, the shear capacity of these deep beams is greater than slender beams. The El-Shakra method accounts for the arching action and also the contribution to the shear resistance provided by fiber reinforcement. As can be seen in Table 6.8-2, the El-Shakra method predicted the actual experimental shear capacities of this investigation’s specimens more closely than any of the governing code agency predictions. The ACI method, which does not account for the shear span to depth ratio in any fashion, was by far the furthest from predicting the actual shear capacity of the beams. The Canadian and JSCE methods, which do account for the shear span, are much closer to actual values but still significantly less than experimental values. Much of the current research literature states, conventional shear equations are not able to accurately predict the shear capacity of deep beams. Many have suggested the development of some sort of strut and tie model to predict the shear behavior of deep beams. The relative closeness, of the El-Shakra method to the experimental results, demonstrates a degree of promise for predicting both the shear capacities of deep beams and estimating the effects of fiber reinforcement on the shear capacity of FRP reinforced beams. It is likely that the ability of this method, to predict shear capacity of FRC beams reinforced with FRP bars, could be improved even further as the body of test data increases and the method is customized to FRP reinforced beams.
CHAPTER 7 - CONCLUSION

7.1 FRP TENSION TESTS

Tension tests performed on the rebar specimens in the laboratory at the University of Missouri – Columbia campus provided data showing a linear elastic stress-strain behavior to ultimate failure at which point the failure was sudden and brittle. The laboratory uniaxial tension test results for tensile strength and modulus were all less than the manufacturer’s listed minimum values except that the tensile modulus of the #4 CFRP specimens was 6.1% higher than the manufacturers listed value. The tensile strength of the tensile test specimens, was as follows with the manufacturer’s published values in parenthesis: #4 CFRP had an ultimate stress of 286,200 psi (300,000 psi) with an elastic modulus = 19.1x10⁶ psi (18.0x10⁶ psi); #4 GFRP had an ultimate stress of 79,800 psi (100,000 psi) with an elastic modulus = 4.23x10⁶ psi (5.92x10⁶ psi); #8 GFRP had an ultimate stress of 65,700 psi (80,000 psi) with an elastic modulus = 4.92x10⁶ psi (5.92x10⁶ psi).

7.2 CONCRETE COMPRESSION TESTS

Uniaxial compression tests were performed on lab prepared concrete cylinders containing from 0.0 – 1.0 % fibrillated polypropylene fibers by volume. They were cured at 100% humidity for a minimum of 28 days and tested using both closed loop and open loop testing apparatus to failure. The results of the compression tests were then compared to predictions for post-peak behavior of fiber reinforced concrete specimens obtained from the “Confinement Analogy” model developed by Gopalaratnam et al. (2005) & El-Shakra (1995). The post peak behavior of the concrete compression
specimens most closely matched an equivalent confinement pressure of approximately 50 psi. The actual test data showed concrete strengths approximately 5 to 10% lower than the “Confinement Analogy” prediction immediately post-peak. At a strain of about 0.005, the 0.5% fiber concrete matched the “Confinement Analogy” prediction, but by 0.0085 strain, the fiber concrete strength was almost double the predicted value. Mixing problems that caused high air content, in the fiber reinforced mixes, may have been a contributor to the reduced post peak strength.

7.3 **PRE-FATIGUE STATIC FLEXURE TESTS**

Four-point static bending tests were performed on two (2) of each type of beam specimen. There were six (6) types of beam specimens. The types were defined by the type of rebar, #4 GFRP, #8 GFRP, and #4 CFRP, and type of concrete, plain or FRC.

Brittle tensile shear with bond splitting failure was the mechanism of failure for the plain concrete beams with #8 GFRP and #4 CFRP. The plain concrete beams reinforced with #4 GFRP experienced a much more ductile failure through the mechanism of concrete crushing, due to shear compression failure, as is desired by the ACI 440 document. The rebar development and cover in these beams was sufficient to prevent bond splitting so they were able to progress to failure by concrete crushing.

