NDOT Research Report

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Development of Specifications for High-Performance Fiber Concrete for Nevada

July 2020

Nevada Department of Transportation
1263 South Stewart Street
Carson City, NV 89712

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16. Abstract

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ABSTRACT

The project stated with an extensive summary of construction practices and experiences from different state agencies. Besides, a detailed summary of state DOT specifications was included in the report. The main objective of this study was to develop a mix design adjustment method for High-Performance Fiber Reinforced Concrete (FRC) that would maintain appropriate workability while improving hardened concrete performance. A literature review was conducted to examine existing methods for adjusting mix designs to account for fiber introduction. It was found that while increasing fine aggregate and cement paste content can make up for lost workability with the addition of fibers, no rational mix design adjustment method is available. Reference mix designs from the Nevada Department of Transportation and the Nebraska Department of Transportation were used, and this study focused on tailoring the mix design based on the parameter of excess paste. Excess paste serves to coat the aggregate particles and is critical for workability. To apply this method, a modified version of ASTM C29 was used to determine the void content of fiber-aggregate skeletons with varying fiber contents. Paste and fine aggregate content were then adjusted to maintain the excess paste quantity between reference mixes and mixes with fiber. A variety of tests, including slump, vibrated L-box, compressive strength, splitting tensile strength, flexural strength, drying shrinkage, and restrained shrinkage were conducted to evaluate the overall concrete performance. Results indicated that, for each mix design, adjusting based on excess paste provided a workable FRC with improved hardened performance. Eight slabs were then prepared for a large-scale examination of the constructability and mechanical behavior of the developed FRC. Throughout the study of FRC, an alternative concrete to Ultra-High Performance Concrete (UHPC) that would considerably outperform High-Performance Concrete (HPC) was developed. This study delves into the development of a new type of concrete called Super High-Performance Concrete (SHPC). SHPC is a high strength, self-consolidating HPFRC that would significantly cut back on cost and production limitations compared to UHPC as it can be produced with conventional drum-type mixers. Results indicate that SHPC outperforms HPC in matters of workability, compressive strength, flexural strength, and toughness and could potentially be a viable alternative of UHPC for applications such as bridge deck connections and overlays. The report also included detailed recommendations regarding the mix design, batching and mixing, quality control methods, and casting of HPFRC and SHPC that can be further used in the development of specifications for NDOT.

EXECUTIVE SUMMARY

Summary of Research Need

Bridge decks are the weakest link among different bridge components and last approximately 30 years. Considering that the design life of a bridge according to the AASHTO LRFD specifications is 75 years (AASHTO 2014), a deck life of 30 years clearly does not meet this criterion. Bridge decks are directly exposed to traffic and chloride elements, but most importantly, these elements can reach reinforcing steel through various forms of cracks, such as restrained shrinkage cracks, which form in the early age of construction. Crack control through traditional bar arrangements may mitigate these cracks, but discontinuous fiber reinforcement can significantly improve this situation by reducing the tensile stresses under restrained conditions.

While the focus of concrete mixtures has primarily been on their mechanical properties, high-durability concrete has drawn more attention in recent years in order to improve the serviceability of structures. By using high-performance fiber-reinforced concrete (HPFRC), for bridge decks and connections (longitudinal and transverse), the service life of a bridge deck can be significantly improved by reducing premature distresses associated with various forms of cracks. Nevada Department of Transportation (NDOT) could also benefit from the longer service life in other applications such as concrete pavement, which will visit similar conditions. Research is needed to determine the most effective applications of HPFRC for NDOT to optimize both structural performance and economic efficiency and to investigate if HPFC can be produced and placed with locally available materials and commonly used batching and placing methods. The fiber types, dimensions, dosage, design requirements, and material and construction specifications need to be developed to make HPFRC more efficient and practical for the application in Nevada.

The goal of this research is to determine the potential use of HPFRC with performance-related specifications and to propose a structurally and economically efficient specification suitable for the design of concrete bridge superstructures and pavements for NDOT. The technical objectives of this research are to:

- Understand current uses, practice, and specifications of HPFRC;
- Develop HPFC mixtures for both bridge deck panel and panel connections with locally available material in Nevada;
- Analyze the cost-effectiveness of the chosen HPFC mixes; and
- Develop HPFC specifications for bridge superstructures using local material from Nevada.

Research Findings

Based on an extensive review of construction practices and experiences from different state agencies, as well as existing methods for adjusting mix designs to account for fiber introduction, a series of experimental studies were performed and below are the major research findings:

- Adjusting a non-fiber reinforced concrete to incorporate fibers using the excess paste method based on maintaining the same excess paste volume is effective in maintaining workability while improving mechanical properties. The excess paste adjustment method was successful for three completely different sets of materials, all of which exhibited satisfactory workability with no visible segregation, significantly improved moduli of rupture, slightly to significantly improved compressive strength, and high toughness.

- As the traditional slump test is not capable of reflecting the true workability of HPFRC, a vibrated L-Box test was developed to evaluate the workability of HPFRC under vibration. The test assesses the flowability, and passing ability under vibration and was able to effectively measure HPFRC workability.
- Construction and performance of lab-scale slabs prepared with the developed HPFRC demonstrated that not only was the developed mix have sufficient workability to ensure appropriate placing, consolidation, and surface finishing in bridge deck construction, the HPFRC slabs also exhibited superior crack resistance compared to conventional bridge deck concrete prepared with the same reinforcement.
- SHPC is a potential alternative to ultra-high performance concrete (UHPC). Not only can the SHPC prepared with conventional drum-type mixers, but it also exhibits a self-consolidating level of workability, high strength, high toughness, strong bond strength, and excellent durability performance. A similar adjustment based on excess paste was used to convert the SHPC mix design to two other sets of materials with very comparable results.
- Full-scale bridge panel connection test justified that the developed SHPC mixes exhibit excellent constructability and mechanical behavior with good ductility that can serve as a cost-effective alternative to UHPC in bridge connection.

Based on the practices and specifications from state agencies, together with the experience from the experimental study included in this project, detailed recommendations regarding the mix design, batching and mixing, quality control methods, and casting of HPFRC and SHPC were included in the report. The information can be further used in the development of specifications for NDOT.

CHAPTER 1. INTRODUCTION

1.1 Background

Concrete is the most commonly used construction material in the world, but its brittle nature, weak behavior in tension, and shrinkage-related cracks can create many issues in the engineering and construction fields. High-Performance Fiber Reinforced Concrete (HPFRC) is a special type of concrete that includes steel or synthetic fibers, which can greatly improve flexural performance, tensile strength, toughness, and crack resistance from a variety of common concrete distresses. HPFRC has the ability to hold concrete cracks formed under flexural load and continue carrying load beyond initial cracking, which is not possible with regular concrete. Furthermore, HPFRC can potentially reduce the necessary amount of traditional steel reinforcement. By using HPFRC for applications such as bridge decks and connections, service life can be significantly improved by reducing premature cracking associated with restrained shrinkage.

While there are many benefits of adding fiber to concrete, developing an appropriate mix design for sufficient workability is always challenging. Fibers cannot simply be introduced into a mix design without any form of adjustment due to their tendency to disrupt the concrete matrix and introduce considerable voids. This often results in a concrete that is severely lacking in paste with an abnormal degree of voids in the hardened state. Likewise, paste cannot simply be added without consideration for materials properties, as this could lead to an overabundance of paste, which results in a non-cohesive and segregated mix. Thus, an adjustment method that can maintain an appropriate degree of workability while providing the aforementioned benefits is necessary.

In terms of fresh, mechanical, and durability related properties, Ultra-High Performance Concrete (UHPC) is the most superior concrete available today. However, UHPC is known for its extremely high cost and difficulties in production in the field. In applications where High-Performance Concrete (HPC) is not adequate, an alternative concrete to UHPC could be highly beneficial. There is a need to develop a new concrete which could be delivered with significantly lower cost and simpler methods of production compared to UHPC, while exhibiting features such as self-consolidation, high compressive strength, high flexural toughness, and improved bonding compared to HPC.

1.2 Research Objectives

The goal of this research is to determine the potential use of HPFRC with performance-related specifications and to propose a structurally and economically efficient specification suitable for the design of concrete bridge superstructures and pavements for NDOT. The technical objectives of this research are to:

- Understand current uses, practice, and specifications of HPFRC;
- Develop HPFRC mixtures for both bridge deck panel and panel connections with locally available material in Nevada;

- Analyze the cost-effectiveness of the chosen HPFRC mixes; and
- Develop HPFC specifications for bridge superstructures using local material from Nevada.

1.3 Report Organization

The proposed study is composed of eight chapters (including this chapter) as summarized below: Chapter 2 consists of a background of fiber reinforced concrete, including fiber types, mix design methods, and general behavior. A literature review for FRC is included as well. Chapter 3 outlines the materials used, mixing procedures, and test methods conducted in the study. Chapter 4 discusses the experimental program and details the screening phase, adjustment phase, and performance evaluation phase with corresponding results during each phase. Chapter 5 focuses on the development of Super High-performance Concrete (SHPC). Chapter 6 involves the details of the lab-scale slab and connection testing and the corresponding results. Chapter 7 provided recommendations for HPFRC practice that could be used in NDOT specifications. Lastly, Chapter 8 summarizes and concludes the report.

CHAPTER 2. BACKGROUND

2.1 Introduction

FRC is generally used in a supplementary role to distribute cracking, improve resistance to impact or dynamic loading, and resist material deterioration. In applications where the presence of continuous conventional reinforcement to resist tensile stresses is not essential, such as in pavements and overlays, the improvement in flexural toughness associated with fibers can be used to reduce section depth, improve performance, or both.

Deterioration in traditional reinforced concrete is a major problem for state agencies. One of the most common deterioration mechanisms is the formation of cracks that lead to the penetration of water and chemicals into concrete, which in turn could initiate or accelerate distresses such as alkali-silica reactivity, reinforcement corrosion, sulfate attacks, and freeze/thaw deteriorations (Ozyildirim, 2005). While reinforcement is used for crack control, cracks may still propagate as a result of the low tensile strength and plastic and drying shrinkage of concrete. One of the major concerns for many DOTs in the United States is early-age bridge deck cracking. The long-term performance and durability of bridges can be significantly affected by the cracks that form within the first few months of the bridge deck's life (Ideker and Banuelos, 2014), which will cause the structure to suffer from deteriorations as well an increase in maintenance costs. As a result, there is a strong need to find a solution for minimizing and controlling crack propagation in bridge decks.

In order to overcome this deficiency of concrete, fiber has been used in the concrete along with conventional reinforcement. Fiber-reinforced concrete (FRC) is a type of concrete that utilizes fibers in order to increase structural integrity and tensile strength. The inclusion of fiber in concrete has many advantages such as reduced plastic shrinkage cracking, improved resistance against impact and abrasion, reduced damages from freeze/thaw attack, and an increase of toughness (Chojnacki, 2000; Ideker and Banuelos, 2014; Ostertag and Blunt, 2008).

2.2 Fiber Types and Categories

There are different types of fibers used in concrete, such as steel, synthetic, glass, and natural fibers. However, there are pros and cons to each of these fibers. For example, steel fibers are far more efficient under flexural loading but can corrode if not appropriately covered with cement paste (Ideker and Banuelos, 2014). This can cause fibers or concrete to break and thus have a reduced ability to control crack width. Furthermore, the corroded fibers can be unsightly to the public. Glass fibers do not corrode, but they can potentially contribute to alkali-silica reactions (Ozyildirim, 2005). For the mentioned reasons, some state Department of Transportation (DOT), such as Virginia DOT, has focused on synthetic fibers (Ozyildrim et al., 1997).

Fibers are manufactured in two size-based categories: macro and micro. Macro fibers are designed to carry a load and are also known as structural fibers (Alhassan and Ashur, 2012). Micro-fibers are generally designed to reduce early age shrinkage cracks and normally have a length between 0.25 in. and 1 in. (Ideker and Banuelos, 2014). However, some DOTs consider fiber shorter than 1.5 in. to be micro-fibers and greater than or equal to 1.5 in. to be macro-fibers (Texas DOT, 2011).

Fibers can be categorized further in many ways. See Figure 2.1 for images corresponding to common fiber distinctions.

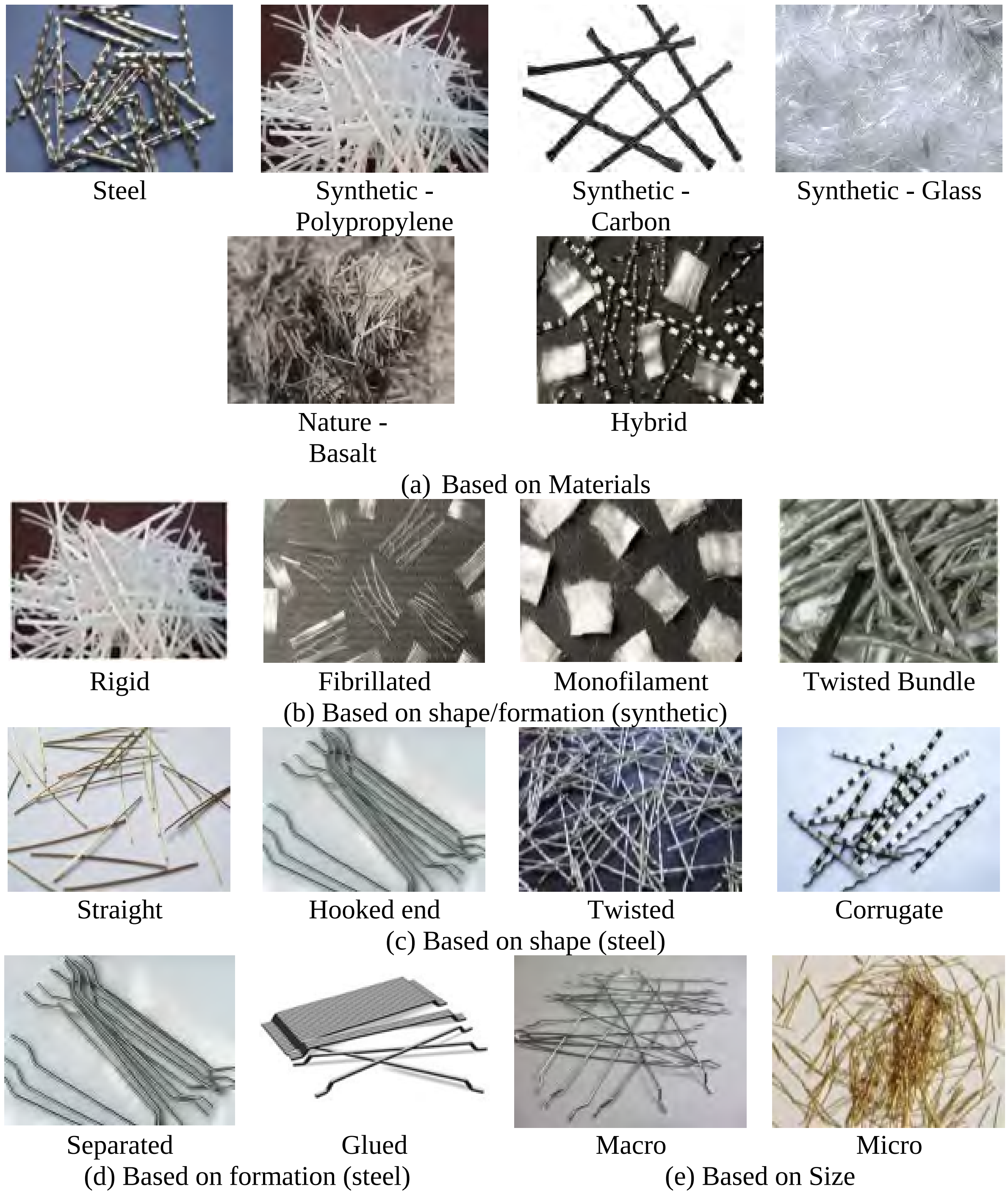


Figure 2.1 Types and categories of fibers.

The technical properties of the fiber characterizations can be seen in Table 2.1. This table illustrates just how much difference there is between each type of fiber. For example, though nylon is relatively high in elongation to failure percentage and lightweight per unit, its modulus of elasticity is very low and thus will lead to a much more rapid increase in deflection after cracking.

Table 2.1 Physical and mechanical properties of different types of fibers.

Types	**Diameter, mil (mm)**	**Specific Gravity**	**Tensile Strength, ksi (MPa)**	**Modulus of Elasticity, ksi (GPa)**	**Elongation to Failure, %**
Cement Paste Matrix	-	2.45	0.44-1.02 (3-7)	1,450-6,500 (10-45)	0.02
Steel	3.94-44.9 (0.1-1.14)	7.85	73-435 (500-3,000)	23,200-30,500 (160-210)	3-4
Polypropylene	0.79-30.37 (0.02-0.77)	0.91	29-110 (200-760)	73-725 (0.5-5)	15-25
Carbon	0.12-0.83 (0.003-0.021)	1.9	261-870 (1,800-6,000)	33,300-87,000 (230-600)	0.5-2
Glass	0.20-0.83 (0.005-0.021)	2.56	290-508 (2,000-3,500)	10,150-12,450 (70-86)	1.5-5.3
Nylon	0.79-15.75 (0.02-0.4)	1.1	110-131 (760-900)	580-595 (4-4.1)	15-20
Basalt	0.24-0.83 (0.006-0.021)	2.7	406-702 (2,800-4,840)	11,500-15,950 (79-110)	3.1-6

Hybrid fibers, as the name implies, are combinations of fibers in a single concrete. There are very few studies on hybrid fiber dosage to this point. A typical combination would be to use micro steel fibers for first-crack strength and ultimate strength improvement while also incorporating macro steel fibers to improve toughness and strain capacity. Another example is to use macro steel fibers for the aforementioned benefits, but in conjunction with synthetic fibers to maintain a desired level of workability. But, as was stated before, there are limited studies on the design of hybrid fiber reinforced concrete and in particular what level of dosage to use for each fiber to maximize results for multiple parameters.

