ABSTRACT

PIKE, CALEB ALEXANDER. A Preliminary Investigation of Fiber Reinforced Concrete under Quasi-Static and Dynamic Loads. (Under the direction of Dr. Abhinav Gupta and Dr. Vernon Matzen.)

The world has been reminded in recent years that there is no boundary that terrorists won’t cross and there are no limits to what they will do to achieve their political objectives. With the United States at the center of their focus, the list of potential targets has expanded to include any critical and/or vulnerable structure that symbolically plays a part in the American way of life, such as government, communications and financial networks, and nuclear power.

With the increased threat to structural infrastructure, research studies have been conducted on various ways to augment defensive capabilities and protect against blast and impact affects in addition to general wear and tear. Nuclear containment structures are of particularly high threat due to the level of potential collateral damage their failure could inflict. Current nuclear facilities construction involves over three feet of concrete and conventional steel reinforcement. This containment structure serves multiple functions including a physical barrier against blast and impact loading. It has been suggested that external fiber reinforced concrete panels be used for the function of blast and impact protection, with the additional benefit of reducing construction time and cost by lowering the required amount of conventional steel. The addition of unconventional reinforcement to concrete, specifically fiber reinforcement, has been shown to have the desired characteristics for blast and impact resistance including increased durability, toughness and high energy absorption.

This is a preliminary study of the fundamental behavior of fiber reinforced concrete. The ultimate goal for this research project is to develop an analytical model to simulate the behavior of fiber reinforced concrete structures under dynamic loading as well as to assist in designing fiber reinforced concrete structures. The aim of this study was to obtain a usable fiber reinforced concrete and conduct quasi-static and dynamic experiments.

This study succeeded in determining an appropriate mixing technique and batch proportions for creating a workable fiber reinforced concrete using Forta Ferro fibers. Furthermore, this project examined fiber reinforced concrete through several experiments. To better understand fiber reinforced concrete properties, test specimens were statically tested.
for compressive strength, splitting tensile strength, and most importantly average residual strength, all using ASTM Standards when applicable. The ability of fiber reinforced concrete to carry load past initial cracking is demonstrated by average residual strength. The mean value of average residual strength of 5 specimens was found to be around 350 psi.

An experimental procedure had to be developed for dynamic impact testing. A drop weight test setup was created by dropping a cylindrical mass through a PVC pipe. A load cell was connected to the falling mass while an accelerometer was attached to the beam. Although, no definite numerical results were obtained, impact resistance of fiber reinforced concrete was demonstrated by test specimens that resisted multiple impacts. Even though a crack developed, some concrete specimens continued to carry load. Most importantly, the development and calibration of the test procedure will benefit future experimentation.

While this study is only a preliminary investigation into fiber reinforced concrete, it has shown that fiber reinforcement has potential for mitigating blast and impact effects and it has laid the ground work for future work.
A Preliminary Investigation of Fiber Reinforced Concrete Under Quasi-Static and Dynamic Loads

by

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A thesis submitted to the Graduate Faculty of North Carolina State University in partial fulfillment of the requirements for the Degree of Master of Science

Civil Engineering

Raleigh, North Carolina

2009

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BIOGRAPHY

The son of a career military officer, Caleb Alexander Pike is a North Carolina native who spent most of his formative years abroad, as his father and mother served in various U.S. diplomatic missions across the African Continent. Born in Fort Bragg, NC, Caleb had never lived more than two years in one location until returning to North Carolina for higher education. In 2005, he began a Master of Science in Structural Engineering and Mechanics in the Civil, Construction, and Environmental Engineering department at North Carolina State University immediately following graduation with a Bachelor of Science in Engineering Mathematics from Elon University and a Bachelor of Science in Civil Engineering from North Carolina State University. Guided by his presence in the bombing of the U.S. Embassy in Kenya, Caleb began his research into fiber reinforcement as blast and impact protection under Dr. Abhinav Gupta and Dr. Vernon Matzen.
This research represents an important milestone on a journey begun many years ago. I did not, however, arrive here alone. There have been numerous family members, friends, and teachers who had a profound influence on me and have guided me in significant ways along my chosen path and I am very grateful for this. I would like to also recognize some people that help me move ahead and complete this project. These, along with other unnamed individuals, shared their time and effort to become my mentors, made material contributions, offered critical advice, provided technical assistance, and/or encouragement that was critical to the advancement of this particular project.

Thanks to my advisors and committee co-chairs Dr. Abhinav Gupta and Dr. Vernon Matzen for the opportunity to participate in this research project and their continued guidance, advice, and patience throughout the duration of this research project.

Thanks to Dr. Rudi Seracino for his concrete expertise and for being a member of my thesis committee.

Thanks to Jerry Atkinson for helping with development of a suitable concrete mixing procedure, assisting with various concrete test setups, and for donating his time as well as Bill Dunleavy and Jake Rhoads for helping with experimental setups.

Thanks to fellow graduate students, Kevin Wilkins for helpful contributions with experimentation and Tyler Barker for assisting with drop weight testing.

Thanks to the Idaho National Lab for providing funding for this research project as well as Jeff Lacy, Steve Novascone, and Bill Richens for their technical assistance and being fantastic tour guides.

Thanks to Mr. Cliff MacDonald and the Forta Corporation for donating the Ferro fibers used in this project and providing helpful background information on fiber reinforced concrete and impact testing.
Thanks to my many friends for getting my mind off of experimental worries, for always showing interest in my progress, and for always being just a phone call away whether in North Raleigh, Elon, or Maryland.

Thanks to Dr. Richard D’Amato for always offering support and guidance as well as his friendship.

Very special thanks to my parents, without whom I would have never leaped from this diving board, for providing endless encouragement and constantly setting me back on track.
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CHAPTER 1: INTRODUCTION

The post-9/11 environment has moved our national security interests beyond the 60-year old Cold War mentality and its associated mutual assured destruction concept of a global nuclear threat scenario. Today, there is a more pragmatic realization that groups adhering to extremist ideologies present an ever present danger to the American way of life, as these groups seek to target critical and/or vulnerable infrastructure as a means to achieve their objectives. In practice, this new reality has increased exponentially the potential target list to include heretofore excluded critical infrastructure. Terrorism is not a new concept; however, the tools terrorist employ has evolved and caused a re-evaluation of its implications, both domestically and internationally, in several different areas. Not only has the list of high value targets grown, but the list of available weapons and delivery means has expanded as well, as evidenced by the use of commercial aircraft used as projectiles.

Physical structures define the majority of potential critical infrastructure targets. At one end of a potential target spectrum are nuclear facilities, as there is ample evidence of the resultant major collateral damage that accompanies an incident to such structures and the surrounding environments. Simulations for disasters, including those implementing aircraft collisions, involving nuclear facilities are being constantly analyzed and evaluated. In addition to the reactor structures, stored spent nuclear fuels and byproducts also present a lucrative target for attack, as the resultant damage significantly impacts on recovery and consequence management. On the opposite end of the spectrum are the more symbolic attacks on softer and more exposed government, communications, and financial structures.

1.1 Motivation for Research

While unconventional concrete reinforcement has been around for many years, it has received more attention in research studies due to increased structural threats that range from
blast and impact loads to general wear and tear of structural elements. Practical applications of unconventional reinforcement range from improving shear resistance of beams and columns, increasing post-fabrication load capacity of structural elements, repairing damaged structures, reinforcing connections, and improving impact resistance of slab and wall components. Fiber reinforced polymers (FRP) and fiber reinforcement concrete (FRC) are among the unconventional reinforcements being studied. Applications of unconventional concrete reinforcement can benefit both new and existing structures. The benefit of fiber reinforced polymer is that it can be placed on steel or concrete structures allowing for the repair and retrofitting of structural components instead of removing and replacing preexisting structures following minor damage.

New construction, however, has the capability to ensure structures are at least moderately prepared to resist structural threats. The various properties of fiber reinforced concrete such as increased durability, ductility, and energy absorption are particularly appealing and have the potential to increase long term serviceability. This unconventional reinforcement focuses more on preemptively resisting damage rather than repairing or retrofitting damage areas. In today’s climate, a particularly beneficial property of fiber reinforcement is its potential to resist blast and impact effects; specifically that of high threat structures such as government buildings and nuclear facilities.

Current nuclear facility construction serves as an illustrative example. Nuclear reactor containment structures serve four main functions as shown in Figure 1-1. In addition to providing radiation containment, resisting seismic loads and internal pressures these containment walls act as physical barriers against blast and impact loading.
In order to successfully perform all of these functions with high factors of safety, current nuclear containment is fabricated with high levels of conventional steel reinforcement. The reinforcement mesh is created with #18 reinforcing bars (each #18 bar has a diameter of 2.257 in.) the bars are 8 layers deep, and there is less than 1ft of spacing between bars around the outer edge. Figure 1-2 shows an example of the steel rebar mesh that is required for nuclear containment.
This mesh presents several practical issues. Not only are the large rebars expensive, but transporting, organizing, placing, and tying the large amounts of steel reinforcing bars slows construction time. Furthermore, the dense steel hinders the flow of the concrete pour and distribution. Once constructed, these large structures need to be maintained based on high quality standards from nuclear regulatory agencies. While quality assurance is of upmost importance for nuclear facilities in general, the size of these massive containment structures makes maintenance and repair difficult and replacement impossible.

The size, reinforcement requirements, quality assurance standards, and multifunctionality of current nuclear containment create issues of construction practicality and cost efficiency. One potential solution is to transfer the blast and impact protection functionality from current nuclear containment to prefabricated external panels; specifically prefabricated fiber reinforced panels that have a high level of energy absorption and ductility. Having the blast and impact protection encased in panel sections allows for relatively easy installment, maintenance, and if necessary, replacement. Fiber reinforcement is known to control cracking and increase dynamic load resistance.
Although fiber reinforced concrete is not new to the industry, the understanding of the material as a whole is lacking. While research has been conducted on the various benefits of fiber reinforced concrete including shear resistance, ductility, and crack width mitigation, the research most relevant for this project concerns blast and impact resistance. There is consensus on some of the general benefits of the material, but gaps in the research remain.

One of the major reasons for these gaps is the absence of standards in both experimental testing and material standards. Fibers are manufactured in an array of shapes, sizes (including cross sectional area and length), colors, and materials. Figure 1-3 below demonstrates some of the options for steel fibers. Multiply the number of available shapes by the available lengths and the available materials and the list of fiber types expands quickly.

![Steel Fiber Types](image)

**Figure 1-3 Various Steel Fiber Types**

Fibers typically range from as small as a small fraction of an inch to over 2.5 inches. The length of fibers has several implications on the resulting concrete mix including bond to the matrix and weight of the mix.

Similarly, the fiber material affects matrix bond and mix weight. For example, since steel fibers typically have poor matrix bond, manufacturers use a range of techniques to improve the bond, including crimping the ends, twisting the fibers into spirals, or varying the
cross section by creating divots along the length of the fiber. It is believed that fibers of synthetic polymers tend to have better bond with the concrete because they lack the rigidity limitations of steel fibers. Moreover, not only does fiber material determine matrix bond, it also affects the weight of the concrete. Steel fibers weigh significantly more than polymer or glass fibers. The fiber length and fiber material along with the physical properties associated with the various fibers such as tensile strength, alkaline resistance, radiation resistance, and conductivity are determining factors in the selection of any practical applications.

Energy absorption and residual strength of FRC are properties needed in the application of dynamic load resistance. These properties are affected by fiber type, especially length and material. Resistance to high energy dynamic loading has been shown to be affected by fiber dosage in addition to factors such as bond strength and tensile strength which are related of fiber length, shape, and material. Fiber dosage is simply the amount of fibers added to a concrete mix. These values generally vary from 0.5% to 2.0% by volume. The high end of this range is generally considered to be the percentage where fiber benefits are maximized without over complicating mixing procedures. The specific value would depend on the type of fibers, but workability becomes an issue with high levels of fiber content.

It has been stated that, “Fibers are not added to increase concrete strength... Fiber reinforcement is not a substitute for conventional steel reinforcement”[1]. While fibers are not a replacement for conventional steel reinforcement, they do increase ductility, damage mitigation, and crack control and hence allow engineers to modify designs for new structures or retrofit existing structures. The many options for fiber material, size, shape, length and other characteristics make standardization difficult. Although existing research shows that post-cracking behavior and energy absorption benefits vary as much as the fiber types, it is evident that all fibers share these desirable properties on some level. This behavior makes them a prime candidate to be used in the protection of high threat structural targets including government, communications, and financial structures as well as nuclear facilities. In the case of nuclear facilities, the use of fiber reinforced external panel sections has advantages.
than just blast and impact protection. These panels have the potential to lower costs and construction time by reducing the amount of steel needed to meet all of the functionality that current nuclear containment structures require by moving some of that functionality to the panels themselves.

1.2 Objective

The overall objective of this study of is to develop a mathematical model of fiber reinforced concrete that can be used as an analytical tool in the design of FRC. Currently there is no design method in part because of the lack of standardization of fibers and the post-cracking characteristics of FRC. However, if a computer model of fiber reinforced concrete that has been reconciled with experimental results could be created then the model could be used as a tool for design. The model would need to produce acceptable results while allowing various fiber reinforced concrete parameters to be adjusted. These parameters would include information including fiber type, fiber tensile strength, and fiber dosage as well as details about the concrete including density, compressive and tensile strengths, and possibly mix proportions. The loadings used in this study would be static and dynamic, including blast and impact.

The first step in developing such a model is to understand how fiber reinforced concrete behaves and how the behavior is affected by the various design parameters. To do this, an array of experiments ranging from quasi-static to dynamic will need to be conducted. Furthermore, as it is a variable in fiber reinforced design, testing how different fiber dosages affect FRC behavior will also be required. The following is a list of steps needed to accomplish the objective:

1. Create a functional fiber reinforced concrete mix.
2. Develop test setups and perform quasi-static tests.
3. Develop test setups and perform impact tests.
4. Adjust fiber reinforced concrete parameters such as fiber dosage.
5. Conduct further experimentation with various fiber reinforced concrete mixes.
7. Test analytical model by comparing its ability to simulate dynamic experimental results, and modify as needed.

Although all of these steps are necessary to achieve this objective, investigating all of them is beyond the scope of the current study. This study used a single fiber type and dosage to obtain a basic understanding of fiber reinforced concrete properties and to calibrate impact experimentation. In support of the overall project, the goals for this study were to:

1. Create a functional fiber reinforced concrete mix by determining the appropriate batch proportions and formulating a reliable mixing process.
2. Conduct splitting tensile strength, compressive strength, and average residual strength tests based on ASTM standards.
3. Develop a test setup for a drop weight impact experiment.
4. Calibrate the drop weight experimental setup by determining the appropriate drop height and weight as well as implementing the data acquisition instrumentation and software.