The FRC beams behaved similarly to the plain concrete beams in the pre-peak region. The onset of failure was produced by the same mechanisms as in the plain beams, but the FRC beams retained significant strength capacity in the post-peak regime unlike the plain concrete beams. The fibers prevented complete loss of strength and facilitated a more ductile failure. In the case of the #4 CFRP and #8 GFRP FRC beams, the peak strength was higher than the similar beams with plain concrete even with weaker
concrete. At the point where the plain concrete beams proceeded to failure, the fibers in the FRC were able to allow even more strength gain before the beams progressed into the post-peak region. The #4 GFRP beams in both the plain and FRC beams had similar peak strengths but the FRC specimens were significantly more ductile.

7.4 **Flexural Fatigue Tests**

All types of the test beams were subjected to a 1.2 million cycle fatigue test at approximately 60% of ultimate bending strength. It was noted in the discussion that these 60% loads could be up to double the design load for a typical concrete member reinforced with FRP that is designed to meet serviceability criteria. During the fatigue tests, the degradation of flexural stiffness was monitored and recorded. Without exception, the FRC beams exhibited a higher rate of flexural stiffness degradation during the fatigue tests than the plain concrete beams. The #8 GFRP FRC beams had approximately 7% greater stiffness loss than the #8 GFRP plain beams. The #4 GFRP FRC beams failed at approximately 300,000 cycles. The #4 CFRP FRC beams experienced approximately 15% greater stiffness loss than the #4 CFRP plain beams. This was likely due to the weaker concrete in the FRC beams given the significantly higher air content in the FRC mixes.

7.5 **Post-Fatigue Static Flexure Tests**

After 1.2 million fatigue test cycles, the beams were subjected to a post-fatigue static flexural test. The beams with #8 GFRP and #4 CFRP exhibited peak strengths similar to or greater than those of the virgin static specimens. The flexural stiffness of the beams seemed to be higher in the post fatigue static tests (not incorporating the residual deflection incurred during the fatigue testing). The plain beams with #4 GFRP
also exhibited the same retention of peak strength as the other beams, but the FRC beams with #4 GFRP did not survive the fatigue test cycles. Again, due to the high air content and lower compressive strength, the #4 GFRP FRC beams were not able to perform as well as their plain concrete counterparts.

7.6 **Shear Strength**

The shear strength of the test beams was compared the predictive values prescribed by ACI (2001), JSCE (1997), the Canadian Standards Association (2002) and a method proposed by El-Shakra (1995). None of the code methods were able to accurately predict the shear strength of the test specimens. The El-Shakra method was reasonably close to the experimental values. The ACI method underestimated the shear capacity of the test beams by 80% to 93%. The Canadian method underestimated the shear capacity of the test beams by 48% to 66%. The JSCE method underestimated the shear capacity of the test beams by 52% to 69%. The El-Shakra method predictions of shear capacity ranged from 8% overestimation to 23% underestimation.

7.7 **Recommendations for Further Study**

Further research is necessary to provide accurate prediction of the shear strength of deep beams. The statistical scatter, given the small sample size in this investigation, does not allow conclusive predictions of shear failure in FRP reinforced FRC beams. Development of a good analytical / empirical equation, able to accurately predict the shear strength of deep beams, could be helpful in providing designers the ability to economize their designs when their loading situations guarantee deep beam behavior. Currently the code equations are very conservative for deep concrete beams reinforced
with FRP. The El-Shakra (1997) method shows promise for providing accurate
prediction of shear capacity of reinforced concrete beams.