2.3 Mix Design of FRC

The incorporation of fibers will influence both fresh and hardened concrete properties. The inclusion of fibers usually reduces the workability of concrete and makes placing and finishing harder than conventional concrete (Suksawang et al., 2014; Brooks, 2000). As far as hardened properties are concerned, while adding fibers will not change the compressive strength significantly, the flexural strength and toughness will increase (Ideker and Banuelos, 2014; Ozyildirim, 2005; Sprinkel and Ozyildrim, 1999) which is favorable for controlling formation and development of cracks in both early age and long term scenarios.

Mix designs for FRC typically have the following characteristics: high cement content, small maximum size of aggregate, high fine aggregate content, and water-reducing admixtures. The effect of fibers with a larger maximum aggregate size is illustrated in Figure 2.2. The larger the maximum aggregate size, the more likely that fibers will interfere with particle packing. Fibers also tend to interlink when there is a lack of smaller coarse aggregate particles to interfere with their matrix.

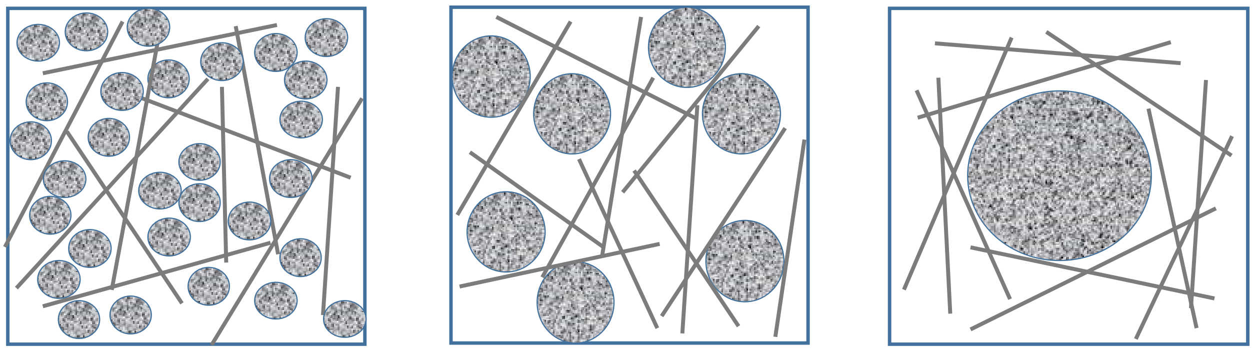


Figure 2.2 Effect of maximum aggregate size on fibers

For a graphical representation of normal concrete versus high-performance concrete, see Figure 2.3. To achieve high strength, durability and/or high workability, HPC usually incorporates supplemental cementitious materials (SCMs). Besides, HPC tends to use a smaller size of coarse aggregate and higher amounts of cement and cementitious materials. As described earlier, with the introduction of macro-fibers, the aggregate skeletons will be interrupted. In other words, coarse aggregate particles are "pushed away" from each other. As a result, a higher amount of sand and cement and cementitious materials are generally needed.

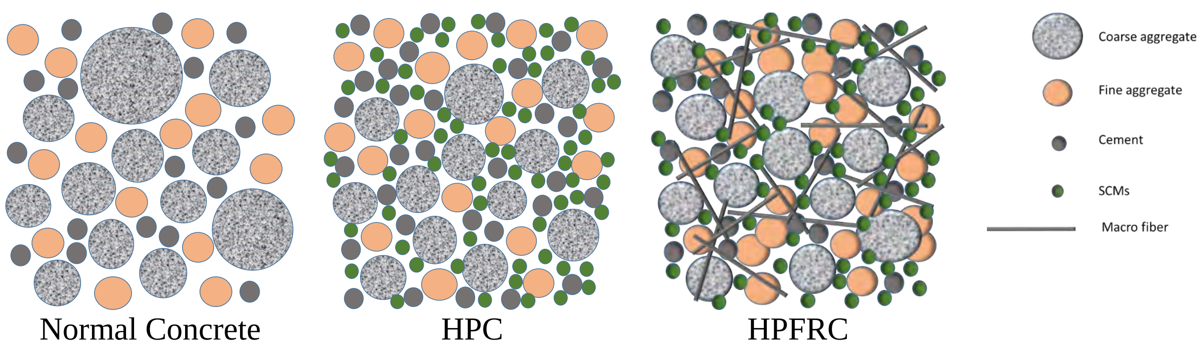


Figure 2.3 Mixture compositions of different classes of concrete.

2.3.1 Empirical Methods

There are very few empirical design methods for fiber reinforced concrete. This is largely due to the wide range of fibers, each of which has a unique effect on workability and flexural performance. However, some practical empirical design methods do exist. For example, a nomogram-based method for steel fiber reinforced concrete was determined in 2018. The quadrants of the nomogram are broken down in Figure 2.4. The user of this nomogram can designate a fiber content and a desired level of workability while selecting one of the following items as the control value: water-to-cement ratio, aggregate-to-cement ratio, flexural strength, toughness, compressive strength, or splitting tensile strength. The nomogram then returns rough expected values for the remaining items (Ulas et al., 2018).

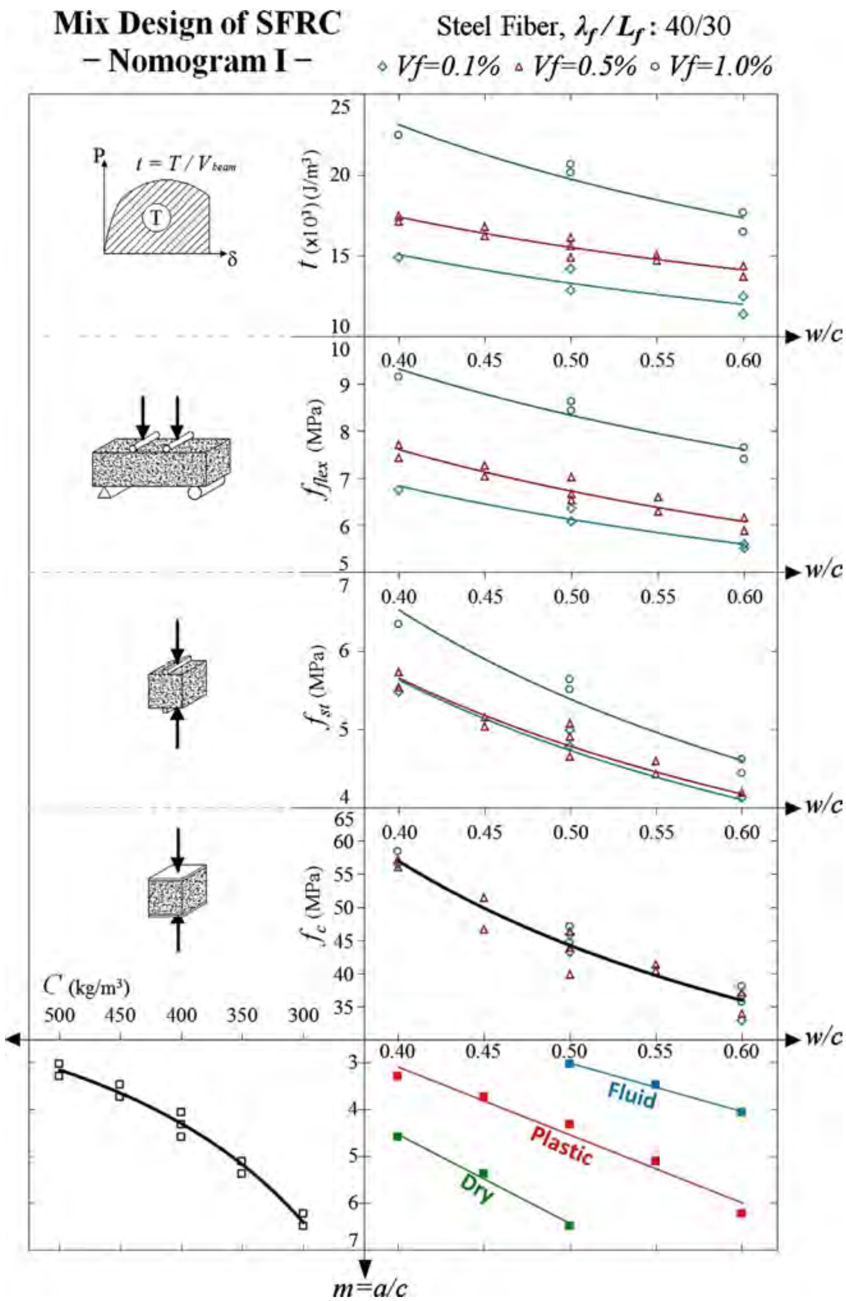


Figure 2.4 Nomorgraph for SFRC Design (adapted from Ulas et al., 2018)

This method is certainly practical but has a few limitations. Firstly, it is not necessarily applicable to all types of steel fibers. As mentioned, one fiber may have a much different impact on workability compared to another. In fact, a separate nomogram is required for different aspect ratios of fiber. Another limitation is that the nomogram does not account for the qualities of the aggregates and does not specify proportions between the aggregates. This is particularly an issue for FRC since fine aggregate content can have a large impact on workability.

In response to the necessary adjustment, some state agencies adopted some empirical practices for concrete design. For example, Florida and Louisiana DOT recommend the addition of superplastizer and a replacement of a particular fraction of coarse aggregate with fine aggregate. Texas DOT specifies a similar practice with coarse and fine aggregates, and their specifications call for the replacement of 100 lbs of coarse aggregate with fine aggregate for their continually reinforced concrete pavement. Both will be discussed in greater detail in Section 2.7.

2.3.2 Scientific Methods

In contrast to empirical design methods, several scientific methods have been presented since the idea of FRC was brought introduced. A common focus of research on the design of FRC was related to the interaction between paste and fibers.

One study by Ferrara et al. (2017) utilized a so-called "equivalent specific surface diameter" to relate a particular fiber to a common aggregate parameter. This idea was built on a rheological design method for self-consolidating concrete (SCC). This "rheology of paste model" for proportioning SCC involved derived theoretical equations for a single spherical particle suspended in a fluid cementitious paste. The average diameter of the solid skeleton particles is calculated with the following equation:

$$d_{av} = \frac{\sum_i d_i m_i}{\sum_i m_i} \tag{2.1}$$

where d_{av} is the average diameter of the particles in the aggregate skeleton, d_i is the average diameter of the aggregate fraction (defined as the average opening size of two consecutive sieves), and m_i is the mass of the associated aggregate fraction (the mass retained at the lower opening sieve). Another equation that uses the average diameter is needed to determine the average aggregate spacing (d_{ss}). This parameter is used to represent the necessary amount and rheological properties of the cement paste that would fill the voids and envelope the aggregate. The equation for the average aggregate spacing is as follows:

$$d_{ss} = d_{av}\left[\sqrt[3]{1 + \frac{V_{paste} - V_{void}}{V_{concrete} - V_{paste}}} - 1\right] \tag{2.2}$$

with the subscripts *paste*, *void*, and *concrete*, applying to the volume of the paste, void, and concrete, respectively. This would conclude the equations used for the design of SCC, but another pair of equations are used when incorporating fibers. These equations serve to relate the fibers to the above equations. Most importantly, an "equivalent diameter of fiber" must be used to calculate the average diameter (d_{av}) that was mentioned in Equation 2.1. This equivalent diameter equation is as follows:

$$d_{eq-fibers} = \frac{3L_f}{1+2\frac{L_f}{d_f}} \frac{\gamma_{fiber}}{\gamma_{aggregate}} \tag{2.3}$$

where L_f and d_f are the length and diameter of the fibers, γ_{fiber} represents the unit weight of the fiber, and $\gamma_{aggregate}$ is the average unit weight of all the aggregate. This can then be worked into the following equation to again determine the d_{av} parameter:

$$d_{av} = \frac{\sum_i d_i m_i + d_{eq-fibers} m_{fibers}}{\sum_i m_i + m_{fibers}} \quad (2.4)$$

where m_{fibers} is the mass of the fiber fraction. With this average diameter, the average diameter spacing can be calculated using Equation 2.2. The average diameter spacing is then applied to Equation 2.2 and used to determine the typical void spacing and thus determine the amount of paste necessary to fill these voids. Though this design method is for self-consolidating fiber reinforced concrete, many of the ideas presented in this mix design method were applied for regular fiber reinforced concrete in Chapter 4. For example, the notion that the void content generated by fiber is the most important parameter and the amount of paste necessary to exceed this void content are two very important principles of the design method applied in this study.

2.4 Test Methods

Quality control tests need to be conducted in order to achieve a consistent product in both fresh and hardened stages. Most of the test methods from the fresh and hardened concrete performance can be used for FRC. There is not a consensus on particular workability or set of workability tests for FRC. However, the inverted slump cone test (ASTM C995) is used by most state DOT's. ASTM C1609 is the standard test for the flexural performance of FRC and is very common to FRC studies. These are the two main unique tests for FRC. Other special test methods from fresh state properties and mechanical properties of FRC can be found in two reports from American Concrete Institute (ACI) Committee 544-Fiber Reinforced Concrete (ACI 544 2017a, 2017b)

2.5 Behavior of FRC

2.5.1 Fresh concrete properties

In typical volume ranges for FRC, the addition of fibers will likely reduce workability. Consolidation with mechanical vibration, in most cases, is necessary. Water reducing admixtures can be beneficial as well. Different fibers have different effects on the fresh behavior of concrete. In a Texas DOT project, it was observed that steel fibers were easier to work with than mixes with macro synthetic fibers while the Iowa DOT reported mixes with synthetic fibers having higher slump values than steel fiber mixes. This furthers the reasoning on why there are so few empirical design methods for FRC.

2.5.2 Mechanical properties

Unlike conventionally-reinforced concrete, in which reinforcing steel is continuous and specifically located in the structure to optimize performance, fibers are discontinuous and generally distributed randomly throughout the concrete mixture. Fibers in fiber-reinforced concrete (FRC) and HPFC can control cracking more effectively due to their tendency to be randomly spaced throughout the entire cross-section. Fibers can control both microcracks and macrocracks that may permit water and chloride contaminants to penetrate and cause corrosion of reinforcing bars (Banthia et al. 2012; Naaman and Reinhardt 2006; Waff 1990).

ACI Committee 544 has published multiple documents regarding the design, construction, physical properties, durability, and measurement of FRC and HPFC (ACI 544 2015, 2010, 1993, 1989, 1988). According to ACI 544 (1996), the introduction of fibers into concrete results in post-elastic property changes that range from subtle to substantial, depending upon a number of factors, including fiber type, fiber length and aspect ratio, fiber strength and modulus, fiber content, fiber surface bonding characteristics, and matrix behavior and aggregate sizes. A low fiber dosage in the range of 0.1% to 0.3% is often provided for control of secondary stresses arising from shrinkage and temperature change (Lawler et al. 2005). At medium dosage rates, the mechanical response of FRC is substantially different from that of the plain matrix due to the post cracking load-carrying ability of FRC. The ability of FRC to absorb energy beyond matrix cracking is often termed toughness. At significantly higher dosages, in addition to post crack toughening, FRCs can also exhibit strain hardening, allowing the composite to support stresses beyond the strength of the matrix (Naaman 2017).

2.5.3 Durability properties

State DOT's often found that Micro fibers improved crack resistance at the micro level. The high tensile strength of macro fibers, however, prevented the widening of cracks. In general, steel macro fibers, and to a lesser extent micro fibers, are far more effective in enhancing the toughness and residual strength of concrete when compared to synthetic fibers.

Many states have conducted research to evaluate the performance of FRC and HPFC in concrete structures, mostly in bridges (Brown et al. 2002, Dhonde et al. 2005, Olek et al. 2001, Ostertag and Blunt 2008, Ozyildirm 2011) and pavement constructions (Folliard et al. 2006, Guirola 2001, Suksawang et al. 2014). Results showed that by adding fibers, concrete can have superior durability that can lead to substantially longer service life. However, the addition of fibers could also cause a reduction in workability and an increased material cost (Hossain et al. 2012, Harding 1996). In recent years, new generations of concrete such as engineered cementitious composite (ECC) and ultra-high performance concrete (UHPC) has also been developed and used in concrete construction. ECC and UHPC generally incorporate high volumes (larger than 5%) of microfibers and with a very large amount of cement and fine particles. Results have shown that these materials can dramatically reduce crack potential and improve structure efficiency (Lepech and Li 2008, Graybeal 2006, Russell and Graybeal 2013). However, due to the very high material cost, they are generally used in smaller components such as panel connections (Hoomes et al. 2014).