Future experimentation can be expanded on several levels to include variations in types of fibers, fiber dosage and types of impact loading while research can be conducted on developing the model for fiber reinforced concrete. The following shows a proposed layout to fiber reinforced concrete research at North Carolina State University:

- Phase 1: Basic understanding of fiber reinforced concrete:
  - FRC batch proportioning and mixing processes.
  - Behavior under static tests such as Splitting Tensile, Compressive, and Average Residual Strength experiments.
  - Calibration of impact tests and observations of results.
• Phase 2: Further experimentation and observations of fiber reinforced concrete:
  o Adjust fiber dosage by percent volume to observe variation in behavior.
  o Possible change in fiber type to include other synthetic fibers.
  o Continued impacting testing utilizing recommendations from Phase 1 experimentation.
• Phase 3: Modeling fiber reinforced concrete behavior:
  o Utilize static test results to model Splitting Tensile Strength Test for benchmarking modeling software.
• Phase 4: Modeling fiber reinforced concrete behavior under dynamic loads:
  o Correlate with experimental results to assist in future fiber reinforced concrete design.

1.3 Scope of Research

This study explored the use and characteristics of fiber reinforced concrete at the fundamental level. The first priority was to obtain a fiber reinforced concrete mix that would be appropriate for testing. The type of fiber used for this study was Forta Ferro fibers. Forta Ferro, or ‘Strong as Steel”, are polypropylene fibers manufactured by Forta Corp. Initial explorations varied batch proportions and fiber content of trial mixes. Based on the idea that fibers can be added to any batch of concrete, a promotional selling point typically used by manufacturers, the first mixes used a fiber content of 2.0% by volume and a ready mix concrete batch from another, unrelated project. Using prefabricated mixes, however, proved to be difficult. Details about the struggles with these batches are described in Chapter 3. A final concrete mix was determined with recommended proportions from Forta Corp and had a fiber content of 0.5% by volume. The lower fiber content was to ensure better workability.

Once an appropriate mix was determined, industry standards and criteria were used, when available, to understand fiber reinforced concrete subjected to quasi-static and dynamic loads. Initially, experimental testing began on beams and cylinders under quasi-static
conditions including beam average residual strength tests and compression and split tension tests on cylinders. Once a better understanding of material properties was achieved from static experiments, energy absorption properties were investigated using a drop weight from varying heights providing data for further analysis of drop heights, weights, accelerations, impact loads, and deflections.

1.4 Thesis Organization

Chapter 2 contains a brief literature review discussing the background of fiber reinforced concrete, existing research on fiber reinforced concrete under blast and impact loads, and gaps in current research.

Chapter 3 describes in detail the experimental setup for all tests performed during this study as well the selection of fibers and formulation of trial concrete mixes while Chapter 4 reviews and discusses the experimental test results.

Finally, Chapter 5 presents recommendations for future work; covering suggested changes for experimental setups as well as how the project should proceed.

1.5 References

CHAPTER 2: LITERATURE REVIEW

2.1 Background of Fiber Reinforced Concrete

Evidence of fiber reinforcement has been traced back to the ancient Egyptian times. Pharaoh was recorded in the Bible as commanding the Egyptian slave drivers and foremen guarding the Jews to “no longer to supply the people with straw to make brick; let them go and gather their own straw.” – Exodus 5:7 [4]. Organic straw fibers were added to adobe to increase the strength and durability of the clay brick. Horsehair was also used for brittle materials in early times [2]. It was noticed as early as these times that fibers could bridge the gaps in fractured ceramic materials. Although such fibers are still in use today for some applications, organic fibers are typically limited to developing nations.

Fiber technology has come a long way since biblical times. Steel, synthetic polymer and glass fibers are all used in today’s construction industry although in limited applications, and primarily for “shrinkage or temperature reinforcement… in relatively low concentrations… primarily to improve resistance to the formation of plastic shrinkage” [6]. Fibers are now being added to concrete used in bridge and parking decks, slabs and pavements because of the ability of fibers to “improve toughness, or postelastic ductility, of concrete” [6].

ACI Committee 544 on Fiber Reinforced Concrete, organized in 1966, has the task of “[developing] and [reporting] information on concrete reinforced with short, discontinuous, randomly-dispersed fibers” [3]. ACI 544 has collaborated with researchers and industry to develop documentation on fiber reinforced concrete materials including design documents, mixing and proportioning guides, and a report on fiber reinforced concrete. While the report on the state-of-the-art of fiber reinforced concrete covers physical properties, applications, research needs and design considerations of various types of fibers, the design documents and mixing and proportion guides were developed specifically for steel reinforcement. This
stems from the history of fiber reinforcement. ACI Committee 544 Report on Fiber Reinforced Concrete states that primitive forms of steel fiber reinforcement were patented as early as the 1910s. Nails, wire, and metal chips were all added to concrete to improve performance, but it wasn’t until the 1960s that full scale investigations into fiber reinforcement began. The evaluation of steel fibers was the start of modern day fiber reinforcement in the United States; the exploration of glass and synthetic fibers would follow later. Since the 1960s, steel fiber reinforcement has had a substantial amount of “research, development, experimentation, experimentation, and industrial application”. Once alkali resistant glass fibers were developed and glass fibers could withstand the alkali in the cement, commercial products took advantage of this new fiber. The largest demand for these fibers was for architectural applications. As technology developed, synthetic materials became another source for fiber reinforcement. Glass and steel fibers took the forefront, however, when synthetic fibers failed to achieve the early success expected. Even though synthetic fibers lacked early success, increased research, improved technology, and refined manufacturing procedures have greatly expanded the use and benefits of synthetic fibers [2].

Even though applications of fiber reinforced concrete are becoming more accepted by the engineering community, the current ACI Committee 318 “Building Code Requirements for Reinforced Concrete”, does not recognize fiber reinforcement benefits for structural elements. However, increased research and experience have demonstrated the benefits of FRC and fibers are being used in some structural industry applications. Table 2-1 gives a list of properties and common applications of various fiber reinforced concrete types presented by ACI 544 [2].

<table>
<thead>
<tr>
<th>Fiber Type</th>
<th>Categories</th>
<th>Properties</th>
<th>Current Industry Applications</th>
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| Steel      | • Cold-Drawn Wire  
             • Cut Sheet  
             • Melt-Extracted:  
             • Crimped-End  
             • Flattened-End  
             • Deformed | • Improved flexural toughness  
             • Impact resistance  
             • Flexural fatigue endurance  
             • Static and Dynamic tensile strength  
             • Energy absorption | • Flat slabs on grade subject to impact loads  
             • Shotcrete applications  
             • Hardened military structures  
             • Tanker docks  
             • Airport Pavements  
             • Dams and vaults |

Table 2-1 Various fiber types, properties, and applications.
Although, applications of synthetic fiber reinforced concrete are mostly limited to slabs and flooring, it is recognized that these fibers have some impact resistance properties. The use of synthetic fibers in warehouse slabs is to absorb some of the energy of large falling objects and to control cracking if any occurs. It is not such a stretch to believe synthetic fibers might have other applications. Military researchers saw potential for fibers in high threat structures and have implemented fiber reinforced concrete in missile facilities and other hardened structures. Generally, the fibers being used for this sort of impact protection have been steel fibers. However, synthetic fibers might present an opportunity to continue to have the same impact resistant structures while reducing weight and cost.

### 2.2 Impact and Blast Loading

Impact properties of fiber reinforced concrete are not well understood. There is currently neither a standard methodology for impact testing [5] nor a standard for materials used to produce fiber reinforced concrete. “A large number of test setups [have] been used to
investigate…polypropylene FRC under impact” [2]. Testing methods range from Charpy-type tests to hammer or drop weight tests [1]. Tests used for dynamic loading of concrete are extremely sensitive to test procedures and data acquisition. Published results tend to vary substantially. “Due to the variable nature of [impact] testing and the need to apply specialized analytical techniques to each test setup, cross test comparisons cannot be made” [2].

In studies of polypropylene fiber reinforced concrete, results had a significant range of impact strength increases depending on the type of test performed. When compared to plain concrete, in a uniaxial tension mode the impact strength improvement was around 15% while, for flexural tests, the impact strength increase was closer to 50% [2]. Studies generally show that there is some increase in impact strength with the presence of fibers and typically that as the fiber content increases the impact strength increases. However, “improvement in fracture energy for polypropylene FRC was reported between 33 and 1000 percent” [2].

The University of British Columbia in Vancouver is home to one of the largest ongoing studies of impact behavior specifically targeting fiber reinforced concrete. University of British Columbia’s civil engineering professors Nemkumar Banthia and Sidney Mindess have conducted extensive research over the last 20 years on high energy behavior of fiber reinforced concrete, including several investigations on impact loading on fiber reinforced concrete containing various fiber dosages and types. The impact experiments were implemented through a drop weight testing machine developed over many years. The impact machine, shown in Figure 2-1, has the ability to drop masses up to 345 kg (760.6 lbs) from heights up to 2.3 m (7.5 ft). The machine is mounted on a solid concrete pedestal and employs a hoist and railing system [6].
In one particular study performed in British Columbia, the impact weight was 60.3 kg with a drop weight of 150 mm for all tests. Below a fiber content of 0.5%, the primary failure mechanism was fiber breakage with only modest increases in fracture energy, while fiber dosages that were “above 0.75% fiber [content,] pull-out was the primary failure mechanism with large increases of fracture energy. The study was conducted on 3 types of fibers. The polypropylene fibers used in this research were 38 mm long and produced by Forta Corp. Two types of steel fibers were used: 50 mm crimped steel fibers from Eurosteel and 30 mm hooked-end steel fibers from NV Bekaert [8].
2.3 Room for Research

While it has been stated in several journal articles that polypropylene fibers typically do not increase fracture energy as much as steel fibers, there are certainly research gaps to make these claims inconclusive. In one study of fiber-reinforced concrete under impact loading, the volumes of polypropylene fibers used were 0.5% or lower while steel fibers volumes were 0.5% or higher. The reason for the 0.5% polypropylene limit is “0.5% [polypropylene] fiber volume was the optimum fiber content that could be employed without major adjustments to mix design” [6]. As discussed in later chapters, the addition of polypropylene fibers at high dosages greatly affects the workability of a mix. At dosages as high as 2.0% mixes fail due to separation of coarse aggregate and fiber clumping. However, it is evident from research results that slight changes in fiber content can affect impact behavior.

In addition to slightly increasing the fiber dosage to get higher energy absorption in fiber-reinforced concrete, the type of polypropylene fiber used could possibly increase energy absorption. At higher dosages in the British Columbia studies, the fiber failure mechanism changes from breakage to pullout. There are however many variations in fiber lengths and tensile strengths. Forta Ferro fibers, at 2.25 in., are ¾ in. longer than the Forta fibers used at British Columbia. Another factor that might change the failure mechanism and energy absorption is the tensile strength. These synthetic fibers have tensile strength at least as high if not higher than comparable steel fibers. This might prolong breakage of fibers and, with longer fibers, pullout or de-bonding might not play such a large part in failure.

One particular need for research is the development of design methods for fiber-reinforced concrete. As a result of not having standard design methods, fiber-reinforced concrete must be tailor made for specific needs and applications [5]. Compared to fiber-reinforced concrete analysis, equivalent stress blocks for compression and tension regions are used in equilibrium calculations for a given cross section. There is some consensus that this analysis method can also be applied to FRC, but the location of steel bars is discrete whereas fibers are distributed throughout the cross section. To accommodate this difference, an
average residual tensile strength can be determined and used in the analysis and design of fiber reinforced concrete. This average residual strength comes from the ability of fibers to continue to carry load after cracking. An average residual strength then could be used in the same way that the stresses and forces that a steel bar would undergo in normal reinforced concrete to design the fiber reinforced concrete. The idea of average residual strength is discussed further in Chapter 3. This is however, all post-cracking behavior and therefore the mechanics are not exactly the same as pre-cracking strengths. “It is probably necessary to adopt a fracture mechanics approach to analysis and design, rather than the current strength-based approach.” [5]

ASTM standard C1399 was developed to determine the average residual strength of fiber reinforced concrete beams. It calls for a small fiber reinforced beam to be tested in a four point bending test with two loading steps. This test will be explained in Chapter 3. The end result of the ASTM C1399 test is that an average residual strength can be calculated. However, because there is no standard in fiber dosage, material or length it is very difficult to create a standard design method. This test method can be applied to a specific type of fiber and dosage and be used to determine usefulness for particular applications.

2.4 References

1. ACI Committee 544 Report 544.4R-88: Design Considerations for Steel Fiber Reinforced Concrete (Reapproved 1999).


CHAPTER 3: EXPERIMENTAL TEST SETUP

3.1 Fiber Selection

A synthetic fiber, designated Forta Ferro meaning “strong as steel”, produced by Forta Corporation was chosen for this study.

![Figure 3-1 Forta Ferro Fibers](image)

Unfortunately, results obtained through Forta Corp experiments on the impact behavior of Ferro fiber reinforced concrete are limited to observations of total debris following impact because of damaged instrumentation and inconsistent measurements. The preliminary testing performed by Forta took place at Columbia University and involved dropping fiber reinforced test panels from a predetermined height. As suggested through personal correspondence with Mr. MacDonald, the Director of Engineering for Forta Corporation, while some impact behavior of fiber reinforced concrete could be inferred by the amount of debris after the tests, without solid data significant insight could not be drawn from these tests. With the hope that the current study could assist in the understanding of how Forta Ferro fibers aid in dynamic properties of concrete, Mr. MacDonald donated fibers for this project.
While past research has generally focused on steel fibers, it is believed that synthetic fibers have certain advantages over steel fibers. Some of the advantages of synthetic fibers are listed below, but others include synthetic fibers weighing less than steel fibers and differences between electrical and thermal conductivity of the fibers. The physical properties of these synthetic fibers and their comparison with steel fibers are described below.

**Physical Properties of Forta Ferro Fibers:**
- Material Type: Virgin Copolymer/Polypropylene
- Form: Monofilament/Fibrillated Fiber System
- Color: Gray
- Acid/Alkali Resistance: High
- Specific Gravity: 0.91
- Tensile Strength: 90-110 ksi
- Length: 2.25 in.
- Compliance: ASTM C-1116

**Comparison with Steel Fibers**

Forta Corp has conducted several studies to evaluate the advantages of Ferro fibers over conventional steel fibers. These advantages are summarized as follows [5], [6]:

- Ferro fibers result in about the same flexural strength as steel fibers.
- Synthetic fibers result in only slightly higher compressive strength of concrete.
- Synthetic fibers significantly increased the impact resistance due to greater energy absorption. Similar benefits anticipated in blast and seismic resistance.
- In comparison of residual strength, Ferro fibers have a steel replacement value of 1:10, i.e. about ten times as many steel fibers are needed as the Ferro fibers.
- At similar dosages, Ferro fibers weigh significantly less than steel fibers.
- Ferro fibers have improved fire resistance due to less spalling and reduced heat conductivity.
- It is easier to pump synthetic fiber based FRC through hoses and elbows.
- Synthetic fibers have corrosive resistant properties.
3.2 Trial Mixes

Before covering experiments performed during the course of this research project it is necessary to discuss fiber reinforced concrete and the issues involved with mixing. Many fiber manufacturers promote the idea that the fiber reinforcement they produce has the capability of being added to any mix and performing to expected results. This might be valid, but not without certain caveats: fiber reinforcement can be added to any mix at certain dosages; and, fiber reinforcement should be premixed with aggregates.