Investigation of the literature for shear behavior of concrete beams reinforced
with FRP showed that for normal strength concrete the shear strength seemed to be
predicted well using a linear relationship of the cube root of the concrete strength. The
very limited test data of shear tests using high strength concrete beams may suggest that
this cube root correspondence may not be suitable for use with high strength concrete
beams. It may produce non-conservative predictions. The beams examined by Razaqpur
et al. (2004) varied in concrete strength from $f'_c = 3,495$ psi (24.1 MPa) to $f'_c = 5,874$ psi
(40.5 MPa) except for the two beams from the Michaluk et al. (1998) study which had an
$f'_c = 9,572$ psi (66.0 MPa). The only beams for which the JSCE and Canadian methods
overestimated the shear capacity were the two high strength beams. An experimental
program that could contrast normal strength vs. high strength concrete in shear dominant
flexure, may be able to provide a clearer idea of the applicability of the code equations
for shear to high strength concrete.
Bibliography:


American Concrete Institute Committee 440 (2001) “Guide for the Design and Construction of Concrete Reinforced with FRP Bars”. Committee Document ACI-440, 1R-01, Farmington Hills, MI

American Concrete Institute – Committee 318 (2002). Building Code Requirements for Structural Concrete and Commentary, ACI 318-02/R-02, ACI, Farmington Hills, MI, USA.


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## APPENDIX – A

*(STATIC TESTING GRAPHED DATA)*

### APPENDIX – A Contents

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<th>Page</th>
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<td>N-4C-ST-02 Graphs</td>
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<td>N-4G-T-01 Graphs</td>
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<td>F-8G-ST-01 Graphs</td>
<td>A.12</td>
</tr>
<tr>
<td>F-8G-ST-02 Graphs</td>
<td>A.13</td>
</tr>
</tbody>
</table>
N-4C-ST-01

Figure A-1  Static Load vs. Mid-Span Deflection Response of Beam N-4C-ST-01 Reinforced with Two (2) #4 CFRP Rebar

Figure A-2  Static Rebar End-Slip vs. Mid-Span Deflection Response of Beam N-4C-ST-01 Reinforced with Two (2) #4 CFRP Rebar
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Figure A-3  Static Load vs. Mid-Span Deflection Response of Beam N-4C-ST-02
Reinforced with Two (2) #4 CFRP Rebar

Figure A-4  Static Rebar End-Slip vs. Mid-Span Deflection Response of Beam N-4C-ST-02
Reinforced with Two (2) #4 CFRP Rebar
Figure A-5  Static Load vs. Mid-Span Deflection Response of Beam N-4G-ST-01 Reinforced with Two (2) #4 GFRP Rebar

Figure A-6  Static Rebar End-Slip vs. Mid-Span Deflection Response of Beam N-4G-ST-01 Reinforced with Two (2) #4 GFRP Rebar
**N-4G-ST-02**

**Figure A-7**  Static Load vs. Mid-Span Deflection Response of Beam N-4G-ST-02 Reinforced with Two (2) #4 GFRP Rebar

**Figure A-8**  Static Rebar End-Slip vs. Mid-Span Deflection Response of Beam N-4G-ST-02 Reinforced with Two (2) #4 GFRP Rebar
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(STATIC TESTING GRAPHED DATA)

N-8G-ST-01

**Figure A-9** Static Load vs. Mid-Span Deflection Response of Beam N-8G-ST-01 Reinforced with One (1) #8 GFRP Rebar

**Figure A-10** Static Rebar End-Slip vs. Mid-Span Deflection Response of Beam N-8G-ST-01 Reinforced with One (1) #8 GFRP Rebar
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*(STATIC TESTING GRAPHED DATA)*

**N-8G-ST-02**

**Figure A-11** Static Load vs. Mid-Span Deflection Response of Beam N-8G-ST-02 Reinforced with One (1) #8 GFRP Rebar

**Figure A-12** Static Rebar End-Slip vs. Mid-Span Deflection Response of Beam N-8G-ST-02 Reinforced with One (1) #8 GFRP Rebar

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(STATIC TESTING GRAPHED DATA)

**F-4C-ST-01**

Figure A-13  Static Load vs. Mid-Span Deflection Response of Beam F-4C-ST-01
Reinforced with Two (2) #4 CFRP Rebar

Figure A-14  Static Rebar End-Slip vs. Mid-Span Deflection Response of Beam F-4C-ST-01
Reinforced with Two (2) #4 CFRP Rebar
**F-4C-ST-02**