2.6 Construction Practice of FRC

2.6.1 Formwork and Reinforcement

There are no notable differences between fiber reinforced concrete formwork and normal concrete formwork. There should be, in the case of a well-designed fiber reinforced concrete, a lower amount of traditional reinforcement. Stiff steel fibers will have a tendency to protrude from concrete edges, but this is generally not a problem.

2.6.2 Mixing

Again, the equipment and mixing method of conventional concrete is not typically modified in any way for fiber reinforced concrete. With steel fibers, it is common practice to load the fibers into the mixing truck directly. Another practice has been to use a conveyor belt to load fibers into the mixing truck so as to increase efficiency and maintain a constant stream of fibers for better dispersion. Oregon DOT loads fibers in this manner. As for synthetic fibers, given that the specific gravity is close to that of water, they can be mixed with the water and incorporated into the mixing procedure as normal. Texas DOT handled synthetic fibers with this practice. Illinois DOT implemented a more unconventional practice with synthetic fibers by using a heavy-duty blower, again with the aim of dispersing fibers more evenly. Washington DOT suggests the use of a screen with a mesh of 1.5” to 2.5” to help prevent fiber balling. All these methods are discussed at greater length in Section 2.7.

A variety of practices are recommended in ACI 544.1R-96 regarding the mixing procedure. The most common method is to wait to load fiber until all other materials have been mixed together. The reason for the popularity of this method is that the fibers are in the mixer for the shortest amount of time and with the entirety of the coarse aggregate, both of which help to prevent fiber balling. It is recommended that steel fibers be added at a rate of approximately 100 lb/min and with a mixing speed of 40 revolutions/min. Other methods mentioned include adding fibers to the coarse aggregate stream before adding the aggregate to the mixer or adding fibers on top of the aggregates after they are weighed in the batcher. These practices would also aid in preventing fiber balling by using the natural tendency of coarse aggregate to separate fibers, but will expose the fibers to a longer period of mixing.

2.6.3 Placing and Finishing

In typical volume ranges for FRC, the addition of fibers will likely reduce workability. To ensure appropriate consolidation, mechanical vibration is necessary. Top surfaces should be struck off with a screed, and the concrete should then be finished with trowels or a bull float. Edging may be necessary to keep fibers from being exposed. This is not much of a concern, but occasionally, after hardening, this can be a weak point where a fiber can easily be pulled out and take a fragment of concrete with it. The timing of sawing at joints is critical so that macro fibers are not pulled up.

2.7 State DOT Experiences

The research team conducted an extensive survey on the lab and field projects performed by different state DOTs, and the results are summarized in Table 2.2. As showed in the table, a wide range of types, sizes, and dosages of fibers have been used in different applications, including pavement, pavement overlay, and bridge deck.

Table 2.2 Summary of fibers used in mix design by different DOTs.

State	Application	Project Type	Fiber Type	Volume %	Dosage (pcy)	Length (in)	Diameter (in)	Material
CA	Bridge approach slab	Lab	Synthetic Steel Steel	0.2 0.5 0.8		0.32 1.18 2.36	0.002 0.02 0.03	PVA
FL	Pavement replacement	Lab	Synthetic Steel Glass Basalt Nylon	0.1,0.3 0.1 0.1 0.1 0.1,0.3		0.5, 0.75, 1.5 0.5 0.5 0.5 0.5		Polypropylene
IA	Overlay	Field	Steel	0.45-1.22		1 2.5		Steel Steel
IA	Overlay	Field	Synthetic	NA		NA		Monofilament Fibrillated
IA	Overlay	Lab/ Field	Synthetic Synthetic Synthetic	0.16 0.05 0.16				Fibrillated Monofilament Structural
IL	Bridge deck overlay	Lab	Synthetic	0.16		0.75 1.55 0.75 0.75 1.5 1.18 1.18		Monofilament Monofilament Collated-Fibrillated Resin-bounded Collated-Fibrillated Monofilament Resin-Bundled
	Ultra-Thin Overlay	Lab	Steel Steel Steel Steel Synthetic Synthetic Synthetic	0.19-1.56 0.19-1.56 0.19-1.56 0.19-1.56 0.19-1.56 0.19-1.56 0.19-1.56		2 2.36 2 1.5 1.57 1.97 2.12		Hooked end Hooked end Crimped Crimped Straight Crimped Twisted

Table 2.2 Summary of fibers used in mix design by different DOTs (continued).

State	Application	Project Type	FiberType	Volume %	Dosage (pcy)	Length (in)	Diameter (in)	Material
LA	Bonded overlay	Field	Steel Fiberglass Synthetic	0.46-1.1 0.88-1.88 0.04-0.08		0.75-2 1.5-2.5 0.5-0.75		Steel Hooked end Fiber Glass Polypropylene
	Bonded overlay	Field	Steel	0.65		1		
	Pavement		Synthetic Synthetic Carbon Steel	0.1,0.2,0.3 0.3, 0.5, 0.7, 1.0 0.3, 0.7, 1.0 0.9		1.5 2.25 4.00 2.00		Polypropylene Polypropylene
	Bridge deck	Field	Steel	0.65		2.36		
OR	Bridge overlay	Field	Steel	0.65		2	0.02	
	Bridge overlay	Field	Synthetic	-		No Info		Polypropylene
	Shrinkage control	Lab	Synthetic Blended	0.27, 0.4, 0.54	5, 7, 10	0.5(micro) 1.8(macro)	0.01	Polypropylene
TX	Pavement	Field	Steel Synthetic	0.19, 0.2, 0.3 0.08, 0.2, 0.3		1.97, 2.36 1.57, 1.18		
	Pavement overlay	Field	Steel Synthetic	0.33 0.20		2.36 NA		Collated polypropylene
VA	Bridge deck	Field	Synthetic	0.47	10	2		Monofilament
	Pavement overlay	Field	Synthetic Steel	0.38-1.1 0.38,0.59		0.75,1, 2 1.26		Polypropylene/ Polyolefin hooked end
	Bridge deck	Field	Synthetic	0.47		2		Monofilament
	Bridge deck	Lab	Synthetic	0.2-1.63		0.75, 1, 2		Polypropylene

The following is a summary with some details from research and/or construction projects in different states.

California

The University of California, Berkeley prepared a report in 2008 for CalTran entitled "Use of Fiber reinforced Concrete in Bridge Approach Slabs". (Ideker and Banuelos, 2014). The research team developed a type C hybrid fiber reinforced concrete (HyFRC) and constructed approach slabs for bridges at "Area II jurisdiction" that have severe environmental conditions. For mix design purposes, the performance of concrete based on deflection hardening was set as the goal. The material flexural performance was quantified using a four-point flexure test on 6 in. deep beam specimens. It was observed that the flexural strength and stiffness were enhanced in the HyFRC. Crack resistance was studied under a cyclic flexure test. A load that exceeded the plain concrete modulus of rupture and was below the yield strength of rebar was applied, and no surface cracks were observed in HyFRC. Cracks were observed, however, in specimens made with plain concrete. Corrosion resistance was also found to increase with HyFRC.

Florida

A project titled "Crack Control in Overlays for Precast Flat Slab Bridge Deck Construction" was concluded in 2006, which focused on developing techniques for improving "reflective cracks" that form in the overlays placed over precast panels on flat slab bridges. The research included the construction of four full-scale 4-ft. x 30-ft precast flat slabs with a 6-in. concrete overlay. Concrete overlays incorporated steel fibers, synthetic fibers, type A hybrid (steel/synthetic fiber blend), and carbon fiber-reinforced composite (CFRP) grids. A shrinkage reducing admixture was also used. Results showed that steel fiber is most effective in reducing reflective cracks. However, the placement, vibration, and finishing in mixes with steel fibers were more difficult than other mixes. Synthetic fiber and blended fiber were rated below steel fibers for crack control. It was also recommended to adjust the mix design when fibers are added for obtaining better workability (Hamilton et al., 2006).

Illinois

"Superiority and Constructability of Fibrous Additives for Bridge Deck Overlays" was a project in 2012 with the objective of investigating the potential of using synthetic fiber to enhance the performance of bridge deck concrete overlays. The project included determining the practical dosage and type of synthetic fibers for usage in concrete while maintaining appropriate workability and finishability. Thirteen different mixes using monofilament, resin-bundled monofilament, and collated-fibrillated synthetic fibers were made and evaluated. Besides the resin-bundled monofilament fibers that were only in micro form, the other two types had both micro and macro forms, which resulted in a total of seven types of micro and macro fibers. It should be noted that the research grouped assigned fibers with a length of 1.18 in. and longer as macro and smaller than 1.18 in. as micro, which is different than the 1.5" cutoff line that most DOT's and research studies specify. Mixes included either micro, macro, or both micro and macro (type B blended) fibers. Results showed that mixes with blended fibers (1.18 in. monofilament and micro collated-fibrillated) exhibited better finishability compared to mixes with only collated-fibrillated macro fibers. However, mixes made with macro monofilament fiber had good finishability as well. Furthermore, the blended fiber mix showed the best results for the rapid chloride permeability test. Significant reduction in drying shrinkage, increase in post cracking residual strength, and improved compression failure modes were observed when compared to plain concrete. The shrinkage of plane concrete after 100 days was about 150 microstrains higher than concrete made with micro monofilament fibers, which had the highest shrinkage compared to the other mixes. The mix made with blended micro and macro monofilament fibers had the least shrinkage. However, the difference between the maximum and minimum values did not exceed more than about 50 microstrains during the test period after 100 days. All the fibrous samples had an increase in flexural strength between 7% to 11% when compared to plain concrete. Also, these samples stayed intact after failure in compression tests, while plain concrete specimens crushed at the ultimate strength (Alhassan and Ashur, 2012). The use of a maximum of 3 pcy of synthetic fiber was recommended. Also, 1.75 in. was recommended as the maximum length of fibers, and 0.75 in. was recommended as the minimum fiber length.

Louisiana

In a broad 1991 study titled “Evaluation of Fiber Reinforced Concrete”, steel, fiberglass, and polypropylene fibers were used with various shapes of steel fibers, including deformed, corrugated, and hooked-end. Superplasticizer was added in order to ensure appropriate concrete workability. Based on Louisiana DOT requirements for workability, concrete should have a slump between 2 in. and 4 in. and an air content between 4% and 6%. FRC mixes met these requirements. It was observed that steel fibers with high aspect ratios were more effective in improving the flexural toughness.

Another Louisiana DOT study from 2004-2008, titled “Flexural Strength and Fatigue of Steel Fiber Reinforced Concrete”, involved the installation of a test section on the Luling Bridge with an SFRC bridge deck installed on top of a steel deck using epoxy in between the two. The evaluation revealed that debonding had happened between the epoxy and the steel deck, which was the primary cause of the failure. The performance of the SFRC was satisfactory, and only minor rutting and cracking was observed.

Oregon

The Oregon DOT conducted a project titled “The Use of Synthetic Blended Fibers to Reduce Cracking Risk in High Performance Concrete” in 2014. The project was conducted to study the effect of blended size polypropylene fibers in controlling shrinkage cracks in HPC. Polypropylene fiber was chosen in the study due to its superior resistance to chemicals and fatigue loads (Ideker and Banuelos, 2014).

Three dosage rates of 5 lb/yd^3, 7.5 lb/yd^3, and 10 lb/yd^3 were used in the study, and results showed that the 10 lb/yd^3 mix had lower workability and needed a higher amount of superplasticizer. The properties of concrete were not dramatically enhanced compared to mixes with lower dosages of fiber. Only a slight decrease of free drying shrinkage was observed when fiber was used. The reduction of cement content did not help in reducing free shrinkage and ended up decreasing compressive strength as much as 25% below the 4000 psi minimum. In the restrained shrinkage tests (ring test), crack widths in specimens containing fibers were significantly reduced (from 0.035 in the control mix to as low as 0.005 in.). Also, the freeze/thaw resistance of fiber reinforced concrete was found to have increased with the incorporation of fibers.

Texas

A Texas DOT report, titled "Restrained Shrinkage Cracking of Concrete Bridge Decks: A State-of-the-Art Review", (Brown et al. 2001) discusses the mechanism of shrinkage and creep issues and presents the tests for evaluating these issues. The use of fibers for eliminating shrinkage cracks was also discussed. However, no specifications for the use of fibers were determined, and mostly different types of fibers are introduced without defined dosages or physical or mechanical properties.

Virginia

A 2005 Virginia DOT project (Pzyildirim 2005) conducted with the University of Virginia with the objective of comparing the performance of high-performance fiber-reinforced concrete and conventional concrete in bridge decks. Synthetic fibers were used in the FRC mixture, and the constructed sections were monitored for five years. It was observed that the FRC sections developed fewer and narrower cracks despite the fact that higher shrinkage occurred.

An earlier study in 1997, titled "Investigation of Fiber Reinforced Concrete for Use in Transportation Structures", was an investigation sponsored by VDOT to determine the properties of FRC with different fibers in pavement and bridge deck overlay applications. In this research, hooked-end steel, monofilament polypropylene, fibrillated polypropylene, and monofilament polyolefin fibers were used. This project was done parallel with an overlay project in 1995. It was concluded that concrete made with steel fibers had the highest toughness, followed by polyolefin and polypropylene fibers. The impact resistance of FRC was found to be enhanced, and field investigations revealed that crack widening was better controlled in FRC.

2.8 State DOT Specifications

To better understand different DOT's experiences of FRC, the research team conducted a comprehensive survey by reviewing published specifications and research/experimental reports from different DOTs, as well as contacting DOT personnel through email and telephone interviews. Based on the information obtained, the research team clarified the status of DOT FRC application in the following five categories:

1. Specifications developed and lab and/or field study completed
2. Specifications developed and research is in progress
3. Lab and/or field study complete
4. Research in progress
5. No information found

With all the 50 states surveyed, the following fourteen states have developed specifications for different applications such as bridge decks, pavement, and shotcrete: Colorado, Florida, Idaho, Illinois, Kansas, Louisiana, Maine, Maryland, Minnesota, Mississippi, Montana, North Carolina, Oregon, and Washington. See Figure 2.5 for a shaded map covering the level of experience for each state.

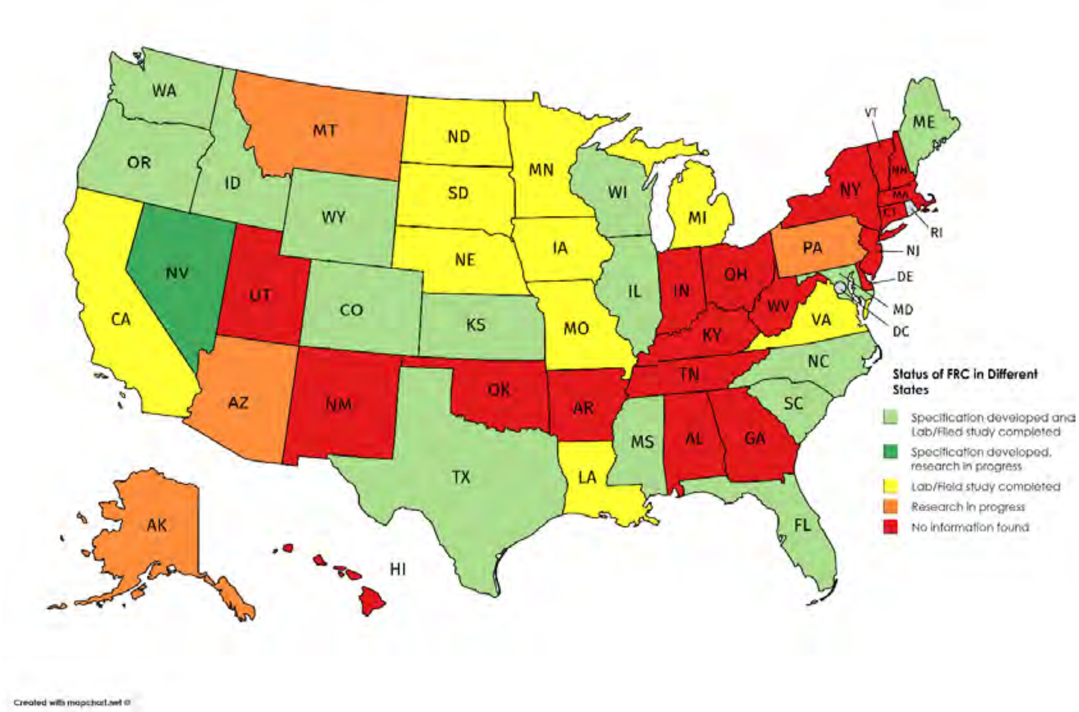


Figure 2.5 Status of FRC in different states.

Three DOTs have developed a prequalified materials list for fibers: Illinois, Texas, and New Hampshire. The list developed by New Hampshire only includes qualified fiber brands for precast drainage structures and bituminous curbs and also provides no information on the properties of the fibers. Lists developed by Texas and Illinois DOTs include recommended dosage and length of fibers as well as the fiber producers. According to the prequalified materials list developed by Texas DOT (TxDOT), the dosage of steel fibers varies from 13.5 to 48 lb/yd^3 with length varying between 1 to 2 inches; the dosage of synthetic fibers varies from 3 to 5 lb/yd^3 with lengths varying between 0.5 to 2.25 inches. Illinois DOT has specified the dosage of synthetic fibers between 4 to 5 lb/yd^3. BASF and FORTA are the producers that are recognized by all three mentioned DOTs.

State DOT specifications are summarized in Table 2.3.