The first step of this study on fiber reinforcement was learning the batching process. Under the assumption, emphasized by fiber manufacturers, that fibers could be added to any batch of concrete and, that the resulting concrete would have the desired behavior, the first trials utilized excess concrete provided by a local concrete supplier ordered for a separate, unrelated project. The concrete batch proportions were not known, but from observing the concrete it was evident that the workability was extremely low, the coarse aggregate was larger than 3/4 in., and the fine aggregate appeared to have a much lower proportion than the coarse aggregate.

Without knowing the specific batch proportions, types of aggregate, water-cement ratio, and other concrete properties it was difficult to determine how much fiber to add to the concrete. Typical values of dosage rates call for fibers to be added as a percentage volume, with rates ranging from 0.2% to 2.0%. However, since percent volumes cannot be determined without knowing the concrete specifications listed above, a quick calculation was made relating the weight of one full 6 in. x 12 in. cylinder mold to the total volume of the mold. Having the rough value of the weight of an assumed-full cylinder, a ratio could be created to calculate 2.0% fiber content by volume to a weight of fibers. The weight of fibers was then extrapolated to the total weight of concrete being added to the mixing drum. In addition to this calculation being unreliable because of the lack of information on the concrete mix, the assumption of the full 6 in. x 12 in. mold was invalid. Although, consolidation of the concrete took place when obtaining a rough estimate of the weight of a full cylinder mold, it was not fully consolidated and therefore the weight associated with the
volume was not accurate. However, for the purposes of this first trial batch, this quick proportioning to determine the amount of fibers was thought to be satisfactory.

Concrete was moved from the ready mix truck into a smaller revolving drum mixer with a capacity of about 3 cubic feet. The drum was dampened and set to rotate while the appropriate amount of fibers was determined. Once the fiber dosage was calculated, the fibers were added to the mix slowly and at a fairly constant rate while the drum continued to rotate. It was immediately evident that the addition of fibers resulted in some undesirable effects. At first the fibers were not breaking apart, they were clumping up and hair-balling towards the front of the mixer.

Next the fibers seemed to soak up the mortar and in effect to expose the course aggregate. Along with the lack of workability of the ready mix concrete that was supplied, the clumping of fibers made mixing the fibers with this batch of concrete very difficult. Although specimens were created from this batch, the specimens were later discarded after removing the molds and finding a large number of air pockets and voids.

At first glance, it appeared that the concrete workability or lack thereof was the primary reason why this first trial batch failed. It was thought that if the concrete displayed more workability, the fibers would be better able to disperse in the batch while avoiding separation of the course aggregate. Therefore, a second trial of the ready mix batch was acquired, but this time superplasticizer was to be added to the batch while in the mixing drum in order to increase workability. A second batch of concrete from the same supplier was moved into the revolving drum. Furthermore, the fiber dosage was lowered to 1.0%. However, even in larger than normal volumes it was evident that the superplasticizer would not be enough for a mix with these ready mix proportions to be successful. The fibers still continued to hairball to the front of the drum and the course aggregate separated from the mortar.

Following the failures with the ready mix concrete, it was decided that the proper path to take was to create the mix in a controlled method on the NCSU Centennial Campus' Construction Facilities Laboratory. Forta has published specifications on concrete mixes
used on various projects where its fibers have been used. Proportions of each aggregate size and type as well as a sieve analysis showing the particle size distribution and gradation are documented in Forta Ferro publications. Although a sieve analysis was not conducted to determine the exact proportions of each aggregate size used in this experiment, it was decided that attempting the mix with the prescribed proportions and a reduced aggregate size would be enough for the initial trials under this new method of mixing concrete. The proposed mix had equal amounts of course and fine aggregates as well as a relatively high water-cement ratio. These two components of the batch should ensure higher mortar content in the mix as well as improving overall workability. The Forta mix proportions are listed in Table 3-1; values are for 1 cubic yard of concrete [5]:

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement (Type I/II)</td>
<td>617 lbs</td>
</tr>
<tr>
<td>Course Aggregate</td>
<td>1586 lbs</td>
</tr>
<tr>
<td>Fine Aggregate</td>
<td>1586 lbs</td>
</tr>
<tr>
<td>Water</td>
<td>292 lbs</td>
</tr>
<tr>
<td>Water/Cement Ratio</td>
<td>.5</td>
</tr>
</tbody>
</table>

Table 3-1 Design Mix Proportions

Along with the different batch proportions, other changes were made to the mixing process. Besides having additional fine aggregate, the size of the coarse aggregate was changed from being 3/4 in. and larger to a smaller coarse aggregate size of 3/8 in. Another significant change was the manner in which the fibers were added to the mix. In the first two trials, the fibers were added after the rest of the concrete components had been mixed together. With the water and cement combined during delivery of the ready mix concrete, time was limited for the fibers to thoroughly mix throughout the entire batch before the concrete began setting. However, for the current and all future mixes, the fibers were premixed with both small and large aggregates in order to break up the fiber bunches and ensure a more even distribution of fibers throughout the batch, and the cement and water were added only after it was determined that the fibers were thoroughly mixed. In addition to the premixing benefit of having the fibers dispersed throughout the mix, adding the water
and cement at a later time allows for the further distribution of the fibers without clumping, hair-balling, or loss of workability due to setting.

Two types of test specimens were created from these revised batches. Standard cylindrical specimens with a diameter of 6 in. and length of 12 in. and beam specimens 14 in. long, 4 in. wide and 4 in. high were created. The larger size of the cylinder was selected over the smaller 4 in. x 8 in. cylinders because of the fiber length. At 2.25 in., the Forta Ferro fibers are on the longer end of the range of fiber lengths. With this length, it was thought the fibers would be able to better display their behavior in the larger specimens. The beam size was governed by the ASTM Standard C1399 that would be used for measuring the average residual strength of fiber reinforced concrete. The cylinders would be used for split tension and compression testing and the beams would be used for measuring residual strength and for dynamic experiments.

### 3.3 Material Testing

This section discusses the various tests performed on fiber reinforced concrete throughout this research project. These include splitting tensile strength as well as dynamic drop weight tests. The following tests were performed on three different types of specimens:

- Splitting Tensile Strength of 6 in. x 12 in. Cylindrical Concrete Specimens
- Compressive Strength of 6 in. x 12 in. Cylindrical Concrete Specimens
- Average Residual Strength of Beam Specimens
  - 4 in. x 4 in. x 14 in.
  - 2 in. x 2 in. x 14 in.
- Drop Weight Beam Tests on 2 in. x 2 in. x 14 in. Beam Specimens

The following section describes in detail the testing procedures for each experiment listed above, the associated ASTM standard used as a procedural guide, any adjustments
made to the ASTM procedure to account for fiber reinforced concrete behavior, as well as any difficulties with the testing procedures. However, since no standard currently exists for drop weight tests, the specifics and rationale for the procedure implemented in this project will be described in depth. Results for all tests will be presented and discussed in subsequent chapters.

3.3.i Splitting Tensile Strength of Cylindrical Concrete Specimens

The split tension test is an indirect way of measuring the tensile strength of concrete. These tests were performed with a Forney Cylinder Testing Machine at the Construction Facilities Laboratory. The test method involves applying a diametric, compressive force along the length of the cylindrical specimen. The stress distribution, shown in Figure 3-2, along the plane of the applied load has a high compressive stress at the extreme fibers, but rapidly changes to a nearly uniform tensile stress in the transverse direction over the remaining cross section.

Figure 3-2 Split Tension Stress Distribution [8]
A tensile failure rather than a compressive failure occurs due to the high compressive strengths of concrete. Given the maximum loading and geometric parameters of the specimen, the splitting tensile force can be determined. The determination of the splitting tensile strength for cylindrical concrete specimens is given in ASTM Standard C496/C496M–04.

Since the test specimens, at 12 in., were longer than the diameter of the bearing face of the upper bearing block and the lower bearing surface, steel plates were used to extend the bearing surfaces to the entire length of the specimen. In accordance with the ASTM standard, plywood bearing strips were placed between the bearing surfaces and the test specimen. The nominally 1/8 inch thick plywood strips were cut to 14 inches in length and one inch in width to span the entire length of the concrete cylinders and were replaced for each test specimen.

According to the standard, the load is applied continuously at a constant rate of 100 to 200 psi/min splitting tensile stress until failure. However, because the rate is monitored as a load and not a stress, a load rate between 1.12 k/min and 2.26 k/min was used. At failure, the maximum applied load, the type of failure, and the appearance of the concrete are recorded and a calculation for splitting tensile strength is performed. The splitting tensile strength, \( T \), of the specimen is calculated by:

\[
T = \frac{2P}{\pi l d}
\]

Where \( l \) is the length of the cylinder, \( d \) is the cylinder diameter, and \( P \) is the applied load. Typically, splitting tensile strengths are greater than direct tension strengths, but less than flexural tensile strengths [3].

While ASTM C496 calls for load controlled testing, this method of application is not ideal for most fiber reinforced concrete experiments. Typically, a deflection controlled process is required to observe any post-cracking behavior since the load significantly drops off after the crack but data must still be measured and recorded. However, since there is
minimal deflection from compressing the specimen, deflection control would not work for this experiment and load control would need to be used. According to the ASTM standard, the maximum load indicated by the testing machine at failure should be recorded [3]. As noted earlier, one beneficial property of adding fibers to a concrete mix is the after cracking behavior, i.e. the concrete continues to carry load after the peak loading is reached and cracking begins. Rather than recording just the load at which the specimen first cracks the effect of the presence of fibers in the concrete will continue to be monitored by an increase in load. This is accomplished by loading the specimen at 1.12 to 2.26 k/min as prescribed, but rather than stopping at the initial crack, the load will continue to be increased until no additional load can be achieved.

The concrete cylinder will crack, but will refrain from completely shattering as load is continuously added past the first crack. The fibers should bridge the gap that is created by the load and assist the concrete in supporting additional load. It is also expected that unless the fibers severe or pullout, that the concrete specimen will not completely separate into two or more discrete segments, but rather maintain some integrity.

3.3.ii Compressive Strength of Cylindrical Concrete Specimens

Testing for the compressive strength of cylindrical concrete specimens is a standardized and highly utilized test from basic construction to research and development of concrete materials. For this project, the tests were performed with a Forney Cylinder Testing Machine. A cylindrical specimen is sandwiched between two flat bearing blocks and compressively loaded until failure. Prior to performing the test, data is collected on the dimensions of the specimens. Average lengths and diameters as well as weights are measured. The dimensions and weight will be used to calculate compressive strength and the density. The test is described in ASTM Standard C39.

The loading takes place continuously and without shock at a rate of 35 ± 7 psi/s or 59.4 k/min. The maximum load carried by the specimen is observed and recorded, in
addition to the failure type and appearance of the concrete. The compressive strength, $f_c$, calculation is simply the peak load, $P$, divided by the cross-sectional area, $A$ [2]:

$$f_c = \frac{P}{A}$$

Although there might be a slight increase in compressive strength because of the fibers, supporting research suggests that Ferro fibers have little effect on the compressive strength of concrete [5], [6]. Most studies agree that, like conventional steel reinforcement, the benefits of fiber reinforced concrete are tensile in nature. Thus, the experimental procedure for compressive strength was used directly in this study on FRC.

3.3.iii Average Residual Strength of Fiber-Reinforced Concrete

There is currently no design method available for fiber reinforced concrete; but one approach to design is to use residual strength which is defined by ASTM as the stress-carrying ability of cracked concrete specimens. It is “an engineering stress computed using the flexure formula for linear elastic materials and gross section properties” [4]. ASTM Standard C1399 is the standard test method for Obtaining the Average Residual-Strength of Fiber-Reinforced Concrete. The residual stresses are determined from the deflections after cracking the specimen under bending. Figure 3-3 below displays several load deflection curves for fiber reinforced concrete as well as for an un-reinforced specimen. The post-cracking behavior of fiber reinforced concrete is clearly demonstrated in the figure. The test method gives load-deflection data beyond the peak loading, i.e. post-cracking strength.
To be able to use residual strength in FRC design, a simplified representation of residual
stress - known as average residual stress - is created. The following figures compare the
sectional analyses of a normal reinforced concrete beam cross section and a fiber reinforced
beam cross section. Figure 3-4 depicts a rectangular beam reinforced with conventional steel.
Notice in particular that in this figure the stress distribution is zero below the neutral axis.
The only force acting on this lower section is the tension in conventional steel reinforcement.
Therefore, once the beam is cracked, the steel bars are the only resistance to further cracking.
In Figure 3-5, a reinforced, rectangular, concrete section which has been further reinforced with fibers is shown. While this beam has the same tensile forces contributed from the conventional steel as the normal reinforced beam above, the addition of fibers changes the overall stress distribution, specifically in the region below the neutral axis. This section analysis shows how average residual stress can be applied to the design of fiber reinforced concrete.

Figure 3-5 Design Assumptions for Singly Reinforced Concrete Beam Containing Fiber Reinforcement

The assumed stress distribution of a cracked section is displayed, showing both the concrete compressive region as well as the post-cracking residual stress region below the neutral axis. This tensile stress region would be obtained from a residual strength test if known values aren’t readily available. In this simplified representation of the stresses, the compression zone and the residual stress tensile zone are represented by rectangular distributions. Similar to the 0.85f’c simplified stress block for the concrete compression zone located above the neutral axis, average residual stress allows the residual stress on the tension side to be represented by a single average stress value, σr. Once this average residual strength is determined, further analysis and design can be conducted on the fiber reinforced
concrete section. As shown in the figure, the average residual stress allows for a resultant
tensile force to be determined in the cracked concrete specimen allowing for conventional
design methods using force and moment equilibrium.

While there is currently no theoretical method for determining average residual stress, ASTM C1399 provides a means to experimentally obtain this value. This test is performed on 4in x 4in x 14in beam sections. The testing procedure resembles a four point bending test but has slight variations. Instead of a single loading step, measuring the average residual strength of fiber reinforced beams as dictated by C1399 involves a two step loading process. The initial loading step is used to crack the specimen through bending so that the post-cracking behavior can be observed. This step of the procedure is performed with a ½ in. steel plate placed between the supports and the concrete specimen. The main purpose of the steel plate is to control the deflection post cracking. The addition of a steel plate is a modification of the standard four point bending test. In adding the plate, the value of the peak load achieved before cracking is of no importance because of the presence of the plate. The peak load does not represent the load the concrete can endure before cracking as it would in a standard four point bending test. This is not a problem since the peak load behavior is not the intended purpose of this procedure. Figure 3-6, shown below, is the schematic from ASTM C1399 for the average residual strength tests.
In this figure, the plate is shown under the concrete specimen. For the second loading step, the load is reduced to zero and the plate removed. The beam is then reloaded; this time until failure. The second loading step demonstrates the ability of the fiber reinforced concrete to carry load post-cracking; i.e. the residual strength.

In addition to the testing apparatus and support structure, the test equipment needed for this standard includes electronic transducers to measure deflection at mid-span and at each support location, a load cell, a steel plate, and data acquisition equipment. To continue record data after reaching the peak load, this test is deflection controlled and post-cracking deflection data is acquired.