**Figure A-15** Static Load vs. Mid-Span Deflection Response of Beam F-4C-ST-02 Reinforced with Two (2) #4 CFRP Rebar

**Figure A-16** Static Rebar End-Slip vs. Mid-Span Deflection Response of Beam F-4C-ST-02 Reinforced with Two (2) #4 CFRP Rebar


**F-4G-ST-01**

![Graph](image)

**Figure A-17** Static Load vs. Mid-Span Deflection Response of Beam F-4G-ST-01 Reinforced with Two (2) #4 GFRP Rebar

![Graph](image)

**Figure A-18** Static Rebar End-Slip vs. Mid-Span Deflection Response of Beam F-4G-ST-01 Reinforced with Two (2) #4 GFRP Rebar
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(STATIC TESTING GRAPHED DATA)

**F-4G-ST-02**

![Graph of Load vs. Mid-Span Deflection](image)

**Figure A-19**  Static Load vs. Mid-Span Deflection Response of Beam F-4G-ST-02 Reinforced with Two (2) #4 GFRP Rebar

![Graph of End Slip vs. Mid-Span Deflection](image)

**Figure A-20**  Static Rebar End-Slip vs. Mid-Span Deflection Response of Beam F-4G-ST-02 Reinforced with Two (2) #4 GFRP Rebar
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*(STATIC TESTING GRAPHED DATA)*

**F-8G-ST-01**

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Figure A-22  Static Rebar End-Slip vs. Mid-Span Deflection Response of Beam F-8G-ST-01 Reinforced with One (1) #8 GFRP Rebar
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Figure A-23  Static Load vs. Mid-Span Deflection Response of Beam F-8G-ST-02 Reinforced with One (1) #8 GFRP Rebar

Figure A-24  Static Rebar End-Slip vs. Mid-Span Deflection Response of Beam F-8G-ST-02 Reinforced with One (1) #8 GFRP Rebar
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(FATIGUE TESTING GRAPHED DATA)

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Fast Cycle Fatigue Graphs (data recorded during fast cycle loading)

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[Reinforced with Two (2) #4 CFRP Rebar]

Figure B-2  Deflection vs. Cycles of Beam N-4C-LF-01  
[Reinforced with Two (2) #4 CFRP Rebar]
**Figure B-3**  Stiffness vs. Cycles of Beam N-4C-LF-01  
[Reinforced with Two (2) #4 CFRP Rebar]

**Figure B-4**  End Slip vs. Cycles of Beam N-4C-LF-01  
[Reinforced with Two (2) #4 CFRP Rebar]
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(FATIGUE TESTING GRAPHED DATA)

**Slow Cycle Fatigue Graph**  
*(slow cycle data taken every 12,000 cycles)*

![Graph showing stiffness vs. cycles for Beam N-4C-LF-01 with 2 #4 CFRP Rebar](image)

**Figure B-5**  
Stiffness vs. Cycles of Beam N-4C-LF-01  
[Reinforced with Two (2) #4 CFRP Rebar]
N-4C-LF-02

*Fast Cycle Fatigue Graphs* (data recorded during fast cycle loading)

![Load vs. Cycles Graph](image1)

**Figure B-6** Load vs. Cycles of Beam N-4C-LF-02
[Reinforced with Two (2) #4 CFRP Rebar]

![Deflection vs. Cycles Graph](image2)

**Figure B-7** Deflection vs. Cycles of Beam N-4C-LF-02
[Reinforced with Two (2) #4 CFRP Rebar]
**APPENDIX – B**

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**Figure B-8**  
Stiffness vs. Cycles of Beam N-4C-LF-02  
[Reinforced with Two (2) #4 CFRP Rebar]

**Figure B-9**  
End Slip vs. Cycles of Beam N-4C-LF-02  
[Reinforced with Two (2) #4 CFRP Rebar]
Slow Cycle Fatigue Graph \textit{(slow cycle data taken every 12,000 cycles)}

\begin{figure}
\centering
\includegraphics[width=\textwidth]{figureB10.png}
\caption{Stiffness vs. Cycles of Beam N-4C-LF-02 [Reinforced with Two (2) #4 CFRP Rebar]}
\end{figure}
**APPENDIX – B**
(FATIGUE TESTING GRAPHED DATA)