Table 2.3 Specifications developed by DOTs

State	Application	Type	Fiber Properties							Concrete Properties/Mixture			
			L_{min} (in)	L_{max} (in)	Dia. (in)	$f_{tensile, min}$ (ksi)	E'c (ksi)	Aspect Ratio $_{min}$	Aspect Ratio $_{max}$	Dosage $_{min}$ (pcy)	Dosage $_{max}$ (pcy)	Vol.%	Mix method
CO	Structural FRC	Polyolefin	-	-	-	-	-	-	-	3.5	-	0.19	Manufacturer recommendation
	Structural Macro-FRC		1.5	2.2	-	65	1000	50	100	4	-	0.22	Manufacturer recommendation
FL	Prestressed deck slab	Steel*, Polymeric, basalt	1.95	2.05	0.035	120	-	51	69	1.5	-	0.08	-
IL	Bridge deck overlay	Synthetic Type III-C1116	1	2.5	-	-	-	-	150	2	5	0.11-0.27	
KS	Shotcrete	Polypropylene	-	-	-	-	-	-	-	-	-	-	-
LA	Depth patching JCP	ASTM A-820 Type I/II	1	1.5	-	-	--	40	60	85	90	0.65-0.68	Glue fibers added last, rate not exceeding 132 lb/min
ME	Precast Elements	Macro synthetic-Polyolefin, carbon, nylon	1.5	-	-	40	400	45	150	Manufacturer should indicate	-	-	-
	Culvert Rehab	Polypropylene, steel	1	NA, 1.375	-	-	-	NA, 60	-	1.5, NA	-	0.08	-
MA	PCC	Synthetic, ASTM C1116 Type III	0.5	1.5	-	-	-	-	-	1.5	-	0.08	-
	PCC Overlay	Synthetic	-	-	-	-	-	-	-	1.5	-	0.08	-
MN	Bridge deck	Polypropylene micro/macro	1.8	-	-	70/ 85	-	-	-	4 pcy or manufacturer's recommendation	-	0.22	Distribute on aggregate belt/ introduce into truck
MS	Bridge Deck***	-	-	-	-	-	-	-	-	-	-		-

* Dimensions are only for steel fiber

** Dosage of steel fiber and synthetic fibers

*** Dosage should be such that the average residual strength ratio of FRC beam be min 20% when tested according to C1609

Table 2.3 Specifications developed by DOTs (continued).

State	Application	Type	Fiber Properties							Concrete Properties/Mixture			
			L_{min} (in)	L_{max} (in)	Dia. (in)	$f_{tensile, min}$ (ksi)	E'c (ksi)	Aspect Ratio $_{min}$	Aspect Ratio $_{max}$	Dosage $_{min}$ (pcy)	Dosage $_{max}$ (pcy)	Vol.%	Mix method
MT	General	Polypropylene/synthetic	-	As graded by manufacturer for aggregate size		80	110	-	-	1.5	-	0.08	At the time of mixing/Manufacture recommended
NC	Precast drainage	Macro-synthetic	1.5		-	40	400	45	150	-	-		-
	FRC-replacement of steel reinforcement	-	-	-	-	-	-	-	-	5	-	0.27	-
OR	HPC bridge deck	Synthetic macro/micro fiber from QPL	-	-	-	--	-	-	-	-	--	-	-
WA	Shotcrete	Steel, macro synthetic****	-	-	-	-	-	-	-	100/10 **	--	0.76/0.54	Only steel for dry mix/ either fiber for wet mix

** Dosage of steel fiber and synthetic fibers

*** Dosage should be such that the average residual strength ratio of FRC beam be min 20% when tested according to C1609

**** If fibers are added during the batching and mixing, a screen having a mesh of 1.5 to 2.5 in. should be used to prevent balling

2.9 Summary

After a review of existing design methods – both empirical and scientific – and a review of state DOT reports and practices, it is clear that fiber reinforced concrete is both a developed idea in some instances and one that still needs a heavy amount of research in others. Empirical design methods are very limited in quantity, and the ones that do exist are questionable in terms of their applicability. Scientific design methods are still in their infancy relative to design methods for other special concretes such as self-consolidating concrete and high strength concrete.

An empirical and a scientific design method were presented in this study. The empirical method involved a nomogram and proved limited in application. The scientific design method was more applicable as it involved a way of accounting for fiber type through a so-called "equivalent fiber diameter". This design method mentions excess paste, which is a very important concept in this research and is heavily featured in Chapter 4 and Chapter 5.

A review of the general experience and specifications for FRC through reports conducted by state DOTs revealed many common findings. In summary, steel fibers provided the most improved hardened concrete performance – particularly under flexural loading – at the cost of workability; synthetic fibers limited negative effects on workability and were less costly but also inferior to steel in flexural performance.

CHAPTER 3. MATERIALS, MIX PROCEDURE, AND TEST METHODS

3.1 Introduction

This chapter presents the information of aggregate, cement, cementitious materials collected from Nevada and Nebraska, as well as the different fibers and chemical admixtures collected for the study. The mixing procedure for both the reference and fiber-reinforced mixes, and the test methods that were used to evaluate the performance of the concrete are also covered. Mix designs were provided by the Nevada and Nebraska DOT's and are common to bridge decks in both these states. The mixing procedure is almost exactly the same for both reference and fiber-reinforced mixes, the only difference being an additional 5-minute mixing period at the end of the procedure, which allows the fibers to distribute properly. Test methods in the fresh concrete state include slump, vibrated J-ring, vibrated L-box, air content, and fresh unit weight. Mechanical tests included compressive strength, flexural strength, splitting tensile strength, and slant-shear; durability tests included drying shrinkage, restrained shrinkage, and freeze/thaw.

3.2 Materials

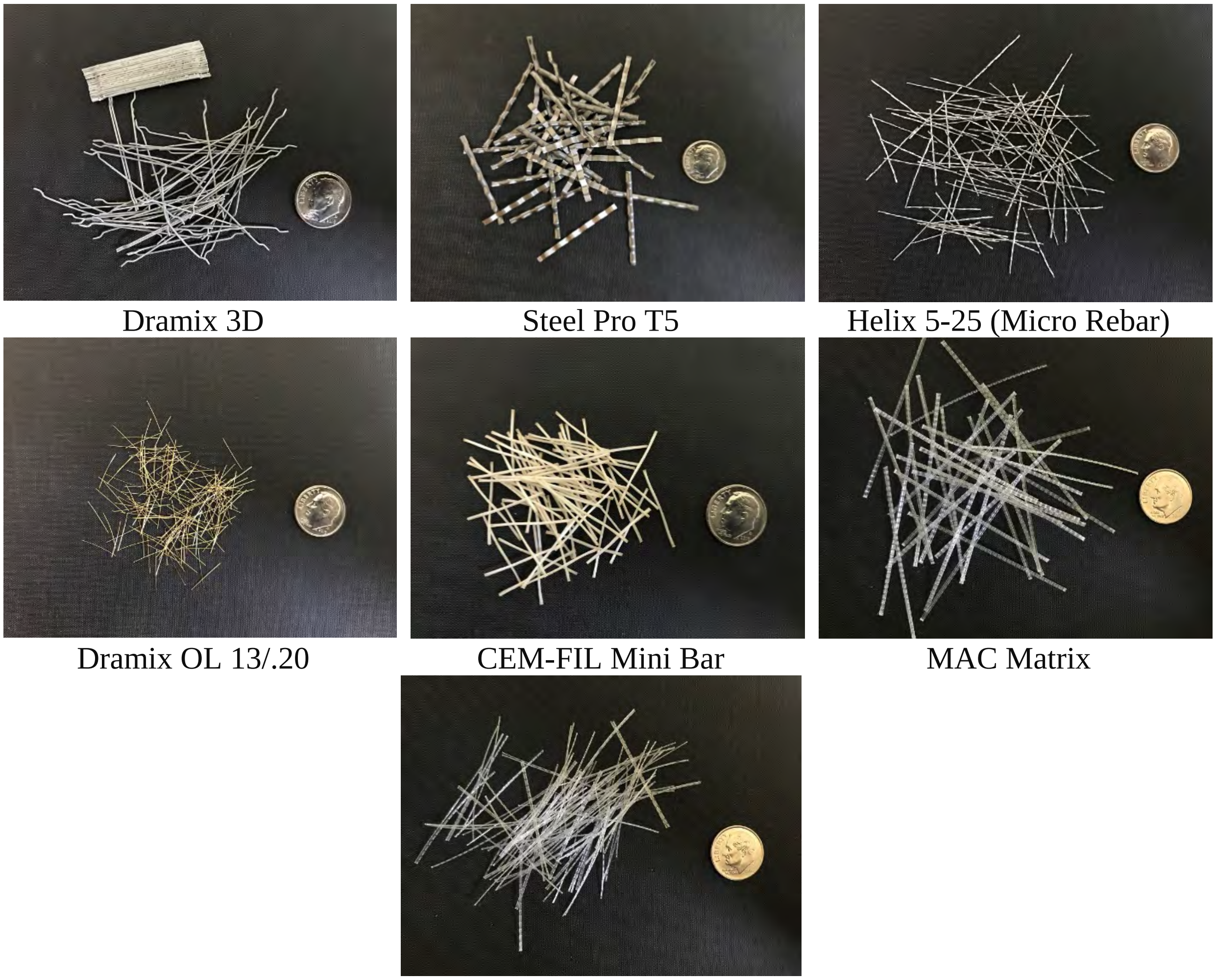
3.2.1 Fiber Selection

There are many different fiber producers available in the US. Based on the information obtained from an extensive literature review of available research and construction reports by different DOTs and prior knowledge of fibers, about 40 fibers were identified. The identified fibers included steel, synthetic, and blended fibers. Also, the brands and products prequalified by other DOTs (Texas and Illinois) were identified. Fibers selected in this study were mentioned in other DOT reports as well, including Iowa and Virginia.

Steel fibers are generally made from carbon steel or stainless steel (ACI Committee 544, 2002); synthetic fibers are produced from petrochemical industries. There are different types of synthetic fibers, such as nylon, polypropylene, and polyethylene, with polyethylene and polypropylene fibers being the most common synthetic fibers used in the concrete industry (ACI Committee 544, 2002). Fibers are produced in different forms, such as hooked-end, twisted, and corrugated steel fibers, and monofilament, collated-fibrillated, and stick-like synthetic fibers. Hooked-end steel and stick-like synthetic macro fibers are the most common fibers used to enhance post crack properties of concrete by bridging the crack (Ozyildrim et al., 1997). Synthetic fibers are resistant against corrosion, alkali reaction, and as a result of low density are added at low dosages (between 0.1% to 0.3% by volume) (Alhassan and Ashur, 2012). However, the improvement of toughness and residual strength achieved by steel fibers is significantly greater than synthetic fibers (almost seven times more (Ozyildrim et al., 1997)). Both micro and macro fibers were analyzed in this study. Microfibers help to control concrete shrinkage at an early age and at the micro-level, whereas macro fibers increase concrete toughness and control crack width in the hardened state. Blended fibers consist of both macro and microfibers. There is currently no consensus related to the benefits of a blended fiber mix, and thus steel and synthetic fibers were primarily studied for this project. A small number of DOT studies utilized blended fibers; it was the primary focus of a study conducted by the Oregon DOT. The studies analyzed the effects of blended fibers, with the common benefit being higher workability and drawback being lower flexural strength, particularly in higher doses. None of these studies recommended blended fibers over macro fibers for usage on bridge decks, as the flexural strength and crack control were simply much higher with macro fibers.

With respect to macro steel fibers, hooked end steel fibers can provide good bonding strength with concrete. Bakaert and Propex are two producers of hooked end fibers that are prequalified by Texas and Illinois DOT. Bekaert produces three different types of hooked end fibers (Dramix 3D, 4D, and 5D), with Dramix 3D being the most common fiber used by DOTs. Propex produces one type of hooked end fiber (Novocon 1050), which is comparable to Dramix 3D in form and shapes with their lengths (2” vs. 1.4”), diameters (0.036” vs. 0.022”), and aspect ratios (50 vs. 60) being slightly different.

In order to study the effect of length and hook type of steel fibers on the performance of FRC, the research team selected mostly macro fibers. No information was found on the application of Dramix 5D, so the research team elected to use Dramix 3D and SteelPro T5 as the focus for macro steel fibers. The two microfibers to be studied were Dramix OL13-0.2, which was primarily selected because of its prevalence in ultra-high performance concrete, and Helix 5-25 (also known as Mini Rebar). Both microfibers were recommended by various DOT’s. As the name suggests, Helix 5-25 has a rebar-like appearance with its twisted form (Figure 2.1 Types and categories of fibers.Figure 2.1). These fibers were selected to study the effect of microfiber shape on the properties and performance of FRC. The selected fibers can be seen in Figure 3.1.



Dramix 3D | Steel Pro T5 | Helix 5-25 (Micro Rebar)

Dramix OL 13/.20 | CEM-FIL Mini Bar | MAC Matrix

HP Omni

Figure 3.1 Fibers used in the study

As for synthetic fibers, abcPolymer Omni HP, BASF MacMatrix, and Owens Corning Cem-FIL minibars were selected. The MacMatrix fibers were both untreated and coated with a chemical to improve concrete bonding. The coated fibers will be labeled “MacMatrix (CB)”. Specific details regarding all fibers documented in the literature review can be seen in Table 3.1.

Table 3.1 Physical and mechanical properties of selected fibers

	Producer	Product	Length	Aspect Ratio	Shape	Form	Modulus of Elasticity (ksi)	Application	Tensile strength (ksi)	Prequalifying DOT
Steel	abcPolymer	SteelPro T5	1.5”	25	Deformed	Corrugated	29000	Composite Deck, Bonded overlay	128	TX, IL
	Bakaert	Dramix 3D	1.4”	65	Deformed	Hooked-End	29000	Various	178	TX, IL
	Bakaert	Dramix OL 13/.20	0.5”	63	Straight	Straight	29000	UHPC	399	Several (used in UHPC)
	Helix	5-25 (micro rebar)	0.5” 1.0”	50	Deformed	Twisted	29000	Paving, slab	246.5	TX
Synthetic	AbcPolymer	Omni HP	2.0”	70	Straight	Stick-Like	N/A	Composite deck, pavement	N/A	IL
	BASF	MAC Matrix	2.1”	70	Straight	Stick-Like	N/A	Composite deck, pavement, overlay	85	TX, IL
	Owens Corning	Cem-FIL MiniBars	1.7”	57	Straight	Stick-Like	6091	Decks, Marine Structures, Tunnels	145	TX

A very important property of fiber is the *aspect ratio,* which is the ratio of length to diameter. Higher aspect ratios will result in better performance of concrete. However, very high aspect ratios can cause balling of fibers (the tendency for fibers to clump together) and lower workability in general as the aggregate matrix is. The research team believes that fibers smaller than 0.75 inches may not be efficient for project purposes. This would only include the Dramix OL13/.20, but as mentioned before, this is a very common fiber that is frequently seen in UHPC.

3.2.2 Aggregate

The aggregates used in this experiment were shipped from Nevada to the University of Nebraska and gradation, absorption, and specific gravity tests were performed in accordance with ASTM C136 and ASTM C128. Both sets of aggregates included a coarse, fine, and intermediate coarse aggregate. Table 3.2 **Error! Reference source not found.**shows the results from the tests performed in the lab and the information provided by Sierra Ready Mix and American Ready-Mix. The absorption values obtained in the lab for the Las Vegas fine and #67 aggregates were different from the values provided by Sierra Ready Mix. The tests were performed again, and the same results were obtained. Thus, lab experiments were performed using the results obtained by the research team. Materials from Omaha and Lincoln were used as well. Lincoln materials were used as an additional trial for the adjustment method presented in section 4.3.1. Omaha materials were used for Super High-Performance Concrete (SHPC), which is detailed in Chapter 5.

Table 3.2 Aggregate properties.

	Aggregate type	Location/Source	Specific Gravity	Absorption (%)
Las Vegas	#67 Coarse	Sierra Ready Mix, Las Vegas	2.68	1.40
	#89 Coarse	Sierra Ready Mix, Las Vegas	2.59	1.38
	Fine	Sierra Ready Mix, Las Vegas	2.62	1.60
Reno	#67 Coarse	American Ready-Mix, Sparks, NV	2.59	2.99
	#89 Coarse	American Ready-Mix, Sparks, NV	2.60	2.81
	Fine	American Ready-Mix, Sparks, NV	2.60	3.52
Omaha	#67 Coarse	Lyman-Richey, Omaha	2.65	1.31
	#89 Coarse	Lyman-Richey, Omaha	2.66	1.50
	River Sand	Lyman-Richey, Omaha	2.65	2.76
Lincoln	#57 Coarse	Ready Mix Concrete Co., Lincoln	2.66	1.20
	Sand and Gravel	Ready Mix Concrete Co., Lincoln	2.62	0.60

A standard sieve analysis, according to ASTM C136 was performed, and the gradation results are seen in Figure 3.2 Gradation of aggregate.Figure 3.2. Using a gradation analysis method (Modified Anderson and Andreassen Model), the Lincoln #57 coarse aggregate combined with sand and gravel proved to be a more similar blend to the Las Vegas materials than if the intermediate aggregate had been used. Lincoln materials were used primarily because it was not feasible to use Nevada aggregates in the large scale slab pours detailed in Chapter 6. These materials also provided an opportunity to test the mix design adjustment method on the third set of materials. Thus, only one coarse aggregate was used for Lincoln mixes so as to maintain similar conditions.