The two loading steps provide two loading curves as shown below in Figure 3-7.
As mentioned earlier, the initial loading curve is not representative of the peak load the specimen can tolerate because of the steel plate below the concrete beam, and therefore this peak load is not used for analysis purposes or for comparison to other tests. The reloading curve the figure depicts shows the values of deflection that should be used to calculate the average residual strength.

ASTM Standard C1399 specifies a loading rate of 0.025 ± 0.005 in/min. Loads are recorded at specific values and loading is terminated at a prescribed value of 0.050 in. In English units, the loads are taken at 0.020, 0.030, 0.040, and 0.050 in. and denoted as $P_A$, $P_B$, $P_C$, and $P_D$. The average residual strength can be then calculated from this data. The equations used to calculate average residual strength are [4]:
\[ k = \frac{L}{bd^2} \]

\[ \text{ARS} = \left( \frac{P_A + P_B + P_C + P_D}{4} \right) \times k \]

Where \( L, b, \) and \( d \) are the length, width, and height of the specimen respectively.

This standard test method was developed as a means for measuring the performance of fiber reinforced concrete. The most valuable advantage of fiber reinforced concrete over normal reinforced concrete is its ability to continue to carry load after cracking. This leads to the potential for high energy absorption which has many practical applications. The test described in ASTM standard C1399 allows for quantitative comparisons between various fiber reinforced parameters including material, shape, size, as well as fiber content.

The ability to quantitatively understand how each of these parameters affects the fiber reinforced concrete is a great asset. Currently, there is no standard design method for fiber reinforced concrete. Understanding how the above parameters change average residual strengths and other properties such as density and weight would greatly assist the designer, and provide an understanding as to which parameters can be adjusted to optimize design and hence minimize costs.

The standard allows for “comparative analysis among beams containing different fiber types, including materials, dimension and shape, and different fiber contents” [4]. However, some of the test procedures needed to be modified from the ASTM standard for this project because the FRC specimens did not deflect in the manner predicted by the standard. It was originally assumed that, due to the wide range of fibers types, the fiber reinforced concrete used in this study would not have the same results that the fiber reinforced concrete used to produce this ASTM standard, but the extent of variation was unknown. Keeping this in mind, it was expected that the prescribed measurement values including the termination value of 0.050 in. would not necessarily work for this study. In fact, as shown in Chapter 4, the beams continued to carry load far past this value. One other modification from the ASTM experimental procedure concerned the initial loading step. It
was found during testing that the initial loading step did not produce a crack before 0.008 in. Even though the ASTM standard states that failing to crack before 0.008 in. invalidates the test, the initial loading step in the current study was carried out until a visible crack was achieved since observing and quantifying the post cracking behavior of the specimen is the purpose of the experiment. It was also evident from the load vs. deflection curves that the loading values taken at the ASTM prescribed range of deflections (0.020-0.050 in.) would not be a complete representation of the average residual strength. The beams continued to carry a significant amount of load past 0.050 inches. The results in the following chapter describe the additional residual strength.

3.3.iii.a Average Residual Strength Tests of 4 in. x 4 in. x 14 in. FRC Beams

The average residual strength tests were performed at North Carolina State University's Construction Facilities Laboratory (CFL) located on Centennial Campus. In the end, two separate test setups were used for this experiment. Due to the high level of traffic in the structural section of CFL at the time of the first trial, the experiment was setup in the concrete lab. The concrete lab at CFL had an appropriate support structure and therefore the initial concerns were limited to the use of a manual hydraulic jack that would be used to apply load to the specimens. This initial test setup was visually monitored and manually controlled. The first trial test setup is shown in Figure 3-8. As shown, a load cell was placed under the piston to record the applied load while a spring loaded linear voltage displacement transducer (LVDT) monitored the stroke of the ram. Two half inch LVDTs were placed over the supports while a string potentiometer was attached at midpoint of the steel plate during the initial loading step and the concrete specimen for the reloading step. The average of the support deflections was subtracted from the mid-span recording in order to obtain net deflection.
With the large number of projects taking place at the CFL the windows based data acquisition was also unavailable for this initial trial; and so a DOS based data acquisition system was used. This DOS based system was only capable of transferring data through floppy disks which greatly reduced the amount of data that could be stored and shared. This limited the test due to the file size that would be produced. Run times had to be estimated and adhered to to ensure all data was collected while care had to taken to not overfill the diskette capacity. Also, since the load was applied manually, a high sampling rate was used to ensure that the load increments could be closely monitored on the computer and that all deflection data was collected. If a low rate had been used, the probability (specifically in the reloading step) of missing deflections, because the load was only slightly increasing with each pump, was high. The high sampling rate and the low storage capacity greatly limited the data acquisition used for this initial test setup.

As previously explained, the standard for measuring average residual strength requires a displacement controlled environment. The hydraulic ram, however, is usually
utilized for load controlled testing. A spring loaded LVDT was attached to the piston in order to measure the displacement of the ram in an attempt to achieve displacement control. However, with the hand pump, it was difficult to apply a steady load rate while monitoring the displacement of the specimen. An approximation to a steady load rate with this setup was to observe the load vs. time graph on a computer display while applying the load.

A second problem involved the string potentiometer (pot) which was used to measure mid-span deflection. The string pot used was initially attached to the specimen using a screw driven through balsa wood epoxied to the center of the beam. However, it was difficult to maintain the epoxy connection once the beam began to crack. In attempt to remedy this problem, monofilament line was wrapped around the beam and then attached to the string potentiometer. With this system the string potentiometer continued reading deflections even after the beam cracked. Another problem was that the potentiometer had a 0-25 in. measurement range while the deflections of the specimens were under an inch. A string pot with this stroke range typically isn't as accurate when measuring smaller displacements, yet there was not enough room between the beam and the base of the support to utilize a deflection-range-appropriate LVDT. Therefore the only method available to measure mid-span deflection was to thread the string of the potentiometer through an existing hole in the base of the support structure while the pot was housed below the base.

Revising the instrumentation, acquisition, and load application systems used in the initial test setup was necessary for future residual strength tests. To overcome the problems with the hand-pumped hydraulic ram, a digitally controlled Material Testing System (MTS) with a hydraulic actuator was used. Fortunately, for the extent of the second set of residual strength tests, the CFL schedule allowed for the use of this equipment where it was being used by other projects during the first trial. The MTS could digitally monitor both load and deflection and could be set to perform the test at the prescribed deflection rate. In addition, a Windows based computer was available removing any data acquisition limitation present in the initial trial. Additionally, as noted by Hatem Seliem - a PhD student, and confirmed through the noise present in the graphs produced from the first tests, the acquisition rate used
for the first test was too high and would not be needed for the second set of tests. While the high acquisition rate was thought necessary for the first setup due to the manually controlled load application, it was not required when using the MTS and Windows DAQ system. Furthermore, although, a load cell was used as part of this setup, it would prove to be redundant because the MTS had its own load cell and recording system.

To be able to use the MTS, an existing support structure was retrofitted for the 14in long beams. The supporting beam had mounting brackets welded to the bottom which were inserted into the MTS grips. In addition to the change in the test support system and load application method, the mid-span deflection was measured with a 4 in. spring loaded LVDT rather than the original string pot. This addressed both issues previously discussed with the string potentiometer. First, the LVDT has a smaller measurement range and hence measures the smaller displacements more accurately and, second due to the spring, the LVDT required no adhesive to maintain contact with the beam specimen or the plate (depending on which load step). Once the plate was removed the spring allowed the LVDT to measure deflections directly from the specimen bottom of the specimen. The new set up is shown in Figure 3-9 with the steel plate for the first loading step and without the steel plate for the second loading step in Figure 3-10.
Figure 3-9 Initial Loading Step for the Average Residual Strength Test
(The plate beneath the beam controls deflection after cracking)

Figure 3-10 Final Loading Step for Average Residual Strength Test
(The steel plate is removed and the beam is tested under deflection control)
3.3.iii.b Average Residual Strength Tests of 2 in. x 2 in. x 14 in. FRC Beams

A series of average residual strength (ARS) tests were performed on 2 in. x 2 in. x 14 in. beam specimens, in addition to those on the 4 in. x 4 in. x 14 in. specimens. The reason for using the smaller beams was that the drop weight tests (to be described in the next section) could only be performed on the smaller specimens and it was desirable to have static test data on beams that were the same size as those used in the drop weight tests. Since the maximum loads were expected to be much smaller than for the 4 in. x 4 in. x 14 in. beams, a different test setup – this time in Mann Hall Structural Behavior Laboratory - was used.

The Mann Hall laboratory did not have the capability to apply loads in the 2000 pound range automatically as was done with the MTS unit; instead a manually operated screw actuator with a 2 kip load cell was used. Although, manual application of the load proved difficult for earlier ARS tests it was workable here because of the overall setup reduction to accommodate the smaller beams size. The actuator and data acquisition system were in adjacent and therefore it was believed that the load rate and deflections could easily be monitored and controlled.

Since the test specimens were smaller, it was no longer appropriate to use a ½ in. thick steel plate. The smaller beams would not be able to deflect enough to crack with a ½ in. plate supporting it. Therefore, a 1/16 in. steel plate was used to control deflection during the initial loading step. This plate would allow a crack to form while controlling deflections and preserving the ability to observe post cracking behavior in the second loading step.

Further adjustments included changes to the support structure and other instrumentation. To measure the required displacements, three linear voltage displacement transducers (LVDT), each with a measurement range of ±½ inch were used at mid-span and the two supports. The voltage changes in the LVDTs were monitored with digital multimeters while the load was monitored by a P-3500 strain indicator. Figure 3-11 shows the
smaller scale residual test setup with roller supports maintaining the appropriate 12 inch span.

Figure 3-11 Average Residual Strength Test for 2 in. x 2 in. x 14 in. Beams

3.3.iv Drop Weight Tests

Unlike other experiments performed during this study, there is no standardized method, ASTM or otherwise, for impact testing on fiber reinforced concrete. While there are certainly accepted methods of impact tests such as Charpy and Izod impact tests, there are no existing standards specific to impact tests for fiber reinforced concrete. With no clear standard to follow, prior research was consulted for ideas. It was found that existing research tends to favor a drop weight system for impact testing. In fact, Forta Corp performed drop tests by dropping fiber reinforced concrete panels onto a support frame. However, the level of impact from dropping the entire panel section onto a supporting structure resulted in large amounts of debris and damaged instrumentation. While refinement of this process was one option, further investigation into existing research revealed a well developed drop weight testing system used for impact testing by Dr. Sidney Mindess at the University of British
Columbia (UBC). Employing the idea of a guillotine, a guided drop weight system, shown in Figure 3-12, was developed at UBC and had been used to test many types of concrete specimens including fiber reinforced concrete. Current UBC tests performed with this drop weight machine implement a variety of data acquisition and instrumentation including accelerometers, load cells, photoelectric sensors, and high speed cameras [6]. The success of research conducted by Dr. Mindess and his colleagues at UBC led to the conclusion that a similar style test would best serve the purposes of this research project. The drop weight test to be used for the current study would need a guided mass centered over the specimen, a quick release method, and at least load and acceleration instrumentation. However, time and funding constraints prevented development of a robust drop weight system as the one in British Columbia for this study. A smaller, more fundamental type of drop weight test was designed.
Keeping the quickly-released-mass idea of a guillotine in mind, the idea for this drop weight system was to find a relatively simple way to guide the mass down to the concrete specimen. In the end, a determining factor in deciding how this would be done was the load cell instrumentation. While there are load cells rated for impact loads, Interface technicians suggest that non-impact rated load cells can be used for impact loading as long as the intended load is less than about half of the rated load cell capacity. A two kip load cell, usually used for quasi-static testing in the undergraduate structural lab course at NCSU, was used for both the residual stress tests on the smaller beam specimens and the drop weight test. Although higher capacity load cells were available, the relative size of the larger load cells compared to the small 2 in. x 2 in. x 14 in. beams seemed excessive. The two kip load
cell was four inches in diameter while the large load cells were at least six inches and weighed significantly more.

Once the load cell was selected, a mass was machined with a diameter of four inches and a threaded connector so that it could be connected to the load cell, shown in Figure 3-13. An eye bolt was attached to the top of the mass to allow for easy hoisting.

![Figure 3-13 The Drop Weight including Steel Mass and Load Cell](image)

Having selected the load cell and mass, the next step was to decide how to guide the weight onto the beam. The basic idea was to have the load free fall through a tube centered over the beam. The tube would have to be slotted to allow the cable to move freely. Furthermore, it was important to allow enough space between the guide and the mass to minimize friction, but not have the space be so large that rattling or rocking of the falling specimen might occur. Early brainstorming thoughts included using wheels or slides to reduce the gap between the weight assembly and the inner walls of the tube while cutting down on friction. However, the end result did not require either of these.

The use of a PVC tube was determined to be a simple, yet effective way to guide the mass to the specimen. A one inch section could be removed from one side of the tube to
allow the data acquisition cable to pass through when dropped. With the diameter of the weight assembly being four inches, the inner diameter of the pipe would need to be slightly larger. The target PVC diameter was five inches; however, this size PVC pipe is rarely used or stocked and must be specially ordered in bulk. The other options were a 6in. pipe or a 4in. pipe. While these were outer diameters, it was felt that the 6in pipe would have too much space between 4in diameter of the weight and inner wall of the tube. Since the PVC pipe would have to have a slit removed to allow the data acquisition cable to move freely, the plan was to use a four inch pipe and use the mounting brackets to pry the pipe open to the necessary diameter for free fall.

Three-sided wooden supports, shown in Figure 3-14, reinforced with plywood corner bracing were used as the brackets for the drop weight setup. The dimensions of these supports were nominally five inches high, eight inches deep, and 13 in. wide. The wood used to construct the supports was one inch thick while the plywood supports added an additional \( \frac{3}{4} \) inches.

![Figure 3-14 Support Brace for Drop Weight Test](image)
Holes were drilled in the backside of the supports so they could be attached to a semi-permanent aluminum support structure used in the undergraduate structures lab. Holes to allow six inch long bolts to anchor the PVC pipe were drilled into the sides. These bolts would be tightened with wing nuts to pry open the slit in the PVC. Corresponding holes were drilled and threaded into the PVC. These holes became an issue when the pressure of maintaining the expanded diameter proved too extreme and stripped the threads. Unwilling to give up on this test set up, it was necessary to develop ways to reduce the stress on the threads in order to maintain this type of connections.

It was first thought that only three of these supports would be sufficient for the guide. The supports were set 13.5 in. from each other with the lowest being four inches from the base of the pipe. However, the third support from the bottom, the highest support, had to withstand more pressure due to the unsupported PVC pipe that extended upwards. The threads in this support were the first to fail. As the wing nuts were tightened to extend the slit in the pipe to allow free fall, the pressure became too much and the bolts pulled out, stripping the threads. In attempt to solve this problem, thicker bolts were used to give more surface area to withstand the force. However, the problems with bolt pull out persisted so a fourth support was added 13.5 in. above the 3rd support. As shown in Figure 3-15, four of these wooden brackets, spaced 13.5 in. apart, supported the PVC pipe.
Even with the additional support, the pressure of maintaining the open gap in the PVC pipe continued to be a problem. With this additional support, the pressure between the brackets seemed to be more uniform, but as the wing nuts were tightened more and more to allow for the weight to free fall and the data acquisition cable to move without restriction, the
threads in the PVC pipe continued to fail. In another attempt to aid this issue, thread lock adhesive was added to the threads, yet showed no improvement.