**N-4G-LF-01**

*Fast Cycle Fatigue Graphs* (data recorded during fast cycle loading)

![Graph of Load vs. Cycles](image1)

**Figure B-11** Load vs. Cycles of Beam N-4G-LF-01
[Reinforced with Two (2) #4 GFRP Rebar]

![Graph of Deflection vs. Cycles](image2)

**Figure B-12** Deflection vs. Cycles of Beam N-4G-LF-01
[Reinforced with Two (2) #4 GFRP Rebar]
Figure B-13  Stiffness vs. Cycles of Beam N-4G-LF-01  
[Reinforced with Two (2) #4 GFRP Rebar]

Figure B-14  End Slip vs. Cycles of Beam N-4G-LF-01  
[Reinforced with Two (2) #4 GFRP Rebar]
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(FATIGUE TESTING GRAPHS OF DATA)

**Slow Cycle Fatigue Graph** *(slow cycle data taken every 12,000 cycles)*

![Stiffness vs. Cycles of Beam N-4G-LF-01](image)

*Figure B-15*  Stiffness vs. Cycles of Beam N-4G-LF-01  
[Reinforced with Two (2) #4 GFRP Rebar]
**N-4G-LF-02**

*Fast Cycle Fatigue Graphs* *(data recorded during fast cycle loading)*

![Load vs. Cycles Graph](#)

*Figure B-16*  Load vs. Cycles of Beam N-4G-LF-02
[Reinforced with Two (2) #4 GFRP Rebar]

![Deflection vs. Cycles Graph](#)

*Figure B-17*  Deflection vs. Cycles of Beam N-4G-LF-02
[Reinforced with Two (2) #4 GFRP Rebar]
Figure B-18  Stiffness vs. Cycles of Beam N-4G-LF-02  
[Reinforced with Two (2) #4 GFRP Rebar]

Figure B-19  End Slip vs. Cycles of Beam N-4G-LF-02  
[Reinforced with Two (2) #4 GFRP Rebar]
**Slow Cycle Fatigue Graph**  (*slow cycle data taken every 12,000 cycles*)

![Diagram of Stiffness vs. Cycles of Beam N-4G-LF-02](image)

**Figure B-20**  Stiffness vs. Cycles of Beam N-4G-LF-02  
[Reinforced with Two (2) #4 GFRP Rebar]
N-8G-LF-01

Fast Cycle Fatigue Graphs (data recorded during fast cycle loading)

Figure B-21 Load vs. Cycles of Beam N-8G-LF-01
[Reinforced with One (1) #8 GFRP Rebar]

Figure B-22 Deflection vs. Cycles of Beam N-8G-LF-01
[Reinforced with One (1) #8 GFRP Rebar]
Figure B-23  Stiffness vs. Cycles of Beam N-8G-LF-01  
[Reinforced with One (1) #8 GFRP Rebar]

Figure B-24  End Slip vs. Cycles of Beam N-8G-LF-01  
[Reinforced with One (1) #8 GFRP Rebar]
**Slow Cycle Fatigue Graph** (slow cycle data taken every 12,000 cycles)

*Figure B-25*  Stiffness vs. Cycles of Beam N-8G-LF-01
[Reinforced with One (1) #8 GFRP Rebar]
**F-4C-LF-01**

*Fast Cycle Fatigue Graphs* (data recorded during fast cycle loading)

*Figure B-26*  Load vs. Cycles of Beam F-4C-LF-01  
[Reinforced with Two (2) #4 CFRP Rebar]

*Figure B-27*  Deflection vs. Cycles of Beam F-4C-LF-01  
[Reinforced with Two (2) #4 CFRP Rebar]
**Figure B-28**  Stiffness vs. Cycles of Beam F-4C-LF-01  
[Reinforced with Two (2) #4 CFRP Rebar]