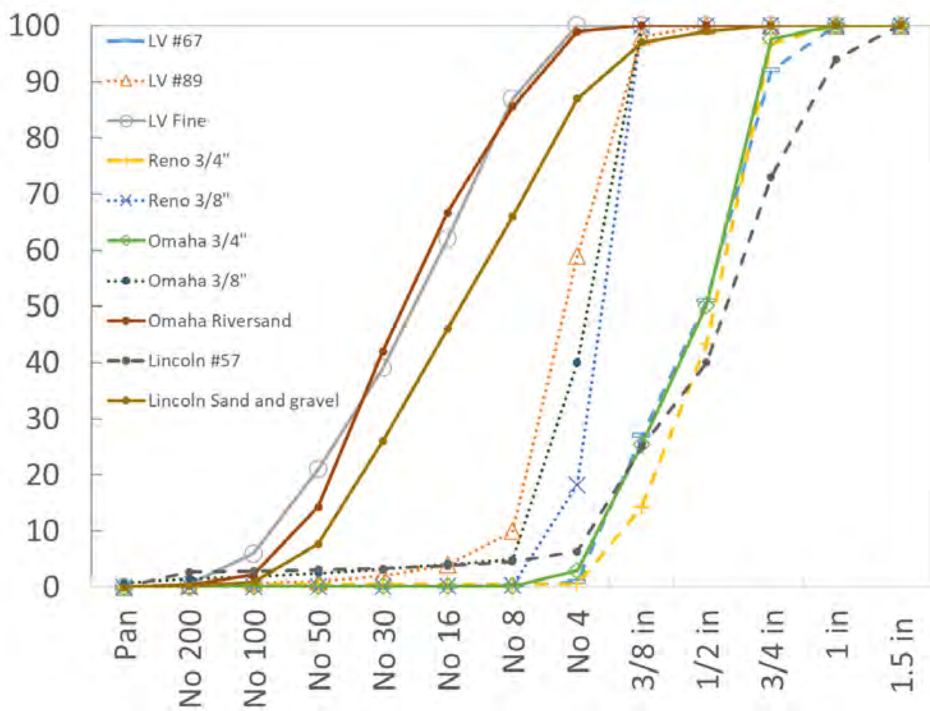


Figure 3.2 Gradation of aggregate.

3.2.3 Cement and Supplementary Cementitious Materials

Las Vegas utilizes Type V cement while Reno uses Type I/II. Type IP cement is very common throughout Nebraska and was used for both Omaha and Lincoln material mixes. Class F Fly Ash is used for both Reno and Las Vegas, but with some minor differences between them as they are from different sources. Lastly, Silica Fume was used in the SHPC mixes to improve strength. A general summary of the materials and their source can be seen in Table 3.3.

Table 3.3 Cement and fly ash types and sources

		Location/Source
Las Vegas	Type V Cement	Ash Grove, Leamington Plant (Nephi, UT)
	Class F Fly Ash	Navajo Plant (Page, AZ)
Reno	Type I/II Cement	Leigh Cement (Redding, CA)
	Class F Fly Ash	Jim Bridger (Rock Springs, WY)
Omaha & Lincoln	Type IP Cement	Ash Grove, Springfield Plant (Springfield, NE)

The chemical properties and specific gravities are summarized in Table 3.4.

Table 3.4 Chemical properties of cementitious materials

Substance	Type I/II Cement	Type V Cement	Type IP Cement	Class F Fly Ash (Reno)	Class F Fly Ash (Vegas)	Silica Fume
Silicon Dioxide (SiO_2) (%)	20.2	21.0	-	59.2	60.6	92.5
Aluminum Oxide (Al_2O_3) (%)	4.8	3.8	-	17.9	22.2	0.5
Ferric Oxide (Fe_2O_3) (%)	3.4	3.6	-	4.9	4.5	-
Calcium Oxide (CaO) (%)	64.4	63.3	-	7.0	4.9	-
Magnesium Oxide (MgO) (%)	1.3	2.5	2.5	-	-	-
Sulfur Trioxide (SO_3) (%)	2.6	2.8	3.1	0.8	0.4	-
Sodium Oxide (Na_2O) (%)	0.39	0.1	-	-	-	-
Potassium Oxide (K_2O) (%)	0.7	0.7	-	-	-	-
Carbon Dioxide (CO_2) (%)	1.7	1.5	-	-	-	-
Limestone (%)	4.3	3.7	-	-	-	-
$CaCO_3$ in limestone (%)	88.0	89.7	-	-	-	-
C_3S	60.0	54	-	-	-	-
C_2S	13.0	19	-	-	-	-
C_3A	6.0	4	-	-	-	-
C_4AF	9.0	11	-	-	-	-
$C_4AF + 2(C_3A)$	24.0	19	-	-	-	-
(Cl^-) %	-	-	-	-	-	0.1
Loss on ignition (%)	2.7	2.5	1.0	2.3	0.7	3.4
Insoluble residue (%)	0.2	0.4	-	-	-	-
Specific Gravity	3.15	3.15	2.99	2.37	2.29	2.22

3.2.4 Chemical Admixtures

Las Vegas mixes included a pair of water reducers with one being mid-range (Eucon X-15) and the other being high-range (Plastol 6200EXT). Reno mixes included four admixtures: a high range water reducer (Glenium 7500), viscosity modifier (VMA 362), hydration controller (Delvo), and an air-entraining agent (Master Air 200). The recommended ranges can be seen in Table 3.5.

Table 3.5 Admixture properties

	Admixture	Location/Source	Type	Range (fl oz/100lb)
Las Vegas	Eucon X-15	Euclid	MRWR	4 – 15
	Plastol 6200EXT	Euclid	HRWR	3 – 12
Reno	Master Air 200	BASF	AEA	0.125 – 1.5
	Glenium 7500	BASF	HRWR	2 – 15
	Delvo Stabilizer	BASF	Retarder	1.5 – 25
	VMA 362	BASF	Viscosity Modifier	2 – 14
Omaha	Master Air 200	BASF	AEA	0.125 – 1.5
	Glenium 7500	BASF	HRWR	2 – 15
	Delvo Stabilizer	BASF	Retarder	1.5 – 25
	VMA 362	BASF	Viscosity Modifier	2 – 14
Lincoln	Pozzolith 322	BASF	HRWR	3-5
	Master Air 200	BASF	AEA	1.5

3.3 FRC Mixing Procedure

The procedure for mixing FRC follows ASTM C192 (Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory) with one additional step at the end. To begin the procedure, coarse and intermediate aggregates are loaded into the mixer with approximately 75% of the water. The air-entraining agent (if applicable) is included with this portion of the water. The mixer is then turned on for thirty seconds. Following this brief mixing period, the remaining ingredients are loaded in order of largest particle size to smallest: Fine aggregate, cement, fly ash, and 20% of the water (with remaining admixtures). 5% of the water is retained and used later if necessitated by low workability. The mixer is then run for 3 minutes, followed by a 3-minute rest period, and closing with another 2 minutes of mixing. At this stage, a slump test, according to ASTM C143 (Standard Test Method for Slump of Hydraulic-Cement Concrete) is performed to determine the workability of the concrete before the introduction of fibers for comparative purposes. The concrete from the slump test is loaded back into the mixer. At this point, fiber is introduced at the beginning of a 5-minute mixing period. In smaller batches, such as the one seen in Figure 3.3, this is done by hand. Ideally, all fiber should be introduced during the first minute of a 5-minute mixing period. With higher fiber contents such as 1.5% and 2.0%, loading fiber in the first minute can be difficult to accomplish, but this is not considered an issue so long as the fibers are introduced continuously and are visibly well distributed in the concrete.

Loading coarse aggregate

Introduction of fiber

Figure 3.3 Batching in lab drum mixer

3.4 Test Methods

3.4.1 Aggregate-Fiber Skeleton Void Content Tests

The aggregate-fiber skeleton void content test is a very important step in the mix design adjustment for the inclusion of fiber. The aim of this procedure is to determine fiber's effect on the void content and utilizing the values to have an idea of the necessary paste increase to retain workability with higher fiber volumes.

The overall process is very similar to that of ASTM C29 (Standard Test Method for the Unit Weight and Voids in Aggregate). It is essentially a modified version of this standard to account for a blend of aggregates and fibers rather than a singular aggregate. Aggregates are oven-dried for 24 hours and mixed together in a bucket. The proportions of the blend are equal to that of the state mix designs but with increasing fiber volumes for each iteration of the test.

All the same variables from ASTM C29 are used, with the notable difference being that these variables are for the entire blend rather than an individual aggregate. The first necessary variable – the combined bulk specific gravity – can be determined with the following equation:

$$G_{sb,combined} = \frac{1}{\frac{P_{CA1}}{G_{sb,CA1}} + \frac{P_{CA2}}{G_{sb,CA2}} + \frac{P_{FA}}{G_{sb,FA}} + \frac{P_{fiber}}{G_{sb,fiber}}} \tag{3.1}$$

where G_{sb} and P represent the specific gravity and fraction of each component. The subscripts of $_{combined}$, $_{CA1}$, $_{CA2}$, $_{FA}$, and $_{Fiber}$ represent the combined mixture, coarse aggregate, intermediate aggregate, fine aggregate, and fiber, respectively. A sample calculation using Las Vegas materials with a 1.5% fiber volume (relative to all mixture ingredients, not just aggregate), can be seen below in Table 3.6.

Table 3.6 Sample Calculation -- Las Vegas Materials with 1.5% fiber

Aggregate/Fiber	Specific Gravity	Proportion	Combined SG
#67	2.68	0.514	2.721
#89	2.59	0.055	
Fine aggregate	2.62	0.372	
Fiber	7.85	0.098	

With the blend inside, the bucket is rolled and flipped several times until the blend appears to be evenly distributed. Using a 0.25 ft^3 measure, the blend is loaded in three layers. Each layer is consolidated with three different consolidation methods. These methods are rodding, jigging, and shoveling. The shoveling values were largely ignored for this project, given that shoveling is representative of a concrete that is not consolidated in the field (i.e., self-consolidating concrete). Thus, rodding and jigging values were used and were averaged. Figure 3.4 Figure 3.4 shows the fiber aggregate blend both before and after the procedure.

Blend before test — Leveled after consolidation

Figure 3.4 Aggregate-fiber skeleton test

Once the blend is leveled off at the top, it is weighed to determine the bulk density (M) of the blend. The bulk density is simply calculated by the following equation:

$$\rho_{bulk} = \frac{m_{blend}}{V_{container}} \tag{3.2}$$

where ρ_{bulk} is the bulk density, m_{blend} is the mass of the blend, and $V_{container}$ is the volume of the container. The void content can then be calculated with the following equation:

$$\%_{voids} = 100 * \frac{(G_{sb,combined} \text{ x } UW_{water}) - M}{(G_{sb,combined} \text{ x } UW_{water})} \tag{3.3}$$

with UW_{water} being the unit weight of water (62.4 lb/cf).

3.4.2 Fresh Concrete Tests

Fresh concrete tests were performed immediately after mixing in order to minimize the effect of settling on the test results. Once all tests were completed, samples were then prepared for hardened concrete tests. For measuring fresh properties of FRC, Slump (ASTM C143), Inverted Slump (ASTM C995), Vibrated J-Ring, Vibrated L-Box, and Unit Weight (ASTM C138) test were performed.

Slump tests are the most common test used to determine workability. This test was originally developed for measuring the consistency of fresh concrete (ASTM C143). However, a slump test is not always the preferred evaluator of workability for FRC since an FRC mix may be workable when vibration is applied despite having a low slump value (Folliard et al., 2006, Ostertag and Blunt, 2008). The slump test proved valuable in determining if the concrete had sufficient workability and consistency before adding fibers and comparing these characteristics with the concrete after adding fibers. The cohesiveness was also visually observed. This can be seen in Figure 3.5.

Good cohesiveness | Fair cohesiveness | Bad cohesiveness

Figure 3.5 Example of concrete with different slump values and cohesiveness

ASTM C995 (Standard Test Method for Time of Flow of Fiber reinforced Concrete Through Inverted Slump Cone) was used to measure the consistency and workability of FRC. Note that this test is withdrawn from ASTM testing methods due to a lack of use. This test is based on the time that a slump cone filled with concrete is emptied into a bucket located four inches below the bottom of the cone by applying vibration. In order to perform the test, the concrete must be stiff in such a manner that it can stay in the cone and not fall into the bucket before applying vibration. In this study, the test was used only in the preliminary stage, largely because most of the mixes included in the preliminary stage are not stiff enough to remain in the inverted slump cone before the vibration. Also, ASTM C995 does not provide any information about the passing ability of fibers through typical steel reinforcement. Another reason for electing not to continue usage of this test was related to macro fibers. Considering macro fibers were primarily used, the four-inch diameter hole in the slump cone was, in most cases, too small to allow for adequate passing.

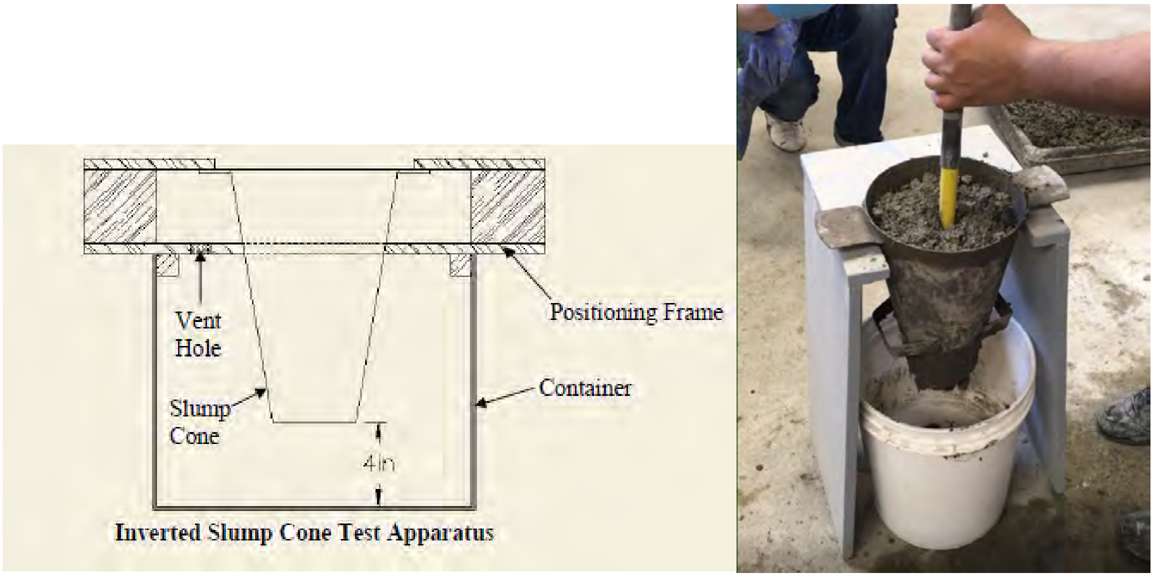


Figure 3.6 Inverted slump cone test

In order to overcome the aforementioned issues, the research team developed a pair of new tests for measuring the workability of FRC: Vibrated J-ring and Vibrated L-Box. Ultimately, it was decided to proceed with the Vertical L-Box test. However, both methods are detailed below.

The first test, the vibrated J-ring test, consisted of a J-ring with dimensions specified by ASTM C1621 (Standard Test Method for Passing Ability of Self-Consolidating Concrete by J-Ring) (ASTM C1621, 2014) and a vibrating rod. The J-ring was outfitted with an external cover around the bars to hold the concrete in before performing the test. This is not standard to the test, as the actual test is for self-consolidating concrete, and thus no vibration is used. The J-ring, before being filled, can be seen in Figure 3.7, in addition to the overall test procedure.

Figure 3.7 Vibrated J-Ring setup and test procedure

The passing ability of FRC through reinforcement is measured by taking the diameter after periods of vibration described in more detail later in this section. In addition, the stability and distribution of the paste can be evaluated through visual inspection (ASTM C1611).

The test is performed by dampening the ring and the nonabsorbent board under the ring. Then, the concrete is placed in the ring using a scoop by moving the scoop around the perimeter of the ring in order to distribute the concrete evenly (ASTM C143). The ring is filled to the top of the bars, with the cover still in place.

After preparing the sample, the cover was removed, and the concrete was vibrated in eight locations (one in the center, seven around the inner perimeter) within the ring in three to four-inch spacings. The vibrator is inserted in and pulled out of the concrete for approximately three seconds. After vibration is completed, the largest diameter (d1) of the concrete, and the diameter perpendicular to the largest diameter (d2), should be measured and recorded. In order to establish the reference indices, the test was performed on plane concrete and reference dimensions were measured. An average diameter of 17.75” was measured for plane concrete and was used for reference to evaluate other mixes.