The dilemma remained that the pipe diameter was too small to allow the weight and load cell to fall freely, but extending the pipe proved to be difficult because the force needed to do so was too much for the threads. A better solution was needed. One possibility was adding a nut on the inner side of the tube so that the nut rather than the threads carried the pressure of the pipe. However, this would present an issue with the possibility of the drop weight hitting the nut as it fell through the guide.

Alternatively, it was decided that the bolts, which had a smooth, rounded head, were to be cut down to 1½ inch and inserted into the tube with the bolt head on the inside. As opposed to a nut bearing the pressure of separating the pipe and possibly hindering the drop weight, the smoother bolt head would sustain the pressure of prying the pipe open while minimizing hindrance of the fall if contact with the bolt was made. Since the bolts were reduced in size in allow for insertion into the tube, a collar splice was used along with 3/8 inch threaded rod to cover the remaining space between the PVC pipe and the support. A wing nut was then threaded onto the rod and tightened. This method, although not ideal, because the bolt heads were now on the inside of the pipe, finally provided a workable solution. The bolt heads were smooth enough that if contact did occur with the drop weight that it would not snag or get stuck in the pipe. Furthermore, the heads carried the pressure and allowed the gap to be sufficiently opened for the data cable to pass through and the drop weight to fall freely.

One downside of having the bolt heads on the inside was that the tube had to be pried open farther to avoid too much contact between the mass and the bolt heads. Although only slightly wider, the mass barely touched the walls of the pipe with the additional space. While minimum contact with pipe walls was always intended, too much space between the wall and the drop weight could increase rocking as the weight dropped. Furthermore, too much space could render the PVC pipe useless. With the additional space, was the PVC guide useless? While there might not have been excess contact with the pipe when the weight dropped, the
pipe was still useful in guiding the mass onto the midsection of the concrete specimen. For instance, if the pipe was not present to steady the mass when raising it to its drop height, then it would be likely that the weight would not drop on the center of the beam. Furthermore, if the PVC pipe was not present then the mass would tumble after impact and possibly damage instrumentation. Again, while contact was not ideal, a complete free fall (without the guidance of the PVC pipe) would make the test very difficult to perform and would put at risk the instrumentation.

Once it was evident that the bolt heads on the inside of the pipe were successful in maintaining the necessary gap in the PVC pipe while allowing for free fall of the drop weight, it was established that this would be the experimental setup for the drop weight tests. After the test set up was determined, the only variation that took place was the size of the drop weight. The full weight, shown in Figure 3-17, was cut into two sections as shown in Figure 3-16. The sections were connected to each other with threaded rod and then attached to the load cell in the same manner. Having the option of using either the full mass or the half the mass would be useful for further research on impact testing.
Figure 3-16 Two Sections of the Drop Weight

Figure 3-17 Two Weights Connected by a Threaded Rod to Allow for Larger Weight
3.4 References


CHAPTER 4: EXPERIMENTAL TEST RESULTS AND DISCUSSION

4.1 Trial Mixes

The mix design used for this research project was based on a sample mix design published by Forta and is shown in Table 4-1 Sample Mix Design Proportions.

<table>
<thead>
<tr>
<th>Table 4-1 Sample Mix Design Proportions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement (Type I/II)</td>
</tr>
<tr>
<td>Course Aggregate</td>
</tr>
<tr>
<td>Fine Aggregate</td>
</tr>
<tr>
<td>Water</td>
</tr>
<tr>
<td>Water/Cement Ratio</td>
</tr>
</tbody>
</table>

Using these proportions and typical standard specific gravity values, given in Table 4-2, batch volumes were determined and used to calculate the volume and weight of fibers added to the concrete mix.

<table>
<thead>
<tr>
<th>Table 4-2 Specific Gravity Values for Concrete Materials</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific Gravity</td>
</tr>
<tr>
<td>Coarse Aggregate</td>
</tr>
<tr>
<td>Fine Aggregate</td>
</tr>
<tr>
<td>Forta Ferro Fiber</td>
</tr>
<tr>
<td>Cement</td>
</tr>
</tbody>
</table>

Since fiber dosages are typically prescribed as percent by volume, having batch weights and specific gravity values provided the means to interchange volumes and weights; volumes for analysis and weights for the mixing process. A data spreadsheet was created to take a given amount of concrete, specifically for x number of concrete cylinders and y number of concrete beams, increase the total concrete by 10% for losses during mixing, and along with a
predetermined fiber dosage as a percent by volume, calculate total weights of coarse and fine aggregates, water, and fibers. For example, the amount of concrete needed to make 5 4 in. x 4 in. x 14 in. beams and 8 6 in. x 12 in. cylinders is shown in Table 4-3.

**Table 4-3 Example of Required Concrete Volume Calculation**

<table>
<thead>
<tr>
<th>Determination of Required Concrete Volume</th>
<th>Cylinder</th>
<th>Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of Specimen:</td>
<td>8</td>
<td>5</td>
</tr>
<tr>
<td>Volume of Single (cubic feet):</td>
<td>0.20</td>
<td>0.13</td>
</tr>
<tr>
<td>Total Volume (cubic feet):</td>
<td>1.57</td>
<td>0.65</td>
</tr>
<tr>
<td>Total Volume x 1.1 (cubic feet):</td>
<td></td>
<td>2.33</td>
</tr>
</tbody>
</table>

The batch weights, in Table 4-4, were then calculated for a fiber dosage of 0.5% using batch proportions and specific gravities:

**Table 4-4 Example Mix Design Obtained After Mix Volume Determined**

<table>
<thead>
<tr>
<th>Example Mix: Fiber Reinforced Concrete</th>
<th>Fiber Dosage: 0.5%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Volume: 2.3 cubic feet</td>
<td></td>
</tr>
<tr>
<td>Component</td>
<td>Weight (lbs)</td>
</tr>
<tr>
<td>-----------------</td>
<td>--------------</td>
</tr>
<tr>
<td>Cement</td>
<td>52</td>
</tr>
<tr>
<td>Coarse Aggregate</td>
<td>135</td>
</tr>
<tr>
<td>Fine Aggregate</td>
<td>135</td>
</tr>
<tr>
<td>Water</td>
<td>25</td>
</tr>
<tr>
<td>Fibers</td>
<td>0.66</td>
</tr>
</tbody>
</table>

Although, the batch weight values were not exact because of the use of typical rather than exact specific gravities, they were acceptable because the goal was not to replicate a mix, but get a better understanding of the type of mix that would be successful. It should be noted that air content is not included in these calculations because the actual total volume is not known.

With the weights of each component, the concrete was ready to be mixed. The mixing process was performed in two steps. After dampening the mixing drum, the sand and rock were added to the drum and then the fibers were added at a rate slow enough to ensure
good distribution of fibers and to minimize clumping. The aggregates and fibers were mixed thoroughly before the second set of materials was added. Once the fibers were well distributed the cement, water and super plasticizer were added to the mix. In the first attempt, the entire batch was mixed together at one time. The result was that the drum - with a capacity of about 3 cubic feet - had difficulty rotating due to the weight of the mix - concrete and fibers spilled over the top of the drum and, once the specimens were cast and the drum emptied, unmixed sand was found at the very bottom of the drum. Although acceptable specimens were made from this batch, it was clear that the procedure needed to be modified.

The first suggestion to avoid overfilling the drum was to reduce the number of test specimens. This would in fact lower the volume of concrete needing to be mixed and solve the problem. However, reducing the number of specimens would also limit the amount of testing opportunities. Therefore before the next batch was mixed the batch weights were divided into two equal segments. This allowed for the same number of test specimens, but reduced the issues with an overflowing mixing drum.

While slight variations between concrete mixes can create slight variations in experimental results, care was taken to control proportioning the weights for each batch, which were now halved, as well as in maintaining a consistent mixing process. The revolving drum had no difficulty mixing the concrete following this change in the mixing process and suitable test specimens were created for various fiber reinforced concrete experiments. Table 4-5 below shows the specimen count created in each batch as well as a breakdown of which tests were performed. Each mix number refers to a different mixing date and the specimen count represents the total number of specimens created on that date.

<table>
<thead>
<tr>
<th>Specimen Size</th>
<th>Test</th>
<th>Specimen Count</th>
</tr>
</thead>
<tbody>
<tr>
<td>6 in. x 12 in.</td>
<td>Splitting Tensile</td>
<td>5  3  -</td>
</tr>
<tr>
<td>4 in. x 4 in. x 14 in.</td>
<td>Compressive Strength</td>
<td>4  3  -</td>
</tr>
<tr>
<td>4 in. x 4 in. x 14 in.</td>
<td>Average Residual Strength</td>
<td>4  6  -</td>
</tr>
<tr>
<td>2 in. x 2 in. x 14 in.</td>
<td>Average Residual Strength</td>
<td>-  -  6</td>
</tr>
<tr>
<td></td>
<td>Drop Weight/Impact</td>
<td>-  -  14</td>
</tr>
</tbody>
</table>

Table 4-5 Test Matrix
Noticeable differences between the mix fabricated in the lab and ready mixed concrete batches used for specimens at the early stages of this experiment include higher sand to rock ratio, higher water to cement ratio, a lower fiber content, and smaller aggregate. These changes in the mix meant that the concrete would have more mortar (water, cement, and fine aggregate) due to the increased sand and water-cement ratio which, along with less fibers and smaller rock, meant more workability. Although precautions can be taken to limit variability, it’s unavoidable with concrete batches of this scale because each mix inevitably has some variation in the nature of aggregates and the process of weighing components.

4.2 Material Testing Results

The experimental results for all tests performed in this research project are contained in this chapter. They consist of the following:

- Splitting Tensile Strength of 6 in. x 12 in. Cylindrical Concrete Specimens
- Compressive Strength of 6 in. x 12 in. Cylindrical Concrete Specimens
- Average Residual Strength of Beam Specimens
  - 4 in. x 4 in. x 14 in.
  - 2 in. x 2 in. x 14 in.
- Drop Weight Beam Tests on 2 in. x 2 in. x 14 in. Beam Specimens

4.2.i Results for Splitting Tensile Strength of Cylindrical Concrete Specimens

The splitting tensile strength tests were performed on specimens from two concrete batches. Loads, the average diameter, length, and weight were recorded for each test specimen. These values were needed to calculate the tensile strength and density of the FRC. Plain, non-reinforced concrete specimens tested for splitting tensile strength typically split
into two or more sections at failure as shown in Figure 4-1. These tests are performed until failure and the maximum load is recorded.

![Figure 4-1 Splitting Tensile Strength Tests Results for Plain Concrete [4]](image)

Although the fiber reinforced concrete cylinders split down the vertical diameter of the specimen as do the plain concrete cylinders, the FRC specimens do not fully break apart due to the presence of fibers as shown in Figure 4-2.

![Figure 4-2 Splitting Tensile Strength Test Results for Fiber Reinforced Concrete](image)
Furthermore, because of the fibers, the maximum loads that were recorded for this test were not the first cracking loads. Unfortunately, the difference between the first cracking load and the failure load was not noticed until after the first set of tests were concluded. While the maximum loads were recorded for both sets of samples, only the second set has data on the first cracking load.

The first set of samples was made from the original batch of concrete, the one that was not divided in half. However, even with the problems associated with this mixing process, the specimens did not appear to have any large flaws or voids and were deemed acceptable for testing. The dimensions of five test cylinders were measured and recorded in addition to the maximum test load obtained for specimen; these values are shown in Table 4-6 and Table 4-7.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Average Diameter, in</th>
<th>Average Length, in</th>
<th>Area, in²</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5.98</td>
<td>12.06</td>
<td>28.08</td>
</tr>
<tr>
<td>2</td>
<td>6.02</td>
<td>12.06</td>
<td>28.47</td>
</tr>
<tr>
<td>3</td>
<td>5.95</td>
<td>12.03</td>
<td>27.79</td>
</tr>
<tr>
<td>4</td>
<td>5.98</td>
<td>12.19</td>
<td>28.08</td>
</tr>
<tr>
<td>5</td>
<td>6.01</td>
<td>12.13</td>
<td>28.37</td>
</tr>
</tbody>
</table>

Table 4-7 Test Results for Split Tension Test 1

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Max Load, lb</th>
<th>f₀, psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>42750</td>
<td>377</td>
</tr>
<tr>
<td>2</td>
<td>43712</td>
<td>383</td>
</tr>
<tr>
<td>3</td>
<td>48179</td>
<td>429</td>
</tr>
<tr>
<td>4</td>
<td>43796</td>
<td>383</td>
</tr>
<tr>
<td>5</td>
<td>41732</td>
<td>365</td>
</tr>
</tbody>
</table>

Unlike the first set of cylinder tests, the second test specimens were monitored more closely. With the basic knowledge of fiber behavior gained from the first sets of tests, it was suspected that the presence of the fibers was allowing the concrete to carry load past initial
cracking. While maintaining safety precautions, the specimens were visually observed as load was applied. The load rate seemed to decrease as the load values crossed first cracking, and then the load rate picked back up and continued to rise. The initial cracking load was recorded for these specimens as a visual crack was observed. These specimens noticeably continued to carry load following the initial crack. Two loads were recorded for this test. The first loading point that was recorded was the load at which the cylinders first cracked. The second recorded load value was the maximum load carried by the test specimen. As with the first set of test samples, the dimensions of the specimens were measured and recorded along with the two loading points; the first cracking load and the maximum load. This data is shown in Table 4-8 and Table 4-9.

Table 4-8 Specimen Data for Split Tension Test 2

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Average Diameter, in</th>
<th>Average Length, in</th>
<th>Area, in²</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5.94</td>
<td>12.00</td>
<td>27.69</td>
</tr>
<tr>
<td>2</td>
<td>6.00</td>
<td>11.94</td>
<td>28.27</td>
</tr>
<tr>
<td>3</td>
<td>6.03</td>
<td>12.06</td>
<td>28.57</td>
</tr>
</tbody>
</table>

Table 4-9 Test results for Split Tension Test 2

<table>
<thead>
<tr>
<th>Specimen</th>
<th>First Crack, lbs</th>
<th>Max Load, lb</th>
<th>f’c, psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>60000</td>
<td>66331</td>
<td>593</td>
</tr>
<tr>
<td>2</td>
<td>53000</td>
<td>59206</td>
<td>526</td>
</tr>
<tr>
<td>3</td>
<td>58000</td>
<td>63730</td>
<td>558</td>
</tr>
</tbody>
</table>

As shown in the tables, the values for maximum load range from 41000 lbs to 48000 lbs for the first test and 53000 lbs to 60000 lbs for first crack load and 59000 lbs to 66000 lbs for the maximum load for the second test. These values seem reasonable. While the average
tensile strength for the first test was 387 psi with a standard deviation of 24 psi, the second test saw a higher average tensile strength of 559 psi with a standard deviation of 33 psi.