**Figure B-29**  End Slip vs. Cycles of Beam F-4C-LF-01  
[Reinforced with Two (2) #4 CFRP Rebar]
**Slow Cycle Fatigue Graph** *(slow cycle data taken every 12,000 cycles)*

![Stiffness vs. Cycles of Beam F-4C-LF-01](image_url)

**Figure B-30**  Stiffness vs. Cycles of Beam F-4C-LF-01  
[Reinforced with Two (2) #4 CFRP Rebar]
F-4C-LF-02

Fast Cycle Fatigue Graphs (data recorded during fast cycle loading)

Figure B-31 Load vs. Cycles of Beam F-4C-LF-02
[Reinforced with Two (2) #4 CFRP Rebar]

Figure B-32 Deflection vs. Cycles of Beam F-4C-LF-02
[Reinforced with Two (2) #4 CFRP Rebar]
Figure B-33  Stiffness vs. Cycles of Beam F-4C-LF-02  
[Reinforced with Two (2) #4 CFRP Rebar]

Slow Cycle Fatigue Graph  (slow cycle data taken every 12,000 cycles)
Figure B-35  Stiffness vs. Cycles of Beam F-4C-LF-02  
[Reinforced with Two (2) #4 CFRP Rebar]
**APPENDIX – B**
*(FATIGUE TESTING GRAPHED DATA)*

**F-4G-LF-01**

*Fast Cycle Fatigue Graphs (data recorded during fast cycle loading)*

![Graph of Load vs. Cycles of Beam F-4G-LF-01](image)

**Figure B-36**  Load vs. Cycles of Beam F-4G-LF-01  
[Reinforced with Two (2) #4 GFRP Rebar]

![Graph of Deflection vs. Cycles of Beam F-4G-LF-01](image)

**Figure B-37**  Deflection vs. Cycles of Beam F-4G-LF-01  
[Reinforced with Two (2) #4 GFRP Rebar]
Figure B-38  Stiffness vs. Cycles of Beam F-4G-LF-01  
[Reinforced with Two (2) #4 GFRP Rebar]

Figure B-39  End Slip vs. Cycles of Beam F-4G-LF-01  
[Reinforced with Two (2) #4 GFRP Rebar]
**Slow Cycle Fatigue Graph**  
(*slow cycle data taken every 12,000 cycles*)

![Graph showing stiffness vs. cycles for Beam F-4G-LF-01 reinforced with two #4 GFRP Rebar.](image)

**Figure B-40**  
Stiffness vs. Cycles of Beam F-4G-LF-01  
[Reinforced with Two (2) #4 GFRP Rebar]
**APPENDIX – B**

(FATIGUE TESTING GRAPHED DATA)

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**F-4G-LF-02**

*Fast Cycle Fatigue Graphs* (data recorded during fast cycle loading)

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**Figure B-41** Load vs. Cycles of Beam F-4G-LF-02  
[Reinforced with Two (2) #4 GFRP Rebar]

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**Figure B-42** Deflection vs. Cycles of Beam F-4G-LF-02  
[Reinforced with Two (2) #4 GFRP Rebar]
**Figure B-43**  Stiffness vs. Cycles of Beam F-4G-LF-02  
[Reinforced with Two (2) #4 GFRP Rebar]

**Figure B-44**  End Slip vs. Cycles of Beam F-4G-LF-02  
[Reinforced with Two (2) #4 GFRP Rebar]
Slow Cycle Fatigue Graph  *(slow cycle data taken every 12,000 cycles)*

![Graph showing stiffness vs. cycles for Beam F-4G-LF-02 reinforced with Two (2) #4 GFRP Rebar.](graph)

**Figure B-45**  Stiffness vs. Cycles of Beam F-4G-LF-02  
[Reinforced with Two (2) #4 GFRP Rebar]


**F-8G-LF-01**

*Fast Cycle Fatigue Graphs* (data recorded during fast cycle loading)

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**Figure B-46**  Load vs. Cycles of Beam F-8G-LF-01  
[Reinforced with One (1) #8 GFRP Rebar]