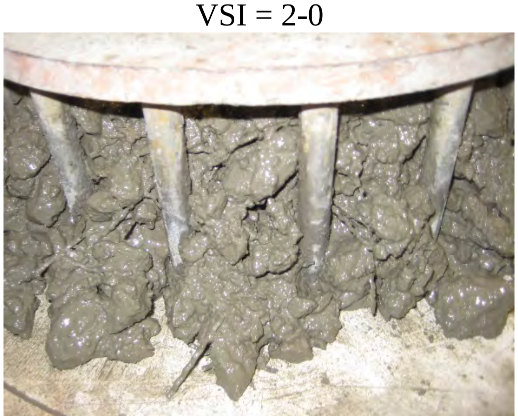
The distribution of paste, aggregate, and fibers were cohesive, and the stability of the mix could be evaluated by examining the concrete visually. Table 3.7 presents the criteria used to evaluate mixes by assigning values from 0 to 2 (similar to ASTM C1611). Each mix was evaluated using two numbers, one for stability and one for cohesiveness. For example, a mix with VSI 2-0 represented extensive bleeding while the aggregate and fibers were adequately covered with paste.

Table 3.7 Classification of Vibrated J-Ring Visual Stability Index (VSI).

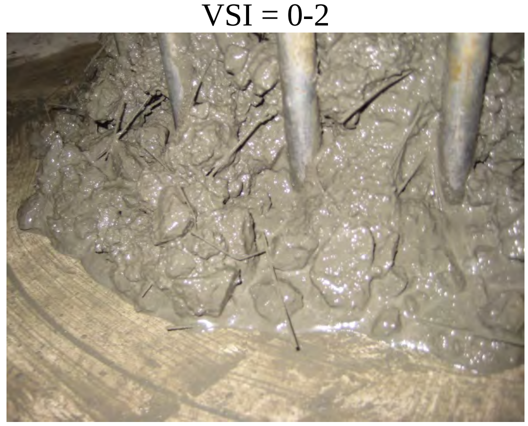
VSI	Stability	Cohesiveness
0	No bleeding	Aggregate and fibers covered with paste
1	Slight bleeding, no aggregate separation	Aggregate covered with paste, fibers covered fairly
2	Extensive bleeding and aggregate separation	Fibers not covered with paste

VSI = 2-0

VSI = 0-2



VSI = 0-0



VSI = 1-1

Figure 3.8 Examples for VSI categorization

The second test, which was elected to be used for the remainder of the project, was a vibrated L-Box test. The benefit of this test is the continuous downward pressure from the weight of the concrete, which is more representative of construction scenarios. Also, the test is better controlled, given that it has only one vibration point. A steel custom made L-box was used, the dimensions and details of which can be seen in Figure 3.9. A singular #5 rod was used and clamped in place to replicate #4 rebar at 3” spacing. Four-inch and six-inch marks are drawn at the floor of the L-Box. These are measured from the gate. The measurement in this test is simply the time it takes the concrete to flow to each of these marks once vibration has begun. These times are denoted as t_4 and t_6.

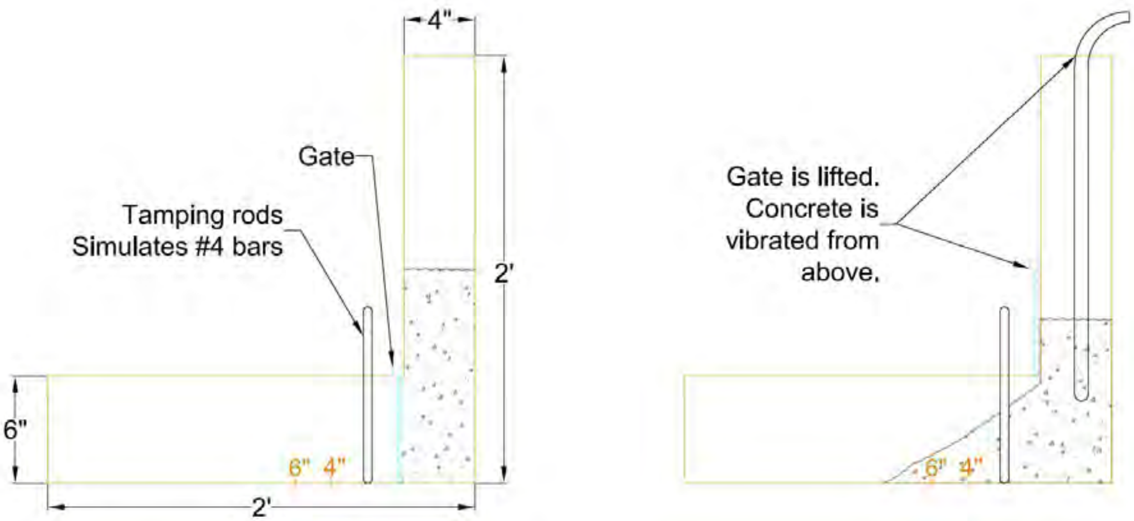


Figure 3.9 Sketch of L-box dimensions and test setup

To begin the test, concrete is scooped into the vertical chute of the L-box with the gate closed. The height of the concrete relative to the floor should be approximately halfway (~12") up the chute. The gate is pulled up, and immediately thereafter, a vibrating rod is inserted from the top of the L-box. The rod is inserted slowly until it is approximately 1 inch above the floor of the box. Care is taken to avoid contacting the walls and floor with the vibrating rod. Vibration is stopped once the concrete has clearly passed the 6" mark on the floor of the L-box. The setup and test processes are shown in Figure 3.10.

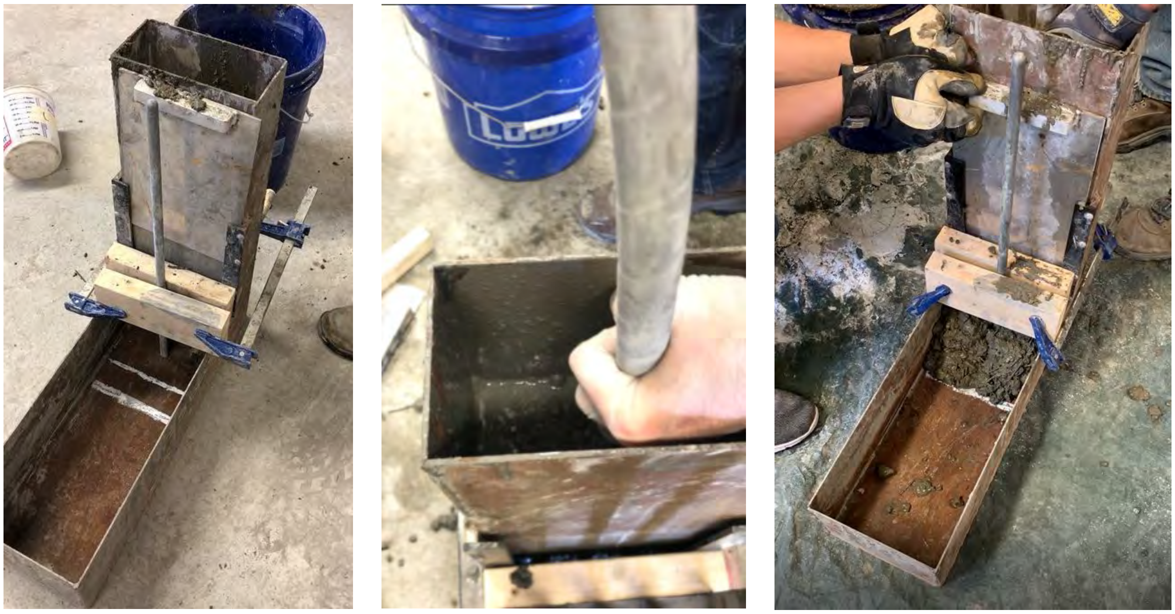


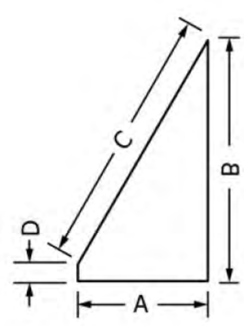
Figure 3.10 Vibrated L-box test for passing ability

3.4.3 Specimen Casting and Curing

After completing fresh concrete tests, 4"x8" cylinders and 6"x6"x20" beams were made according to ASTM C192. The 4”x8” cylinders were cast in two layers, with vibration at each layer, as per ASTM C192. A study on the effect of rodding versus vibrating fiber reinforced concrete cylinders with regard to splitting tensile strength determined that vibration can increase the strength of FRC by a substantial amount, which is believed to be due to a more uniform paste distribution and thus improved bonding between the paste and the fiber (Shaaban and Gesund, 1993). When vibrating, the rod is inserted approximately 1” into the layer and is held in position for approximately 4 seconds. Following the vibration of the upper layer, the sides of the mold were patted with a cupped hand to close any pore created by the vibrating rod. After completing this form on the second layer, the top is then leveled off with a trowel.

Beams were also cast in two layers. The vibrating rod is again inserted at an approximate 1” depth but at six locations: the corners and the 1/3rd points along the longitudinal centerline of the beam. The sides of the mold are then tapped lightly with a rubber mallet to close any pores created by the vibrating rod and to collapse large internal air pockets. This process is repeated for the second layer before leveling off the surface. The samples were demolded 24 hours after casting and were placed in the curing room at 73.5± 3.5 ^{o}F with 100% relative humidity. The cylinders were used to measure 7-day and 28-day compressive strength and splitting tensile strength according to ASTM C39 (Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens) and ASTM C496 (Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens), respectively. The beams were used for flexural tests according to ASTM C78 (Flexural Strength of Concrete Using Simple Beam with Third-Point Loading) and ASTM C1609 (Flexural Performance of Fiber Reinforced Concrete Using Beam with Third Point Loading).

Slant shear tests, according to ASTM C882 (Standard Test Method for Bond Strength of Epoxy-Resin Systems Used with Concrete by Slant Shear) were performed to measure the bond strength of the concrete. The Nebraska state bridge deck mix design (47BD) detailed later in Table 4.1 was used as the base concrete, while Reno and Las Vegas materials (both plane and with fibers) were used as the control concrete. Sixteen 4”x8” Nebraska cylinders were cast. After the same curing method was used for the cylinders mentioned at the beginning of this section, the cylinders were cut with a concrete saw in the fashion detailed in Figure 3.11.



	Dimensions mm	in.
A—Diameter	75±2	[3.00 ± 0.08]
B—Height	140±2	[5.60 ± 0.08]
C—Slant height	150±2	[6.00 ± 0.08]
D—Base height	10±2	[0.40 ± 0.08]

Figure 3.11 ASTM C882 specified slant shear cutting dimensions

Three ¼” notches were then cut into each cylinder to improve bonding. An example of a cut cylinder can be seen in Figure 3.12. These notches were cut perpendicular to the previously cut surface.

Figure 3.12 Typical slant shear base concrete specimen

After a minimum of 28 days of curing, the top concrete was cast and kept in the mold under wet towels and plastic wrap for 7 days. Once demolded, the composite cylinder is cured for an additional 21 days (in the same manner as a normal cylinder).

In addition to the cylinders and beams, several other specimens were cast for durability testing including restrained shrinkage rings, free shrinkage prisms, and freeze-thaw prisms. The details for the casting of each of these specimens is detailed in the durability section of this report (3.4.5).

3.4.4 Mechanical Property Tests

Depending on the fiber material, length, diameter, deformation geometry, and the volume percentage, typical mechanical properties can be slightly or significantly improved.

A standard compressive test using ASTM C39 was carried out for the majority of the cylinders. 7-day and 28-day compressive strength tests were performed with 4”x8” cylinders. These cylinders were loaded in a standard Forney testing machine at a rate of 440±50 lb/sec, which falls within a broader range specified in the standard. The smaller range was used to keep consistency between all cylinders. Compressive strength was then calculated with the following equation:

$$f_c' = \frac{P}{A} \tag{3.4}$$

where f_c' is the compressive strength, P is the peak load, and A is the cross-sectional area of the 4”x8” cylinder (12.56 in^2).

The test setup can be seen below in Figure 3.13.

Equipment FRC Normal Concrete

Figure 3.13 Compressive strength test

A splitting tensile test was also performed at 7 and 28 days according to ASTM C496 (Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens). As stated in the standard, bearing strips were used to distribute the load on the concrete surface. The loading rate was 120±20 lb/sec. Just as with the compressive strength test, this rate is well within the broader range defined by the standard. The splitting tensile strength was calculated using the following equation:

$$T = 2P/\pi l d \tag{3.5}$$

where T is splitting tensile strength, P is the maximum applied load, l is the length of the specimen, and d is the diameter of the specimen. This equation represents the load divided by the surface area of half the cylinder (excluding top and bottom). The test setup can be seen in Figure 3.14.

Figure 3.14 Test setup for splitting tensile strength

A flexural strength test was performed according to ASTM C1609 (Standard Test Method for Flexural Performance of Fiber reinforced Concrete Using Beam with Third-Point Loading). The 6”x6”x20” beams were set up for 3rd point loading with an 18-inch span. A servo-controlled testing machine was used to control testing by displacement. Two linear variable differential transformers (LVDT) were attached to the sides of the beam to measure deflection through contact with the bracket attached to the top of the specimen. The average of the measurements represents the net deflection. The LVDTs were connected to a Data Logger shown to record the data at a frequency of 25 Hz (0.02 seconds between readings). Figure 3.15 Specimen test setup and data logger.Figure 3.15 shows the test setup and data logger.

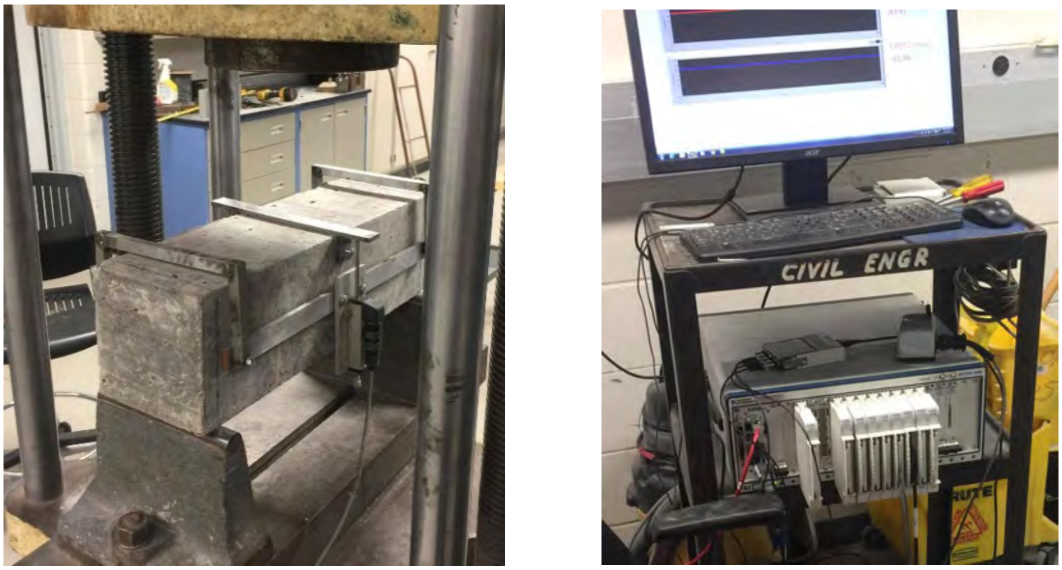


Figure 3.15 Specimen test setup and data logger.

The schematic of the test setup can be seen in Figure 3.16.

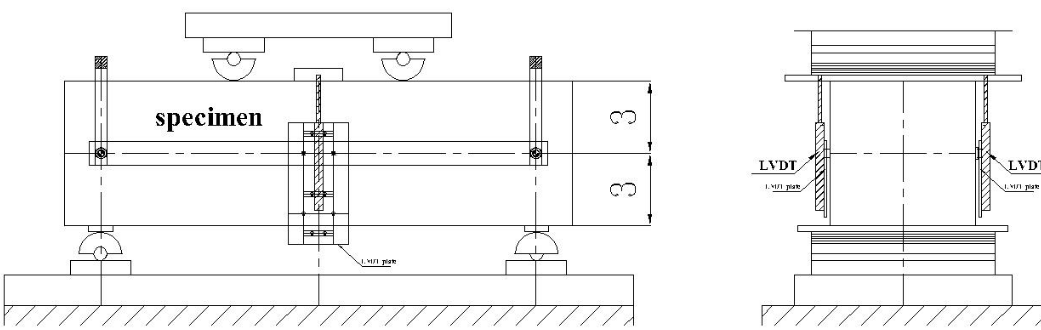


Figure 3.16 Schematic view of the flexure test setup

The specimen was pre-loaded to 10% of the expected peak load. The test was then started at rate of 0.002 to 0.004 in/min until the specimen reached a net deflection of (L/900) or 0.02 inches. After a net deflection of 0.02 inches, the rate was increased to a range of 0.008 to 0.012 inch/min for the remainder of the test. The test was terminated when a total net deflection of (L/150) or 0.12 inches, was reached.

The load history from the machine and the deflection data from the LVDT's were combined to create a load-deflection curve. Toughness, which is the area under this curve, is a very important aspect of FRC. It is defined as the ability of concrete to absorb energy and plastically deform without a full fracture. In accordance with ASTM C1609, the toughness was recorded under the entire curve i.e., to a deflection of L/150. A typical FRC deflection curve and the toughness area can be observed in Figure 3.17.