ASTM C496/C496M – 04 Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens suggests performing the test on at least two specimens. According to the standard, the test results from two specimens from the same batch should not differ by more than 14% of the average tensile strengths. The first test average tensile strength is 387 psi with a standard variation of 24 psi which is a difference of 6.2%. As for the second test, the tensile strength values vary by roughly 33 psi which is less than 6% of the average of 559 psi.

An interesting trend to notice in the second test is that the first cracking load occurs approximately 6000 lbs before the maximum load for all three test specimens. Furthermore, there is obviously a difference in the maximum loads the cylinders could withstand between the first and second set of tests. The lower results in the first sample are most likely attributed to the issues that arose during mixing. As previously stated, once the test specimens were cast, there was unmixed sand in the bottom of the mixing drum which could have led to the variation in test results. Although the results were not replicated between concrete batches, the variation between these batches is understandable given with the mix differences and is acceptable since the variation within each batch was under the 14% limit.

The reasons the number of test specimens tested for splitting tensile strength was reduced from five to three are a change in concrete allocation and a change in the mixing process. When the concrete was divided evenly to allow for easier batching for the second set of test specimens, some of the concrete was allocated to make more beam specimens. Moreover, there was not enough excess concrete to make complete concrete cylinders.

4.2.ii Results for Compressive Strength of Cylindrical Concrete Specimens

Compression strength tests were performed on both sets of concrete specimens. Some concerns arose for test results after the initial set of tests was completed. The densities
of the specimens seemed alarmingly low. It was determined that the weights of the specimens were erroneously measured. The weights of the cylinders and the resulting density values are shown in Table 4-10.

Table 4-10 Specimen Weight and Density for Compressive Strength Test 1

<table>
<thead>
<tr>
<th>Specimen Data</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Weight, lbs</td>
</tr>
<tr>
<td>1</td>
<td>11.82</td>
</tr>
<tr>
<td>2</td>
<td>11.94</td>
</tr>
<tr>
<td>3</td>
<td>12.12</td>
</tr>
<tr>
<td>4</td>
<td>11.94</td>
</tr>
</tbody>
</table>

Typical values for normal weight concrete are approximately 150 lbs/ft³ while lightweight concrete densities typically range from 90 lbs/ft³ to 120 lbs/ft³. Clearly, 60 lbs/ft³ are very low. The weights for the cylinders were very low, but the consistency in the weights demonstrated that the scale must not have been calibrated and tared, as opposed to errors in dimension measurement. The rest of the specimen data collected is shown in Table 4-11.

Table 4-11 Specimen Data for Compressive Strength Test 1

<table>
<thead>
<tr>
<th>Specimen Data Continued</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen</td>
</tr>
<tr>
<td>1</td>
</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>3</td>
</tr>
<tr>
<td>4</td>
</tr>
</tbody>
</table>

The compressive strength values for the first test samples are shown below in Table 4-12.

Table 4-12 Results for Compressive Strength Test 1

<table>
<thead>
<tr>
<th>Compression Test Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen</td>
</tr>
<tr>
<td>1</td>
</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>3</td>
</tr>
<tr>
<td>4</td>
</tr>
</tbody>
</table>
With the error in measuring the weight of the first specimens, care was taken when measuring the weights of the second set of test specimens. On average, these cylinders were about 15 lbs heavier. This data was more realistic and corresponded with expected values. As previously noted, typical densities for normal concrete are around 150 lbs/ft$^3$. Table 4-13 and Table 4-14 show the specimen data for the second test cylinders.

**Table 4-13 Specimen Weight and Density for Compressive Strength Test 2**

<table>
<thead>
<tr>
<th>Specimen Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weight, lb</td>
</tr>
<tr>
<td>28.7</td>
</tr>
<tr>
<td>27.7</td>
</tr>
<tr>
<td>28.1</td>
</tr>
</tbody>
</table>

**Table 4-14 Specimen Data for Compressive Strength Test 2**

<table>
<thead>
<tr>
<th>Specimen Data Continued</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen</td>
</tr>
<tr>
<td>1</td>
</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>3</td>
</tr>
</tbody>
</table>

The second set of compressive tests, like the second set of splitting tensile tests, resulted in an increase in strength. The average compressive strength increased from around 3050 psi to 4725 psi. The compressive strength results are shown below.

**Table 4-15 Results for Compressive Strength Test 2**

<table>
<thead>
<tr>
<th>Compression Test Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen</td>
</tr>
<tr>
<td>1</td>
</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>3</td>
</tr>
</tbody>
</table>
ASTM Standard C39 gives an acceptable range of individual cylinder strengths as 7.8% for laboratory conditions for 3 cylinders. For the first set of compression tests the standard deviation of 142 psi is 4.7% of 3051 psi. However, the second set of tests had a standard deviation of 478 psi which is over 10% of 4728 psi. This value is only 0.1% higher than the acceptable range for 3 cylinders as prescribed by the ASTM standard. The test provides additional helpful information on the quality and behavior of fiber reinforced concrete. It is generally accepted that fibers contribute more to tensile strength than compressive strength. However other useful information about the mixing procedure and fiber distribution can be ascertained specifically from the density of the cylinders. A large variation in density would depict a poor distribution of fibers from a lack of fibers or clumping of fibers. Furthermore, the fracture pattern can lend some insight into how the fiber reinforcement changes the concrete fracture behavior.

The ASTM C39 schematic of typical fracture patterns is shown in Figure 4-3 below along with figures of fiber reinforced cylinders after compressive strength tests in Figure 4-4.
As shown in the ASTM schematic, plain concrete typically breaks into multiple segments; either large pieces as in a diagonal or conical break or smaller pieces as in a type 2 or type 3 break. The pictures of the fiber reinforced concrete specimens show some similarity with the schematic fracture patterns with slight variations. For instance, figures 4A and 4C look like they are forming a conical break similar to the Type 2, but they do not show a vertical fracture. Figure 4B shows what looks like a diagonal fracture, but is not a clean break through the cross section. The cracks begin at the top and propagate vertically for a few inches before angling off. Furthermore, the fractures are not penetrating. The concrete seems to flake near the cracks but the cylinders failed before any large segments of concrete broke free. While the fibers may not add to the compressive strength of concrete, the presence of the fibers definitely changes the fracture mechanics and failure modes of concrete.
4.2.iii Results for Average Residual Strength of Fiber Reinforced Concrete

The average residual strength (ARS) experiments for fiber reinforced concrete were performed in three groups. The first two tests were conducted on the 4 in. x 4 in. x 14 in. beams while the final experiment was performed on 2 in. x 2 in. x 14 in. beams which were cut from existing 4 in. x 4 in. x 14 in. specimens. The tests performed on the larger beams came from the first and second concrete batches, while the final ARS tests specimens were fabricated from concrete mix 3.

As stated in the previous section of this thesis, the displacement results for this test did not satisfy the ASTM standard. ASTM prescribes a range of deflections (0.020-0.050 in.) in which to take load values for the average residual strength calculation, but this range would not be a complete representation of the average residual strength because the beams continued to carry load past this range. It was decided to load the beams until failure occurred which turned out to be beyond the deflection limit of 0.050 in.

4.2.iii.a Average Residual Strength Tests of 4 in. x 4 in. x 14 in. FRC Beams

The first set of tests was used to validate the test procedures and make adjustments to the loading apparatus and instrumentation.

Following the first set of tests, changes detailed in the Chapter 3 were made to the test setup, instrumentation, methodology, and data acquisition systems. Aided by the use of a digitally controlled Material Testing System and a Windows based data acquisition system, a second set of average residual strength tests was performed on the 4 in. x 4 in. x 14 in. beams. The load vs. deflection graphs shown in Figure 4-5, Figure 4-6, Figure 4-7, Figure 4-8, and Figure 4-9 depict the data for the second set of experimental results.
Figure 4-5  Average Residual Strength Trial 2 - Specimen 1

Figure 4-6  Average Residual Strength Trial 2 - Specimen 2
It should be noted that some slippage occurred during Test 3. Two deflection plateaus occur between 0.007-0.050 in. and 0.047-0.093 in. The plateaus displayed in the figure above represent a jump in the deflection values with relatively small increases in loads. This resulted in the post-cracking load not reaching levels that other test specimens achieved. Looking at the graph, it appears that the load and deflection started correlating shortly after 0.09 in., however, being deflection controlled the test was terminated before a higher load was achieved based on the deflection value obtained. If the slippage had not occurred, it is very likely that the average residual stress would be higher and similar to the other test results as the load appears to be approaching 1000 lbs in the figure. To maintain uniformity in determining the average residual strength, the deflection values, discussed later in this section, at which the load points are obtained, were not changed for this specimen.
Figure 4-8 Average Residual Strength Trial 2 - Specimen 4

Figure 4-9 Average Residual Strength Trial 2 - Specimen 5
Because of the adjustments made to the experimental test setup after the first set of tests, the experimental results for this series of tests were judged to be reasonable.

Three out of the five beams specimens for this set of tests – 3, 4 and 5 – showed relatively consist and expected behavior. Even though the specific value of the peak load is not a valid comparison to a standard four-point bending test due to the presence of the steel plate, the relative values between tests does have some significance. All three of these tests had a peak load of 4100 to 4600 lbs, but more importantly, they demonstrated the expected trend when comparing the initial loading and final loading steps. Unlike the first two specimens in this test set, these three tests had a greater initial loading stiffness relative to the final loading stiffness. In the first two tests, the peak loads were around 5800 to 6800 lbs, but the greater discrepancy lies in the relative stiffness of the initial and final loading curves for Test 1 and 2. The cause of this discrepancy is unclear.

The loading stiffness is the slope of the inclined section of the graph. When comparing the initial and final steps, it was expected that the final step would have a lower stiffness, a lower slope, than the initial loading step. The reason for this is that the first loading step is performed on an un-cracked section with a steel plate support while the second loading step is performed on a cracked section which has a reduced moment of inertia. Therefore, the final loading slope should be smaller than the slope of the initial loading curve. This behavior is also shown in the sample schematic from ASTM Standard C1399 shown in Figure 4-10.
Looking at all tests performed on 4 in. x 4 in. x 14 in. test specimen, it is unclear why the relative stiffness of the initial and final loading curves does not behave in a similar manner as tests 3, 4, and 5. Whether or not the plate is behaving in composite action with the concrete beam should not affect the fact that the un-cracked section should display more stiffness than the cracked section. If the plate and beam are in composite action then the stiffness of the plate adds to the stiffness of the beam by greatly increasing the moment of inertia. While the plate is only ½ in. thick, due to the high ratio of the modulus of elasticity of steel to the modulus of elasticity of concrete when behaving as a composite section the steel plate lowers the centroid of the specimen greatly increasing the moment of inertia. In turn, this raises the stiffness of the specimen. However, if the beam and plate are not acting as a composite section, then the stiffness of the plate and the stiffness of the beam act independently. It is believed that the plate and beam are not acting in full composite action because there is no bonding agent between the two surfaces other than friction. Whatever the case may be, the stiffness of the composite section or the un-cracked beam alone should be higher than the cracked section and therefore the initial loading stiffness should be higher than the final loading stiffness.

Furthermore, the reloading curves of tests 4 and 5 appear very similar. The load continues to rebuild until 1500 lbs before the curve plateaus and the post cracking behavior
of the fibers begins. In this post cracking region, the beam continues to carry the 1500 lbs while increasing deflections before tapering off after reaching 0.07 in.

It is worth noting that the use of the Material Testing System greatly improved this testing process. The deflection rate was programmed in the controller and the test procedure was easily replicated. This allowed for better monitoring of deflections as the test was conducted. The test was halted at the first sign of cracking for the initial loading step and the deflection values were monitored for the final loading step. While some tests were carried out to slightly higher deflections than others, the values could still be compared because the loading rate was the same for each specimen. All of the tests for the second trial set, save the first, were loaded past 0.11 in. deflection. However, since the first specimen was not, the data for all tests in this set was excluded past this point. The changes in the second test setup added much needed control and while the deflections continued past this point for some of the tests, for the most part, the behavior was considered complete at this point; the crack nearly propagating through the entire beam and the load falling to zero.

With the better results, average residual strengths were calculated. The deflection values that were used to obtain the load points for this experimental set were 0.025, 0.05, 0.075, and 0.10 in. – double the deflection range of the ASTM Standard. Figure 4-11 through Figure 4-15 shown below displays the loading points.
Figure 4-11  Load Values for Calculating ARS for Trial 2 - Specimen 1

Figure 4-12  Load Values for Calculating ARS for Trial 2 - Specimen 2
Average Residual Stress Test 3

Figure 4-13 Load Values for Calculating ARS for Trial 2 - Specimen 3

Average Residual Stress Test 4

Figure 4-14 Load Values for Calculating ARS for Trial 2 - Specimen 4
The associated values for load and deflections as well as the calculated average residual strengths are shown in the following tables.