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**Figure B-47**  Deflection vs. Cycles of Beam F-8G-LF-01  
[Reinforced with One (1) #8 GFRP Rebar]
Figure B-48  Stiffness vs. Cycles of Beam F-8G-LF-01  
[Reinforced with One (1) #8 GFRP Rebar]

Figure B-49  End Slip vs. Cycles of Beam F-8G-LF-01  
[Reinforced with One (1) #8 GFRP Rebar]
**Slow Cycle Fatigue Graph** *(slow cycle data taken every 12,000 cycles)*

![Graph](image-url)

**Figure B-50** Stiffness vs. Cycles of Beam F-8G-LF-01
[Reinforced with One (1) #8 GFRP Rebar]
APPENDIX – C
(POST-FATIGUE STATIC TESTING GRAPHED DATA)

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**N-4C-LF-01**

![Graph](image)

**Figure C-1** Static Load vs. Mid-Span Deflection Response of Beam N-4C-LF-01 Reinforced with Two (2) #4 CFRP Rebar

![Graph](image)

**Figure C-2** Static Rebar End-Slip vs. Mid-Span Deflection Response of Beam N-4C-LF-01 Reinforced with Two (2) #4 CFRP Rebar
Figure C-3  Static Load vs. Mid-Span Deflection Response of Beam N-4C-LF-02 Reinforced with Two (2) #4 CFRP Rebar

Figure C-4  Static Rebar End-Slip vs. Mid-Span Deflection Response of Beam N-4C-LF-02 Reinforced with Two (2) #4 CFRP Rebar
N-4G-LF-01

**Figure C-5** Static Load vs. Mid-Span Deflection Response of Beam N-4G-LF-01 Reinforced with Two (2) #4 GFRP Rebar

**Figure C-6** Static Rebar End-Slip vs. Mid-Span Deflection Response of Beam N-4G-LF-01 Reinforced with Two (2) #4 GFRP Rebar
**APPENDIX – C**
*(POST-FATIGUE STATIC TESTING GRAPHEd DATA)*

**N-4G-LF-02**

![Graph of Mid-Span Deflection vs. Load](image)

**Figure C-5** Static Load vs. Mid-Span Deflection Response of Beam N-4G-LF-02 Reinforced with Two (2) #4 GFRP Rebar

![Graph of End Slip vs. Mid-Span Deflection](image)

**Figure C-6** Static Rebar End-Slip vs. Mid-Span Deflection Response of Beam N-4G-LF-02 Reinforced with Two (2) #4 GFRP Rebar
Figure C-7  Static Load vs. Mid-Span Deflection Response of Beam N-8G-LF-01 Reinforced with One (1) #8 GFRP Rebar

Figure C-8  Static Rebar End-Slip vs. Mid-Span Deflection Response of Beam N-8G-LF-01 Reinforced with One (1) #8 GFRP Rebar
**F-4C-LF-01**

![Graph](image)

**Figure C-9**  Static Load vs. Mid-Span Deflection Response of Beam F-4C-LF-01 Reinforced with Two (2) #4 CFRP Rebar

![Graph](image)

**Figure C-10**  Static Rebar End-Slip vs. Mid-Span Deflection Response of Beam F-4C-LF-01 Reinforced with Two (2) #4 CFRP Rebar
**APPENDIX – C**

*(POST-FATIGUE STATIC TESTING GRAPHED DATA)*

**F-4C-LF-02**

![Graph](image)

**Figure C-11**  Static Load vs. Mid-Span Deflection Response of Beam F-4C-LF-02
Reinforced with Two (2) #4 CFRP Rebar

![Graph](image)

**Figure C-12**  Static Rebar End-Slip vs. Mid-Span Deflection Response of Beam F-4C-LF-02
Reinforced with Two (2) #4 CFRP Rebar

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Figure C-13  Static Load vs. Mid-Span Deflection Response of Beam F-8G-LF-01 Reinforced with One (1) #8 GFRP Rebar

Figure C-14  Static Rebar End-Slip vs. Mid-Span Deflection Response of Beam F-8G-LF-01 Reinforced with One (1) #8 GFRP Rebar