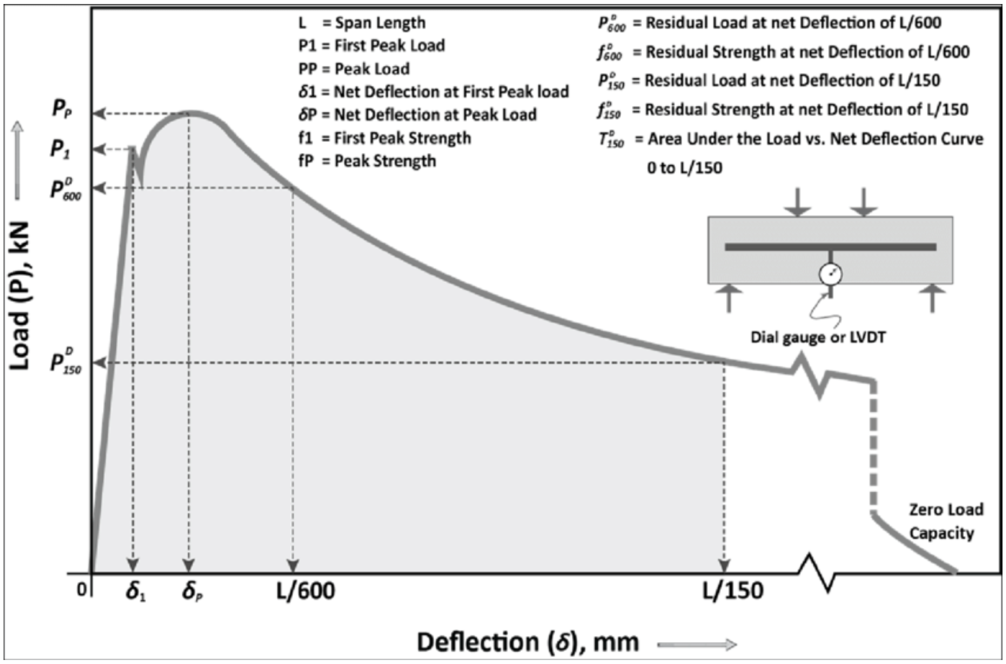


Figure 3.17 Typical FRC load-deflection curve and typical toughness area (adopted from Hamad and Sldozian 2019)

In addition to the toughness, ASTM describes several other values to be obtained from the graph, each of which is visible in Figure 3.17. The first peak load (P_1), which is the point just before the beam endures its first crack (whether micro or visible) is recorded. The actual peak load (P_P), which in many cases will surpass the first peak load, is recorded as well. Two residual loads are recorded, one at L/600 (P_{600}) and one at L/150 (P_{150}). The Modulus of Rupture (MOR) was calculated using the following equation provided by ASTM C1609:

$$f = \frac{PL}{bd^2} \tag{3.6}$$

where f is strength (ksi), P is peak load (kips), L is span length (inches), b is width (inches), and d is depth (inches).

An important property of any concrete, but FRC in particular given its common usage in beam connections, is the bond strength. To quantify this, slant-shear tests were performed following the standard for compressive testing (ASTM C39) as specified by ASTM C882.

3.4.5 Durability and Volume Stability Tests

Durability tests made up the final portion of the experimental program. The addition of fibers can help to mitigate shrinkage issues common to concrete. More specifically, fibers can reduce plastic shrinkage cracking and drying shrinkage cracking. After cracking, fibers are believed to transfer tensile stress across cracks and act to arrest or confine crack tip extension so that many fine micro-cracks occur instead of larger cracks (Shah and Weiss, 2006).

Drying shrinkage cracking is a frequent issue with concrete structures. These cracks can lead to early rebar corrosion and chloride penetration. A restrained shrinkage ring can replicate this issue and give a comparison of crack resistance of different concretes. Restrained shrinkage rings were cast and tested as described in ASTM C1581 (Standard Test Method for Determining Age at Cracking and Induced Tensile Stress Characteristics of Mortar and Concrete under Restrained Shrinkage). One concrete ring per mixture was cast for restrained shrinkage tests. The testing device includes a 13” inside diameter ring surrounded by a 16” outside diameter ring mounted on an impermeable board to ensure appropriate consolidation. The concrete was rodded in two layers and rodded 75 times each layer. After rodding of the top layer, the mold and specimen were placed on a vibrating table and were vibrated for approximately 10 seconds. The specimen was then finished with a trowel. A sample specimen is seen in Figure 3.18. The specimens were stored in an environmental chamber with 73.5±3.5 ^{o}F (23.0±2.0 ^{o}C) temperature and 50±4.0% R.H., and the strain gauges were immediately connected to the data logger. These gauges can be seen in Figure 3.18 as well. The gauges are connected to the inside of the inner steel ring using a special adhesive. The rings were then cured for the first 24 hours under damp towels and plastic wrap. Once the initial curing concluded, the outer ring was removed, and the concrete was coated with wax on the top surface. The strain was measured for 28 days or until the stress release was noticed due to concrete cracking with normal concrete. FRC rings were allowed to proceed with testing after cracking (if applicable) to examine post-crack shrinkage properties. The strain readings were taken every hour and monitored for sudden strain reduction. A sudden reduction of strain greater than 30 microstrains can be considered cracking. The age at which cracking occurred was reported to the nearest 0.25 day.

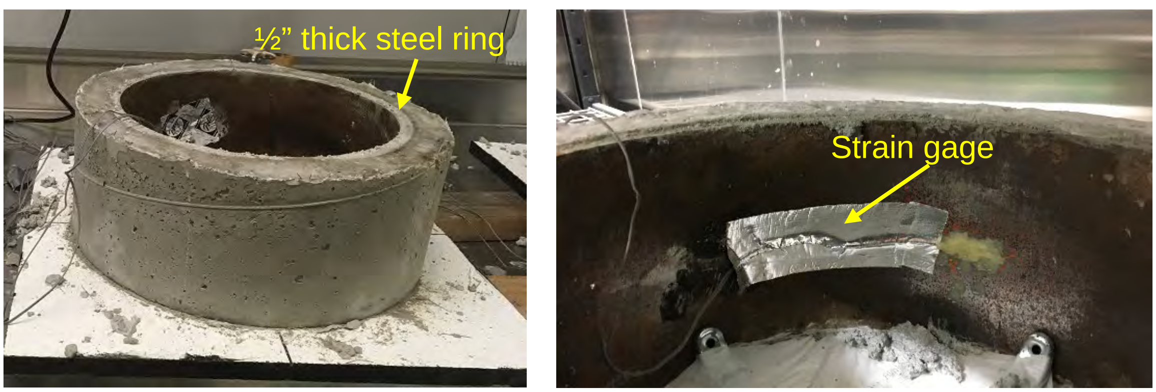


Figure 3.18 Restrained shrinkage test set up

Two free shrinkage prisms were cast for each mix design in order to monitor the volume stability of fiber reinforced concrete relative to plane concrete. Based on ASTM C157 (Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete), 3” by 3” by 11.25” concrete prisms were cast. These prisms were rodded twenty-five times in two layers, tapped four times on the sides with a mallet, and finished with a trowel. The prisms were cured for 24 hours under wet towels and plastic wrap. The prisms were placed in a water bath immediately after demolding. At 28 days, the prisms were taken to an environmental chamber with 73.5±3.5 ^{o}F (23.0±2.0 ^{o}C) temperature and 50±4.0% R.H. Using a length comparator (Figure 3.19), the change in length of each prism was measured. These measurements were taken at 0 days, 3 days, 7 days, 14 days, 28 days, 42 days, and 56 days.

Figure 3.19 Free shrinkage specimen length measured by length comparator

A freeze/thaw test was conducted following ASTM C666 (Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing). This test was particularly important for northern Nevada but was conducted with both Reno and Las Vegas materials. 3” by 4” by 16” prisms were cast in the same manner as the free shrinkage prisms. Using a Humboldt freeze/thaw cabinet (Figure 3.20), the prisms were exposed to freeze/thaw cycles for two months beginning after 14 days of curing. At the end of every 30 cycles, the mass loss of the specimens was recorded, and the relative dynamic modulus of elasticity was calculated. In order to perform this calculation, the fundamental transverse frequency was needed. It was obtained using an NDT E-meter, seen in Figure 3.20. The equation for the relative dynamic modulus is as follows:

$$P_c = \left(\frac{{n_1}^2}{n^2}\right) * \ 100 \tag{3.7}$$

where P_c is the relative dynamic modulus of elasticity after c cycles of freezing and thawing (%), n is the fundamental transverse frequency at 0 cycles, and n_1 is the fundamental transverse frequency after c cycles.

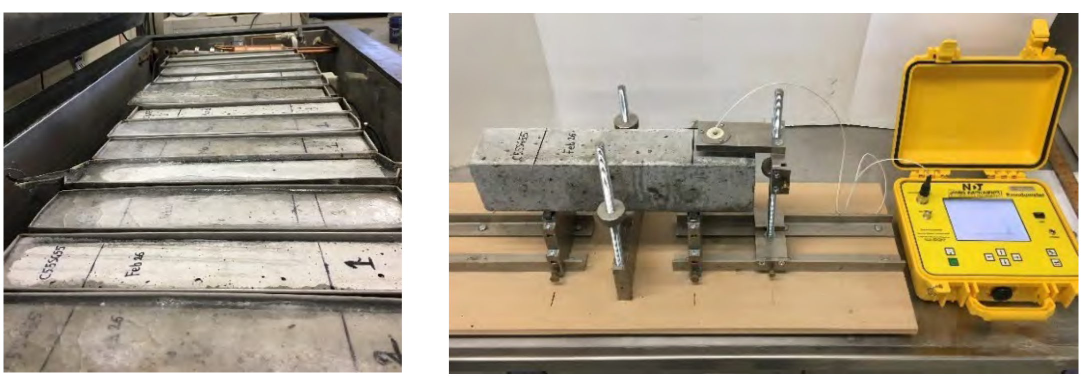


Figure 3.20 Freeze/thaw chamber and NDT E-meter

CHAPTER 4. EXPERIMENTAL PROGRAM

4.1 Introduction

The experimental program consisted of three separate phases. The aim of the initial phase, referred to as the “screening phase”, was to narrow down fiber candidates to one or two fibers that presented the most promising results. The second phase, the “mix adjustment phase”, involved the determination of a mix design based on a developed adjustment method that would account for the addition of fibers. Once promising designs were found, the “performance evaluation phase” concluded the program by studying the results of mixes under more advanced tests such as rapid chloride permeability and drying shrinkage tests.

Two standard mix designs commonly used in Nevada bridge decks in the Reno and Las Vegas areas were provided by NDOT for use in this study as reference mixes. These two designs (5000 EA Modified from Reno and Class E Modified from Las Vegas), were mixed several times for result comparison with FRC mixes. In addition to these two reference mixes, a Nebraska state mix commonly used on bridge decks, called 47BD, was provided by the Nebraska Department of Transportation. This mix was used for large scale casting in which the use of Nevada materials would not have been practical. A secondary benefit to the provided Nebraska design was another set of materials for use with the developed adjustment method. The standard specifications of these designs can be seen below in Table 4.1. Adjustments were then made to these reference mixes to account for increasing fiber content. For the screening phase, a simple adjustment method was used for a fair comparison between fibers. In the adjustment phase, the adjustment method and control parameters were improved upon to establish mixes that would maximize performance. The details of the various adjustment procedures are detailed later in section 4.3.

Table 4.1 State mixes provided by Nevada and Nebraska DOTs

Location	**Reno**	**Las Vegas**	**Omaha/Lincoln**
Mix	5000 EA Modified	Class E Modified	47BD
Specified Strength, psi	5000	4500	4500
Specified Slump, in.	6.0	0.5-4	N/A
Specified air, %	4-7%	1.5-4.5%	6.0-8.0%
Cement, pcy	529	504	658
Fly ash, pcy	176	126	0
Total cementitious material, pcy	705	630	658
w/b	0.38	0.41	0.39
NMAS, in.	3/4	3/4	1

4.2 Phase I Study (Screen Phase)

4.2.1 Mix Designs

In the screening phase, mixes were carried out with the selected fibers (Figure 3.1) using three different dosages of fibers (low, medium, high). The aim of Phase I was to narrow down the selection of the type(s) of fiber with which to proceed. The dosages were selected based on the producer recommendations and information obtained from the literature review. Fresh properties (slump and unit weight), as well as mechanical properties (compressive and flexural strength), were obtained and necessary modifications were then applied to the NDOT mix designs in order to achieve workable mixes that would meet mechanical and durability requirements.

Las Vegas material was used throughout this phase. Eleven mixes, in addition to a reference mix based on the design provided by Sierra Ready Mix, were performed. These mix designs can be seen in Table 4.2. The mix ID's are in the following format:
"location - fiber used - fiber dosage". For example, LV-Mac-0.5 denotes a mix with Las Vegas materials and Mac Matrix fibers at 0.5% dosage. The LV reference mix was cast based on the mix design provided by Sierra Ready Mix.

Table 4.2 Mix proportioning of Las Vegas material study.

		Type V Cement	F Fly ash	#67	#89	FA	Fiber	Water	WR (fl oz/cy)	w/b	Fiber Vol%
LV-Ref		504	126	1692	182	1225	0	252	28	0.40	0
Steel Fibers	LV-3D-2	516	129	1591	171	1232	262	253	32	0.40	2
	LV-Helix-2	511	127	1602	172	1230	256	253	32	0.40	2
	LV-OL-2	516	129	1591	171	1232	256	253	32	0.40	2
	LV-T5-1	515	129	1602	172	1252	132	258	32	0.40	1
	LV-T5-2	516	129	1591	171	1232	262	253	32	0.40	2
	LV-Hyb-2-1	518	128	1615	174	1241	132	255	32	0.40	1
Synthetic Fibers	LV-Mac-0.5	524	131	1615	174	1250	8	257	33	0.40	0.5
	LV-MacCB-0.5	524	131	1615	174	1250	8	257	33	0.40	0.5
	LV-Omni-0.5	524	131	1615	174	1250	8	257	33	0.40	0.5
	LV-CEM-1	521	130	1607	173	1244	34	256	33	0.40	1
	LV-CEM-2	521	130	1588	171	1230	67	253	33	0.40	2

The reference mix needed to be modified in order to include fibers since the addition of fibers reduces workability (in the case of steel fibers) and reduces unit weight (in the case of synthetic fibers) (Narayanan and Kareem-Palanjian, 1982; Afroughsabet et al., 2016). For this purpose, up to 100 lbs of coarse aggregate per cubic yard of concrete were reduced (Folliard et al., 2006) and replaced by fine aggregate and cementitious materials. The fine aggregate-to-cementitious materials ratio was held constant. The dosage of admixture suggested by Sierra Ready Mix was reduced from 32 oz/cy to 28 oz/cy for the reference mix considering the desired workability was achieved with a slump of 3.25". The dosages were kept at 32 oz/cy for the FRC mixes, however. It should be noted that the total amount of fine and cementitious material that was increased was not equal to the amount of reduced coarse aggregate since the specific gravity of coarse aggregate, fine aggregate, cement, and fly ash were different. Lastly, a more efficient adjustment method, called the excess paste adjustment method, was used in Phase II.

4.2.2 Results

The results of fresh concrete tests from Phase I are presented in Table 4.3. Steel fiber reduced slump of concrete significantly (by 60 to 70%), while synthetic fibers had a varying effect on the slump of FRC. In all synthetic fiber mixes, however, the slump was reduced by at least half, indicating that synthetic fibers aren't necessarily that much more efficient in workability conservation than steel fibers. In addition, unit weight of FRC made with steel fibers slightly increased compared to the reference mix while mixes made with synthetic fibers had slightly lower unit weight.

Table 4.3 Fresh concrete properties of mixes with Las Vegas materials.

		Slump before Fiber (in.)	Slump after Fiber (in.)	Fresh Unit Weight (pcf)	Vibrated J-ring Flow (in)	Vibrated J-ring VSI	Cohesiveness	Finishability	Compactability
LV-Ref		3.25	N/A	150.6	17.75	0	Good	Good	Good
Steel Fibers	LV-3D-2	5	0.5	132.5	15	NA	Bad	Bad	V Bad
	LV-Helix-2	5	0.25	145	15	NA	Bad	Bad	V Bad
	LV-OL-2	5	0.25	132	14.5	NA	Bad	Bad	Bad
	LV-T5-1	5	1.5	151	16.5	0	Good	Good	Good
	LV-T5-2	5	0.75	138	15	NA	Bad	Bad	V Bad
	LV-Hyb-2-1	4.75	0.75	146	17	2	Good	Good	Good
Synthetic Fibers	LV-Mac-0.5	5	2.5	144	18	1	Good	Good	Good
	LV-MacCB-0.5	6	2.75	147	18	1	Good	Good	Good
	LV-Omni-0.5	4.25	0.75	148	17	1	Good	Good	Good
	LV-CEM-1	5	0.75	147	16.5	0	Good	Good	Bad
	LV-CEM-2	4.75	0.5	141	15	0	Fair	Bad	V Bad

In order to identify a type and dosage of fiber with the most promising results, higher dosages of steel fiber were used for certain mixes. For example, given the relatively high slump of 1.5" after adding 1% of T5 fibers, it was elected to attempt this mix with 2% of fibers. The slump of concrete decreased significantly by adding 2% of steel fibers in all cases, while this decrease was not as significant when synthetic fibers were used. However, the slump result for T5 fibers at 2% dosage was still higher than other steel fiber mixes with a 2% fiber dosage and was comparable to some of the synthetic fiber mixes. In addition, the unit weight of the FRC in concrete with 2% of steel fibers was lower than concrete made with 1% of steel fibers. From a visual inspection standpoint, none of the mixes made with 2% of steel fibers were flowable when vibration was applied, and the fibers were not covered with paste (Figure 4.1).