**Table 4-16 Average Residual Strength for Trial 2 - Specimen 1**

<table>
<thead>
<tr>
<th>Displacement, in</th>
<th>Load, lbs</th>
<th>ARS, psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.025</td>
<td>2138</td>
<td>514</td>
</tr>
<tr>
<td>0.05</td>
<td>2493</td>
<td></td>
</tr>
<tr>
<td>0.075</td>
<td>2530</td>
<td></td>
</tr>
<tr>
<td>0.1</td>
<td>2249</td>
<td></td>
</tr>
</tbody>
</table>

**Table 4-17 Average Residual Strength for Trial 2 - Specimen 2**

<table>
<thead>
<tr>
<th>Displacement, in</th>
<th>Load, lbs</th>
<th>ARS, psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.025</td>
<td>1829</td>
<td>494</td>
</tr>
<tr>
<td>0.05</td>
<td>2142</td>
<td></td>
</tr>
<tr>
<td>0.075</td>
<td>2529</td>
<td></td>
</tr>
<tr>
<td>0.1</td>
<td>2531</td>
<td></td>
</tr>
</tbody>
</table>
Table 4-18 Average Residual Strength for Trial 2 - Specimen 3

<table>
<thead>
<tr>
<th>Average Residual Strength Test 3</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Displacement, in</td>
<td>Load, lbs</td>
</tr>
<tr>
<td>0.025</td>
<td>288</td>
</tr>
<tr>
<td>0.05</td>
<td>647</td>
</tr>
<tr>
<td>0.075</td>
<td>661</td>
</tr>
<tr>
<td>0.1</td>
<td>856</td>
</tr>
<tr>
<td>ARS, psi</td>
<td>134</td>
</tr>
</tbody>
</table>

Table 4-19 Average Residual Strength for Trial 2 - Specimen 4

<table>
<thead>
<tr>
<th>Average Residual Strength Test 4</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Displacement, in</td>
<td>Load, lbs</td>
</tr>
<tr>
<td>0.025</td>
<td>1510</td>
</tr>
<tr>
<td>0.05</td>
<td>1596</td>
</tr>
<tr>
<td>0.075</td>
<td>1346</td>
</tr>
<tr>
<td>0.1</td>
<td>1017</td>
</tr>
<tr>
<td>ARS, psi</td>
<td>300</td>
</tr>
</tbody>
</table>

Table 4-20 Average Residual Strength for Trial 2 - Specimen 5

<table>
<thead>
<tr>
<th>Average Residual Strength Test 5</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Displacement, in</td>
<td>Load, lbs</td>
</tr>
<tr>
<td>0.025</td>
<td>1565</td>
</tr>
<tr>
<td>0.05</td>
<td>1528</td>
</tr>
<tr>
<td>0.075</td>
<td>1546</td>
</tr>
<tr>
<td>0.1</td>
<td>1274</td>
</tr>
<tr>
<td>ARS, psi</td>
<td>324</td>
</tr>
</tbody>
</table>

Table 4-21 Average Residual Strengths for Trial 2

<table>
<thead>
<tr>
<th>Average Residual Strengths for Trial 2</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen</td>
<td>ARS (psi)</td>
</tr>
<tr>
<td>1</td>
<td>514</td>
</tr>
<tr>
<td>2</td>
<td>494</td>
</tr>
<tr>
<td>3</td>
<td>134</td>
</tr>
<tr>
<td>4</td>
<td>300</td>
</tr>
<tr>
<td>5</td>
<td>324</td>
</tr>
</tbody>
</table>

The Average Residual Strengths for Trial 2 are summarized in Table 4-21. With the exception of Specimen 3 (where slippage occurred), the results are quite uniform. Granted there was some variation in the mix process that would explain some of the difference in values; most of this disparity, however, probably arose from the different test setups -
primarily the instrumentation, the load application, and data acquisition system. ASTM C1399 dictates an acceptable range of 1.49 MPa (216 psi) between maximum and minimum results for three results; the difference between the high and low values for this set of tests (excluding specimen 3) is 214 psi. However, the standard also states that the results should have a standard deviation of 0.45 MPa (65 psi) [2] or less, which is not found with these test results even with the exclusion of test three. Furthermore, Forta documents their average residual strength test values ranging from 250 psi up to 300 psi with higher fiber dosages [3]. Some of the values obtained during this second set of tests were in this range. With further testing and comparison, it is expected that better results for average residual strength could be achieved and eventually used for design purposes.

Altogether the benefits of fiber reinforcement were demonstrated throughout these tests. The fibers allowed the concrete to carry post-cracking loads. It was evident from the test specimens that the fibers successfully bridged the gap of the crack and resisted further cracking. Although specific post-cracking load bearing capacities were not determined due to the variation in test results, it was evident that the fibers provided some residual strength.

4.2.iii.b Average Residual Strength Tests of 2 in. x 2 in. x 14 in. FRC Beams

In order to provide a comparison between dynamic and quasi-static test results, the average residual strength (ARS) testing procedure was conducted on a set of smaller beams – 2 in. x 2 in. x 14 in. as opposed to the 4 in. x 4 in. x 14 in. beams used in previous tests. The reason for reducing the size of the beams was to accommodate the limitations of the dynamic apparatus. The quasi-static test setup then was significantly scaled down to accommodate the smaller beam size. Figure 4-16 through Figure 4-21 depict the load vs. deflection data for six smaller beams.
Residual Strength Test 1 (2x2x14)

![Graph showing Load vs. Displacement for Trial 3, Specimen 1](image1)

Figure 4-16 Average Residual Strength Trial 3 - Specimen 1

Residual Strength Test 2 (2x2x14)

![Graph showing Load vs. Displacement for Trial 3, Specimen 2](image2)

Figure 4-17 Average Residual Strength Trial 3 - Specimen 2
Figure 4-18 Average Residual Strength Trial 3 - Specimen 3

Figure 4-19 Average Residual Strength Trial 3 - Specimen 4
Figure 4-20  Average Residual Strength Trial 3 - Specimen 5

Figure 4-21  Average Residual Strength Trial 3 - Specimen 6
These test results seemed reasonable; however there are some observations of the test results and behavior that should be noted. There was no standardized test for average residual strength of the smaller 2 in. x 2 in. x 14 in. beams; therefore there was no prescribed load rate. Application of load was conducted through trial and error. The loading rate played a significant role in the success of these tests. For test 1, the peak of the initial step seems much lower than the rest of the tests. The load was applied too quickly and the peak load recorded was probably not the maximum that was carried by the beam, but rather only the maximum load observed after cracking. In tests 2 and 4, it appears that the reloading stage suffered the same fate. The load was applied too quickly for intermediate deflections to be observed and therefore the associated intermediate loads were not recorded. However, the tests seemed to display the expected behavior of the final loading curve stiffness being lower than that of the initial loading curve. The average residual strength values obtained for each test are displayed in Table 4-22. Since there was no real consistency in deflection data, the average residual strength was calculated from the average of all load points taken in the final loading step.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>ARS, psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>155</td>
</tr>
<tr>
<td>2</td>
<td>40</td>
</tr>
<tr>
<td>3</td>
<td>104</td>
</tr>
<tr>
<td>4</td>
<td>48</td>
</tr>
<tr>
<td>5</td>
<td>153</td>
</tr>
<tr>
<td>6</td>
<td>232</td>
</tr>
</tbody>
</table>

The 2 in. x 2 in. x 14 in. beam tests results had considerable variation; ranging from as low as 40 psi to as high as 232 psi. The low end of the spectrum is similar to the first set of results for the larger beams while the higher end are closer to those of the second set of
larger beams. The small beam tests that had higher average residual strength values were also the tests that had better test results based on reloading curve appearance.

Due to the fact that there was no prescribed loading rate for this size of beam, the first beam was used to get a feel for what would be appropriate. Unfortunately, the first test was loaded too fast. The load that is shown for the initial loading step is most likely lower than the maximum value the supported beam could carry but unfortunately, the beam cracked too quickly and the maximum load was lost. For the most part, subsequent tests were loaded more slowly and therefore saw much high initial peak loads. However, the reloading curve of test 4 appears to have been loaded too quickly as well; shown by the lack of any significant load being carried in the reloading stage. Although the loads for this test were expected to be fairly low, this test seems to be highly load rate sensitive. This is one risk that is taken in using a manually applied load. While the loads and deflections were monitored closely and recorded at each step, it would be beneficial to use a digital data acquisition system for this test setup as well. Having a slightly higher sampling rate for loads and deflections would undoubtedly make this test more successful.

Again, some post-cracking behavior was observed with the smaller 2 in. x 2 in. x 14 in. beams, but conclusive quantitative results could not be determined with the range in results. However, even with the smaller beams and size effects typically implicit with concrete, the fibers did display some benefit to a residual strength. The application of the average residual strength test on these smaller beams is in need of review due to the instrumentation and load application issues, but smaller beams do display the post-cracking behavior, which advocates the use of fiber reinforcement.

4.2.iv Drop Weight Tests

As described in Chapter 3, the setup for the drop weight tests consisted of a four inch diameter weight guided by a four inch diameter PVC pipe. The pipe was bracketed to an aluminum frame by four supports evenly spaced at 13.5 inches. The mass was manually
hoisted through a simple pulley system and anchored until the time to drop. When it came
time for each drop, the data acquisition system was triggered in three steps. First, the digital
camera was set to record images to capture the impact. The next step was to run the
oscilloscope, used to measure accelerations as voltages, which could collect ten seconds
worth of data. Finally, because it only ran for three seconds, the load cell data acquisition,
implemented through LabVIEW, was triggered. Following the drop, the acceleration data,
the load cell data, and the images relevant to the impact were saved.

The drop weight tests were performed in two sets. The first set of tests was subjected
to the full mass of the drop weight while the second set of tests was subjected to half of the
weight. For the most part, the drop weight broke through the entire beam specimen for both
groups, but there were some specimens that resisted multiple drops from the smaller weight.
This test was an iterative process. Each step corrected an issue from the test before it. The
adjustments ranged from simply changing the drop height to eventually changing the drop
weight as well. In order to observe the energy absorption of the specimens, it was desired
that a specimen resist a drop while developing a significant crack. By developing a crack, it
would be clear that the fiber reinforcement absorbed some of the impact. The idea was to
take the deflection of the beam after an impact and multiply by the force of the impact. The
initial drop height was selected arbitrarily while keeping in mind the goal of achieving
multiple blows on a beam. The initial height was 13.875 in.; however, the weight broke right
through the beam. The height was adjusted without knowing how much load each beam
could withstand. The next test had a drop weight of 10.75 in., but again the beam was broken
in a single blow. At 7.25 in. the weight again broke straight through, and again at 2.175 in.
After lowering the height down to 0.875 in. and still not absorbing an impact, it was decided
that the weight should be reduced.

For the second set of tests, the drop weight was lowered to 13.75 lbs. Because the
weight was cut in half, it was thought that the final height used at the end of the first trials
would not be able to crack the beam. For the second set of tests, the drop heights began at
12.75 in, but the beam was again broken in a single drop. The heights were lowered much
faster for these tests. It was not until 3.125 in. that a specimen resisted an impact. The drop height was lowered to 3 in. for the next two drops. On the first, test T2-5, the beam underwent three drops. However, the load cell, for the two drops that failed to break through the beam, failed to acquire any data. The second specimen with a drop height of 3 in., T2-6, withstood eight blows from the drop weight. Prior to discovering that T2-6 actually had a higher concentration of fiber reinforcement in the center of the beam, the drop height was raised slightly because the beam took several drops before developing sizable a crack. Unfortunately, the final height of 3.625 in. was still too high for the specimens to resist multiple drops.

In addition to the issues with the experimental test setup discussed in Chapter 3 and issues with the testing process discussed above, occasional issues presented themselves in the data acquisition system, specifically the load cell. Timing delays and instrument saturation occurred during testing. More common in earlier tests, load data was not always collected. It was found that this typically occurred as a result of timing issues. If the data acquisition software was triggered to start too close to the time when the weight was dropped, the load cell data only reported noise and did not see any spike on impact because the processing time between triggering the software and actually starting the acquisition. It was apparent that there needed to be a delay between triggering the DAQ and dropping the weight. Another issue to note was that the load cells typically indicated a value ranging from -28 to -35 lbs. This was the offset zero for the load cell. Furthermore, keeping in mind that the load cell used for this test had a two kip capacity; some values obtained were out of the range of the load cell and were reported as 2245 lbs. These values are assumed to have saturated the load cell.

As for other instrumentation, acceleration results were obtained by converting voltages recorded by the accelerometer - placed on the center of the beam - into accelerations using the accelerometer’s voltage sensitivity of 10 mV/g. The accelerometer had a capacity of 500 g or 5 volts. Most of the values ranged from as low as 0.19 volts to as high as 3.67 volts, save specimen nine which saturated the accelerometer at 500 g.
A total of sixteen 2 in. x 2 in. x 14 in. beam specimens were drop weight tested for impact behavior and the data for each is shown in Table 4-23. The designator distinguishes the test series as well as the test weight. Designators beginning with T1 were tested with the full mass while T2 specimens were only subjected to half the weight. The following summary of results, in Table 4-23, details which weight – full or half – the tests were subjected to, the height the weight was dropped from, how many drops it took for the beam to completely break, and any load or acceleration data that was obtained. The loads that did not max out the load cells ranged from 965 to 1829 lbs and the recorded accelerations range from 52 to 367 g (T2-2 saturated the accelerometer). There does not seem to be a straightforward trend between the load and acceleration data.

Table 4-23 Summary of Drop Weight Tests

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Beam Designator</th>
<th>Drop Weight (lbs)</th>
<th>Drop Height (in)</th>
<th>Drops</th>
<th>Max Accel. (g)</th>
<th>Max Load (lbs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>T1-1</td>
<td>27.5</td>
<td>13.875</td>
<td>1</td>
<td>235</td>
<td>2245</td>
</tr>
<tr>
<td>2</td>
<td>T1-2</td>
<td>27.5</td>
<td>10.75</td>
<td>1</td>
<td>52</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>T1-3</td>
<td>27.5</td>
<td>7.25</td>
<td>1</td>
<td>291</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>T1-4</td>
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<td>3.625</td>
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The following graphs show the relevant load and acceleration data for different tests. Some of the data is excluded, because either the load or acceleration data was not recorded by the acquisition system. Some of the graphs show saturated data, but the important thing to note is the corresponding peaks of accelerations and loads. The time scales have been adjusted to the same size for the load and acceleration graphs.

![Load Graph for T1-5](image1)

**Figure 4-22 Load Graph for T1-5**

![Acceleration Graph for T1-5](image2)

**Figure 4-23 Acceleration Graph for T1-5**
Figure 4-24 Load Graph for T1-7

Figure 4-25 Acceleration Graph for T1-7
Figure 4-26 Load Graph for T2-3

Figure 4-27 Acceleration Graph for T2-3
Figure 4-28 Load Graph for T2-4, drop 2

Figure 4-29 Acceleration Graph for T2-4, drop 2
Figure 4-30 Load Graph for T2-5, drop 2

Figure 4-31 Acceleration Graph for T2-5, drop 2
Figure 4-32 Load Graph for T2-6, drops 1, 2, and 3

Figure 4-33 Acceleration Graph for T2-6, drops 1, 2, and 3
Figure 4-34 Load Graph for T2-8

Figure 4-35 Acceleration Graph for T2-8
It is important to note the difference in results between the fiber contents among the test specimens. Following the tests, the beams were inspected to see how the fibers failed—where they pulled out, stretched or ruptured. Most beams showed a combination of these fracture modes. The beams were also inspected to determine what kind of fiber content was present. This value was not monitored until after test T2-6. Unlike any other test specimen,
this test presented an interesting anomaly, as it took a total of eight drops before complete fracture, compared to 1 to 3 drops for all other beams. The clear reason for this was the fiber concentration in the beam shown in Figure 4-38; this beam did not have the same fiber distribution as the other beams. A large clump of fibers was located in the middle of the beam, spanning the cracked section. With this beam’s high resistance to the drop weight, the concrete seemed to spall around the cracked section as more and more drops occurred. Rather than a single crack at failure, multiple cracks formed as the concrete began to break apart.

![High Fiber Content Beam After Fracture](image)

Figure 4-38 High Fiber Content Beam After Fracture

After this test, all earlier and later test samples were examined for fiber content and distribution at the impact point. It was found that with the exception of T2-5, the fiber content was relatively consistent for the other samples. Test T2-5 seemed to have a slightly higher content than the majority of beams, but still not as large of a bunch as evidenced in T2-6. A photo of fractured beams is shown in Figure 4-39 with the highest fiber content on the left and lowest on the right.
Although the results were not consistent enough to draw firm conclusions, this test managed to identify and work out possible kinks in the test methodology and instrumentation providing a useful framework for future tests. The benefit of fiber reinforced concrete under impact loads was best characterized in the behavior of test T2-6. While the fiber content in this beam sample was unintentionally high, the beam allowed for fiber reinforcement behavior to be exaggerated. The beam absorbed several impacts even after cracking occurred on the second strike and developed into a significant crack on the third strike. This beam continued to withstand blows several more times. As a result of T2-6, a look into different fiber content ratios should allow greater understanding of fiber reinforced concrete.

Furthermore, there was some difficulty determining the appropriate drop height to allow for the beam to crack, but not completely break. Once this point is determined, subsequent tests can be conducted to allow actual energy absorption data, not just observations, to be collected and analyzed which will be useful for future modeling and design. Whether or not it is best to conduct the drop weight test on the 2 in. x 2 in. x 14 in. sized beams or on larger ones is yet to be seen, however the larger beams might withstand enough impact force to break without complete fracture, allowing for additional data.
collection. Further testing would also allow for a better understanding of the properties, and more conclusive results of the impact resistance of fiber reinforced concrete.

4.3 Conclusion

While this part of the study did not achieve the overall objective of this research project, which is to develop a model to use as an analytical tool allowing for the design of fiber reinforced concrete, this phase of the project did take the necessary first steps to accomplish this goal. Each step taken during this study, while not without hiccups, will assist in the development of the analytic model by developing experimental procedures for future testing and providing some comparative results. The aims of the study were to work through the early difficulties of a steep learning curve and develop testing procedures to support later work, specifically:

1. Create a functional fiber reinforced concrete mix by determining the appropriate batch proportions and formulating a reliable mixing process.
2. Conduct splitting tensile strength, compressive strength, and average residual strength tests based on ASTM standards.
3. Develop a test setup for a drop weight impact experiment.
4. Calibrate the drop weight experiment by determining the appropriate drop height and weight as well as implementing the data acquisition instrumentation and software.

Each of these goals proved to be an iterative process. It took some practice to successfully obtain a mixing process that produced acceptable concrete. However, it was not the mixing process alone that needed refinement, but also the batch proportions. Although promoted by fiber manufacturers as having the capability of being added to any mix, fibers require mixes with high workability. Early attempts at mixing failed due to low workability attributed to large coarse aggregate and low mortar content. An acceptable mix was achieved
when the size of the coarse aggregate was lowered and batch proportions were adjusted to have a higher fine aggregate to coarse aggregate ratio, higher water content, and superplasticizer admixture, all of which contributed to higher workability. Additionally, mixing was moved to the laboratory to better monitor the process rather than obtaining concrete from an outside source.

Furthermore, this study conducted a number of static tests in order to better understand the material properties of fiber reinforced concrete. ASTM standardized procedures for compression strength, tensile strength, and residual strength were preformed on fiber reinforced specimens. While the results for the splitting tensile strength tests were well within the ASTM prescribed acceptable limits of variation, the compression strength test ranged from well below to 0.1% above the ASTM acceptable limits of variation. Overall the results for these two experiments seemed reasonable.

Once the issues with the average residual strength test setup were resolved, fairly reasonable results were obtained. While the numerical results could be improved, the residual strength behavior was clearly documented. There remain some questions about the stiffness characteristics of some of the tests; particularly how some of the final loading steps had higher stiffness than the initial loading steps. This seems counter intuitive because the final loading step is performed on a cracked section.

Finally, the impact experiment required the most attention. Since, there is no standardized method for impact tests on fiber reinforced concrete, a test setup had to be developed. However, existing research from the University of British Columbia by Dr. Sidney Mindess was a useful guide in the development of a guided drop weight system. Although no conclusive numerical results could be drawn from the experiment developed for this project, fiber reinforced concrete behavior under impact was demonstrated. Furthermore, any issues that arose with the setup, instrumentation, or results will help improve the drop weight test experiment for future work.

Overall, it could be said that the first phase of this larger, multi-phase project on fiber reinforced concrete, repeated below, was a success.
- Phase 1: Basic understanding of fiber reinforced concrete:
  - FRC batch proportioning and mixing processes.
  - Behavior under static tests such as Splitting Tensile, Compressive, and Average Residual Strength experiments.
  - Calibration of impact tests and observations of results.

4.4 References


CHAPTER 5: RECOMMENDATIONS

This project marked the beginning of fiber reinforced concrete research on North Carolina State University's campus. Although there has been previous impact testing performed at NCSU, it was limited to the impact characteristics of glass plates. Consequently, this project broke new ground and came with a steep learning curve. Although progress was made in understanding fiber reinforced concrete, there is much more work to be done to learn about the impact behavior FRC beams.

In a manner of speaking, this project evolved at a deliberate pace with each step of the process – from material preparation to determining testing methodologies – becoming a learning experience in itself. Hence, the project contributed a useful methodology for calibrating future testing and modeling, and identified areas for additional research to increase overall comprehension of FRC beams. Before continuing, recommendations need to be made in two distinctive areas. The first recommendations are practical in nature and address the test procedure and test setup. These recommendations address errors made while testing and from observed limitations as a result of the process that could not be rectified in a timely manner. The second set of recommendations proposes additional research phases for future work.

This project was the first step of a broader work and the following recommendations lay out a framework for the follow-on phases. While the final picture may not be clear at the moment, an expansion of this project could confirm potential benefits of FRC.

5.1 Recommendations for Experimental Setups and Procedures

While some of the tests seemed unsuccessful due to the lack of conclusive numerical results, these tests provided useful insights for future project phases particularly by determining faults in the experimental process and identifying improvements in the setups,
procedures, or instrumentation. It is fitting to start at the beginning of the experimental procedures when discussing the recommendations for procedural changes.

First, fiber reinforcement adds additional variability to a material that already lacks uniformity; concrete. The material properties of concrete vary depending on a wide range of components including aggregates size and shape, water, and cement proportions as well as mixing processes. Slight adjustments in these properties can adjust the tensile, compression, and undoubtedly the average residual strengths of the concrete. The lack of homogeneity is generally accepted along with the fact that concrete from a single batch will have small variations in material properties. Furthermore, since individual batches can differ slightly in properties, for design purposes concrete properties are typically accepted to be similar between batches with the same proportions and mixing process. Fibers, unlike conventional steel reinforcement, are scattered throughout the concrete mix much like the aggregate and therefore the chances for variability increases within the concrete section. The aim as with any concrete mix is to minimize this variability by maintaining prescribed proportions and mixing techniques.

After several attempts at adjusting both the batch proportions and the mixing process, a good mix and mixing process has been established for this project. General proportions provided by Forta, smaller coarse aggregate, higher water-cement ratio, and premixing aggregates with the fibers seemed to be the key to finding a workable mix. It is recommended that the following mix guidelines be followed for future mixes.

1. Moisten the mixing drum.
2. Mix coarse and fine aggregates until distributed while assuring that all materials are thoroughly mixed and not clumping on mixing blades or around the drum.
3. Add fibers into the mix in small increments. Mix fibers into the premixed aggregates for an extended period of time. Be mindful to avoid fiber hair balling and clumping by scrapping mixing blades and the mixing drum.
4. Premix water and superplasticizer.
5. Add cement and about ¾ of water to the mix.
6. Add remaining water slowly as drum begins to rotate.
7. Scrape mixing blades and the mixing drum to check for any raw material clumping.

Granted the mix proportions and types of aggregates used will also affect the workability of the concrete mix to a degree. Use of these guidelines should help ensure higher workability. Furthermore, it is recommended that a smaller coarse aggregate should be used. Aggregate sizes of \( \frac{1}{2} \) in. or higher can affect the workability and hinder the distribution of fibers.

In addition to the mixing process, one recommendation would be to fabricate a plain concrete batch with the same mix proportions minus the fiber reinforcement for comparison. There are certain properties of fiber reinforced concrete such as the post-cracking load bearing capacities that are definitely an improvement over plain concrete, but it would be worth observing the differences in material properties such as compressive and tensile strengths between the FRC and plain concretes for modeling purposes. While the main objective of this study concerns the post-cracking behavior of fiber reinforced concrete, comparative results would be beneficial for modeling as well as be able to conclusively determine the benefits of fiber reinforcement for all concrete properties. Specifically, the benefits of fibers were observed in splitting tensile strength and average residual strength. The benefits were not so clear in the compressive strength tests.

Additionally, the benefits of fibers were observed during impact testing, but mainly at extreme concentrations. Quantifying the effect of fiber content on beam behavior was not as clearly demonstrated during impact loading due to the fact that the drop weight broke through in a single blow. Therefore, as an extension of the recommendation to fabricate plain concrete, fabrication of concrete batches with various fiber contents is suggested. Various fiber contents from 0.0% to as high as 1.5% will allow for a better understanding of the behavior fiber reinforcement under impact loading. Fiber contents higher than 1.5% might not be practical for mixing purposes; workability may suffer and fibers may clump.
As for specific experimental procedures, with the exception of the drop weight tests and small beam average residual strength tests, most kinks and failures were worked out during the testing procedures. As a note, necessary changes to the experiments as learned through this research project include monitoring and recording the first cracking load for splitting tensile tests and closer observation of the comparative stiffness of the initial and final loading steps of the average residual strength tests on full size (4 in. by 4 in. by 14 in.) beams.

With regard to the small beam (2 in. x 2 in. x 14 in.) average residual strength tests, the implementation of a digital data acquisition system would be extremely beneficial. With the size of the specimen and the post cracking behavior of fiber reinforced concrete, a slightly higher data acquisition rate should have been used to better monitor the load and deflection. The tests were initially thought to have small enough load-deflection changes to be recorded using an analog system; however, a higher sampling rate is needed as some load-deflection data was lost during the testing process. Correlating with the data acquisition suggestion is a recommendation to use a digitally controlled actuator. While an increased sampling rate should cover the lost load-deflection data and the manual screw actuator should be sufficient, a digitally controlled actuator would provide further reliability in this test setup.

The greatest need for improvement is the drop weight test setup, specifically in the data acquisition system. Two issues arose with the data acquisition used for the drop weight tests performed for this project: timing the start of the digital acquisition and the load cell. The tests performed during this project did not use a single trigger for acquisition but rather three separate ones. The procedure for this experiment was to start recording digital images, then acceleration data, and finally load cell data then drop the weight. The load cell had the shortest acquisition length and therefore was triggered to record last. However, since the sampling time was so short there was a chance to miss the drop load data. With the three triggers an issue with matching time scales arose and could easily be remedied by using one DAQ program with a single trigger. This would ensure that all devices start at the same time.
and record all necessary data. It is suggested that a subroutine be written into LabVIEW to collect data for both the accelerometer and load cell while triggering the camera for digital imaging. The data acquisition for the load cell is already being run through LabVIEW and the digital camera has the capability of being controlled through the program. However, it is currently unknown how to capture accelerometer data through LabVIEW. Along with changing the triggering system, it is recommended that a different load cell be utilized. The load cell used for this project was not rated for impact or shock loads and had a load capacity of two kips. Ideally, a non-shock rated load cell should be subjected to no more than half of the load capacity under impact loading. From the data, the loads that were observed were typically over one kip and there were even some loads that maxed out the instrument. Obtaining a shock rated load cell will help ensure better data acquisition for loads which along with deflections will be necessary for determining energy absorptions.

Furthermore, with respect to the drop weight tests, the full potential of the camera is not being realized. The camera has the ability along with the digital imaging software to gather deflection data. National Instruments’ Vision Assistant has the ability to track displacements by calibrating a grid of known spacing and relating that spacing to pixels. Images of an impact are then compared to the calibrated grid and displacements can be determined. Vision Assistant uses shape recognition to track the displacement. Ideally, the image recognition software would track a beam edge by monitoring changes of the start and end coordinates of the edge. However, with respect to this study, after reviewing the test specimens that withstood multiple drops there was no clear tracking edge to determine deflection data. The images were analyzed with shape detection, yet the edges detected in initial images were not consistently tracked throughout the subsequent images. Moreover, the edges were lost in some images and therefore the displacement tracking was incomplete. To adjust for the inconsistent edge detection of the beam specimens, it is recommended that a tracking image be placed on the specimens themselves that can be monitored through the tracking software. A small, reflective, circular sticker may be the best option. A reflective marker will help ensure tracking of the sticker while size and shape will be monitored.
through the shape recognition software. Granted the placement of the sticker relative to the crack will vary with each test, but tracking midspan deflection with the tracking software should provide useful information.

At this point in testing, the above recommendations for testing procedures are summarized below:

- Adhere to established mixing process for small scale concrete batches.
- Vary the fiber content in future batches to range from 0.0% to 1.5%.
- Monitor and record first crack loads for splitting tensile tests.
- Observe relative stiffness of initial and final loading steps for average residual strength tests.
- Implement digital data acquisition and load actuator for small beam (2 in. x 2 in. x 14 in.) average residual strength tests.
- Implement a script to start all data acquisition for impact testing with a single trigger.
- Utilize a shock rated load cell with a greater load capacity.
- Utilize digital camera for deflection tracking through shape recognition of reflective sticker.

5.2 Suggested Implementation and Recommendations for Future Work

As previously stated, the study of fiber reinforced concrete under dynamic loading is in the early stages of development. Fundamental understanding of fiber reinforced concrete properties is the first phase of a much broader, multi-phase research project. This study used a single fiber type and dosage to aid in the understanding of fiber reinforced concrete properties such as compressive, splitting tensile, and average residual strengths. Furthermore, this project helped calibrate impact experimentation and provided a basic level of understanding how FRC behaves under impact loading. While a general understanding of fiber reinforced concrete has been obtained, more in-depth knowledge of this material is
required. Variations in types of fibers, fiber dosage and types of impact loading can all be adjusted to further the progression of this research project and provide additional understanding.

The mostly likely course of action following this particular study in the research project would be to continue with experiments. The next step should be to vary fiber contents during testing to better understand quantitatively how fiber reinforcement benefits post-cracking behavior, both in dynamic and quasi-static situations. The focus should be on further development of the drop weight test. Refining the drop weight and height to be able to successfully test energy absorption will be crucial to developing a computer model. However, even though the quasi-static experimentation was fairly straight forward, it is recommended that ASTM C1399 continue to be used for all specimens and dosages. Continuation of these tests will provide more reliable evaluation of results. As these tests start to show reasonably consistent results, models should be developed and compared to actual results.

Specific changes to experimental set ups are documented earlier in this chapter. These recommendations list failures in testing procedures and setup in addition to where results went wrong during the process of this study. Although, further trials will be needed, hopefully this study has worked through the majority of issues with the experimental side of this project and has presented a means to achieve the ultimate goal for this fiber reinforced concrete project.

The following layout shows a proposed guide to fiber reinforced concrete research at North Carolina State University:

- Phase 1: Basic understanding of fiber reinforced concrete:
  - FRC batch proportioning and mixing processes.
  - Behavior under static tests such as Splitting Tensile, Compressive, and Average Residual Strength experiments.
  - Calibration of impact tests and observations of results.
- Phase 2: Further experimentation and observations of fiber reinforced concrete:
- Adjust fiber dosage by percent volume to observe variation in behavior.
- Possible change in fiber type to include other synthetic fibers.
- Continued impacting testing utilizing recommendations from Phase 1 experimentation.

- Phase 3: Modeling fiber reinforced concrete behavior:
  - Utilize static test results to model Splitting Tensile Strength Test for benchmarking modeling software as well as other static experimentation.

- Phase 4: Modeling fiber reinforced concrete behavior under dynamic loads:
  - Correlate with experimental results to assist in future fiber reinforced concrete design.