Figure 4.1 Example of FRC made with an insufficient paste

An effort was made to make the mix with Dramix 3D fibers (which have the highest aspect ratio of the steel fibers) more flowable by using a higher dosage of water reducer, which increased the slump to 4.5” after addition of fibers. However, the mix was not stable, and separation of paste and fiber/aggregate was observed, which resulted in a VSI of 2-0 (see Figure 4.2). This suggests that simply adding more water reducer will not necessarily make the mix workable and may lead to significant segregation and surface voids. Thus, the mix design should be modified.

Figure 4.2 Effect of excessive water reducer on stability

The mixes made with synthetic fibers were easier to work with compared to mixes with steel fiber, and they had workability similar to plain mixes. Mac-0.5 and Omni-0.5 showed slight bleeding when the modified J-ring test was performed, while CEM-1 did not have any bleeding. In addition, CEM micro rebar improved mechanical properties of concrete more so than other synthetic fibers and had the good passing ability. This shows that an FRC mix may still be workable even if the slump is low and that slump is not necessarily a proper evaluation of the workability of FRC.

In order to confirm the applicability of the vibrated J-ring test on mixes with steel fibers, LV-T5-1 was made with Las Vegas materials. The mix had good flowability and passing ability. In addition, the fibers were covered by paste, and no bleeding was observed.

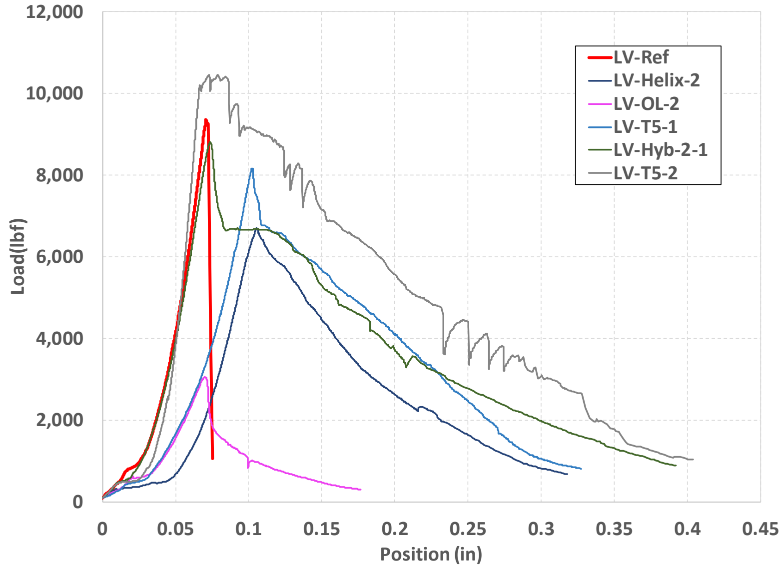
The results from compressive and flexural tests can be seen in Table 4.4. The compressive strength results indicated that synthetic fibers would typically have a neutral or negative impact, while steel fibers may have a neutral or slightly positive effect without any form of adjustment. For example, the 7-day compressive strength of FRC with 0.3% synthetic fibers was reduced in comparison to the plain mix, while steel fibers increased the compressive strength.

It should be noted that the flexural toughness was calculated using the flexural machine’s displacement monitor rather than Linear Variable Differential Transformers (LVDTs), which were used in Phase II. This is the reason for the higher toughness values seen in Phase I results compared to Phase II. An LVDT (or in the case of this study, a pair of LVDTs) will provide much more accurate toughness results as this device measures deflection rather than overall beam displacement. The test is intended to be concluded at 0.12” deflection, and in order to get FRC type results in which the load stabilizes after an initial crack, the machine’s displacement is carried on much further than this. However, for the sake of comparing the efficiency of fibers, the machine's monitor was sufficient.

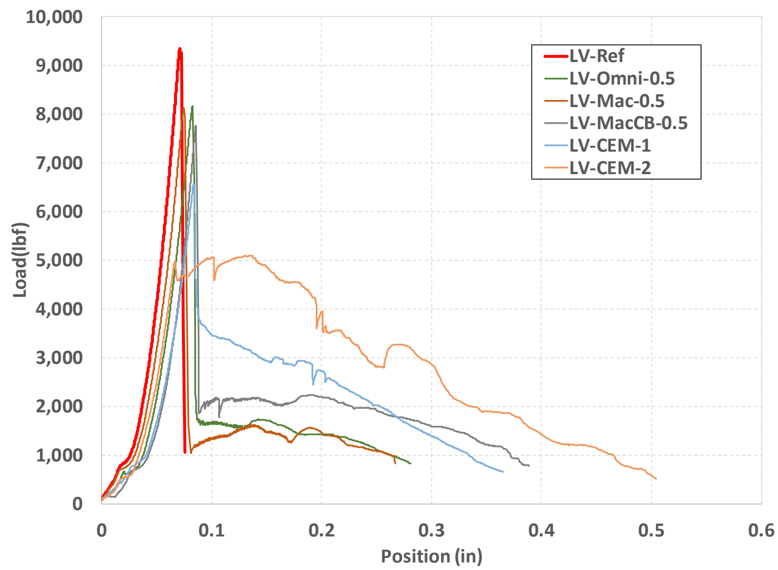
Table 4.4 Hardened properties of mixes with Las Vegas material.

		$f'_{c,7}$ (psi)	$MOR_{,7}$ (psi)	$Toughness_{,7}$ (lb-in)
LV-Plane		4703	779	NA
Steel Fibers	LV-3D-2	2679	478	761
	LV-Helix-2	2898	558	721
	LV-OL-2	2308	255	146
	LV-T5-1	5688	751	505
	LV-T5-2	6482	871	2612
	LV-Hyb-2-1	4207	734	1193
Synthetic Fibers	LV-Mac-0.5	5008	678	NA
	LV-MacCB-0.5	4580	648	521
	LV-Omni-0.5	4370	680	NA
	LV-CEM-1	4427	548	702
	LV-CEM-2	4945	425	1280

Figure 4.3 shows the results of the flexural tests for each type of fiber.



Steel Fibers



Synthetic Fibers

Figure 4.3 Flexural strength test results from Phase I LV mixes

The synthetic fiber reinforced mixes had very low load capacity after initial cracking in comparison to the steel fiber mixes. This was likely because of the low modulus of elasticity of synthetic fibers and the sporadic distribution of fibers in the cross-section. A broken cross-section (Figure 4.4) with Omni fibers confirmed this.

Figure 4.4 Synthetic fiber distribution in the cross-section of the beam

The mixes made with steel fibers were able to carry load after the peak load by bridging the cracks. Overall, LV-T5-2 carried the highest peak load, highest load after cracking, and, as a result, the highest toughness of all fibers tested. The 7-day compressive strength of all mixes made with steel fibers decreased with increasing dosage of fibers to 2% compared to mixes with 1% percent fiber, which was the result of high void content in the samples. The toughness of all steel FRC with 2% fiber decreased when compared to mixes with 1% steel fibers (with the exception of LV-T5-2). When combining results of workability, toughness, modulus of rupture, and minimal loss or even improvement of compressive strength, T5 fiber was deemed the most promising for the adjustment study in Phase II. The phase, in addition to applying an adjustment method, will involve adjusting fiber content from 0.0% to 2.0% in increments of 0.5%.

4.3 Phase II Study (Mix Adjustment Phase)

4.3.1 Mix Design Adjustment Method

A pair of adjustment methods were adopted to determine an appropriate method that would maintain the fresh concrete properties of the reference mixes while improving the mechanical properties. As was illustrated in Phase I, when the fiber is introduced, and all other proportions are maintained, a significant drop in workability can be noted. Thus, two adjustment methods were adopted: maintaining paste-to-fine aggregate and maintaining excess paste. Maintaining excess paste yielded far more positive results, and the process for this method is detailed in this section. Though the paste-to-fine aggregate method yielded poor workability results, an important observation was made regarding fiber content. That is, the fiber volume for T5 fibers yields the best balance between workability and toughness when using 1.5% of fibers. No fiber volumes between 1% and 2% were attempted in the previous phase, and this second phase created an opportunity to analyze fiber performance with more specific fiber contents. The most important aspect of this particular volume of fiber is that it improves flexural strength over the 1% content, while 2% barely increases strength over 1.5%. With this option, the benefits of 2% of fiber are preserved, and workability is improved at a more economical rate.

Maintaining excess paste is effectively a three-step process. This process is summarized in Figure 4.5, in which the last two boxes are part of the same step. The core of this design method is to increase the fine aggregate-to-total aggregate ratio, increase paste, and then decrease overall aggregate content to the point that excess paste is the same for the reference, 0.5%, 1.0%, 1.5%, and 2.0% mixes. Throughout this adjustment process, the water-to-binder ratio is maintained, as are the air content and the admixture content.

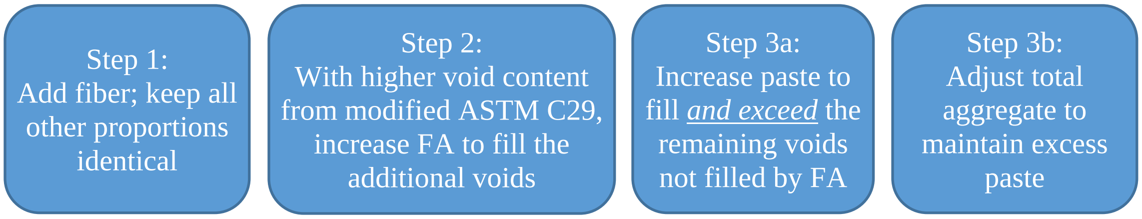


Figure 4.5 Summary of excess paste adjustment method

Each step is fully detailed below with Las Vegas materials used as an example. The Mix-ID's for these designs are simply the name of the source, followed by a value of either 0.5, 1.0, 1.5, or 2.0 to represent fiber content. Air content and admixtures are not changed throughout the design adjustment and, therefore, are not shown in the mix design tables.

Step 1: Introduce fiber

The initial step of the adjustment method is very simple. All that is done in this stage is introducing fiber into the mix design and factoring down all other ingredients so that the design adds up to 27 cubic feet. The following set of tables shows the minor changes between the materials with increasing fiber percentages. Noted that excess paste is shown in this step but is not actually calculated until Step 2. These values were calculated retroactively and are only shown for reference with the final mix designs.

Table 4.5 Mix designs after Step 1 of excess paste adjustment method

Mix ID	Unit	Type V Cement	Class F Fly Ash	Water	#67 C.A.	#89 C.A.	F.A.	T5 Fiber	Excess Paste
LV-Ref	pcy	501	125	250	1682	181	1218	0	**3.01%**
	cf	2.55	0.88	4.01	10.06	1.12	7.45	0.00	
LV-0.5	pcy	498	125	249	1673	180	1212	66	**2.60%**
	cf	2.54	0.87	3.99	10.01	1.11	7.41	0.14	
LV-1.0	pcy	496	124	248	1665	179	1205	132	**0.75%**
	cf	2.52	0.87	3.97	9.96	1.11	7.37	0.27	
LV-1.5	pcy	493	123	247	1657	178	1199	198	**-3.09%**
	cf	2.51	0.86	3.95	9.91	1.10	7.34	0.41	
LV-2.0	pcy	491	123	245	1648	177	1193	265	**-6.20%**
	cf	2.50	0.86	3.93	9.86	1.10	7.30	0.54	

Increasing fiber content will increase the void content considerably. Fibers interfere with the concrete matrix and push apart the coarse aggregate and, to a lesser extent, the smaller particles. Figure 4.6 illustrates the negative effect that ensues from simply introducing fiber and making no adjustment.

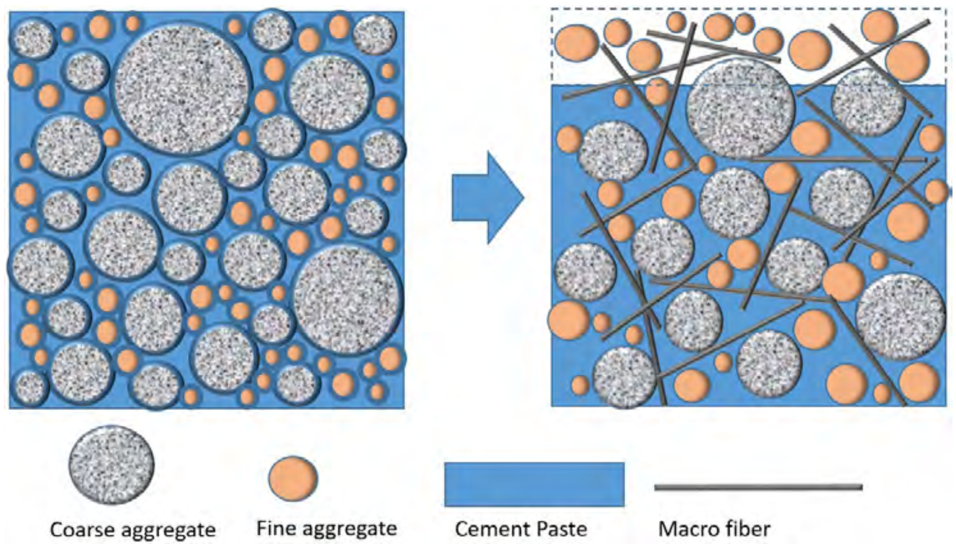


Figure 4.6 Effect of fiber on the concrete matrix

Step 2: Determining void content and adding additional fine aggregate

The second step of the adjustment method is the most involved. Before it can be carried out, the void content and bulk density of the fiber-aggregate blend needs to be determined using the modified ASTM C29 standard detailed in section 3.4.1. The Las Vegas results of this test can be seen below.

Initially, this test was performed with several intervals of fiber (twelve equally increasing intervals of F/A ratio) to ensure a trend was present. The results of this preliminary set of tests can be seen in Figure 4.7.

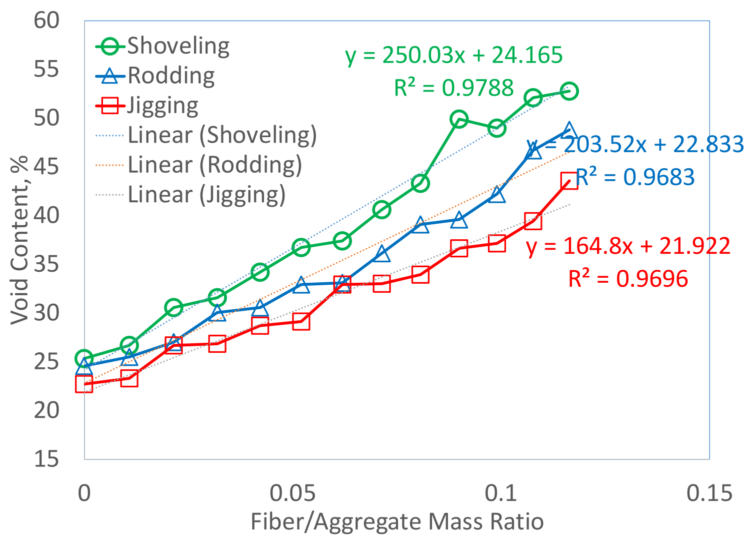


Figure 4.7 Results from preliminary fiber-aggregate skeleton void content test

With the preliminary results showing a clear linear trend as expected, the test was performed for Reno, Las Vegas, and Lincoln aggregates, but with increasing fiber intervals of 0.5% by volume.

As can be seen in Figure 4.8, the higher the fiber dosage, the more visible the voids in the blend. The text in each image in the figure indicate the fiber-to-aggregate ratio.

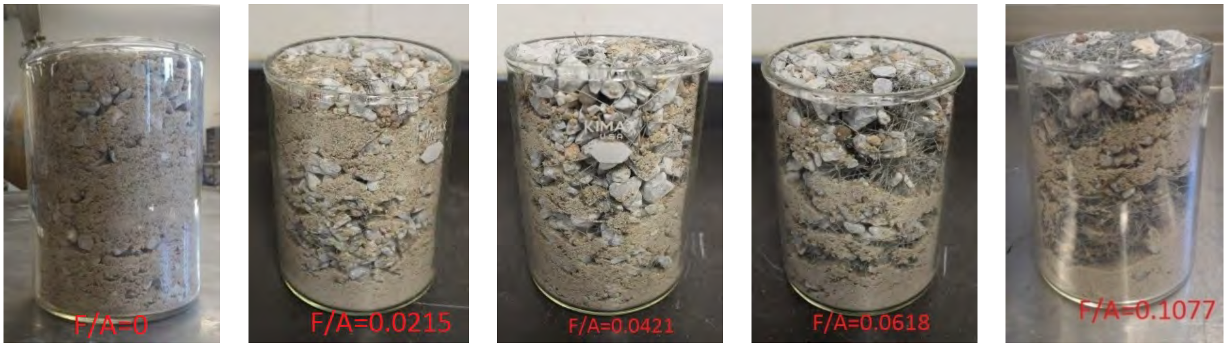


Figure 4.8 Appearance of Las Vegas blends with increasing fiber content

The results from Las Vegas testing using T5 fibers can be seen in Figure 4.9.

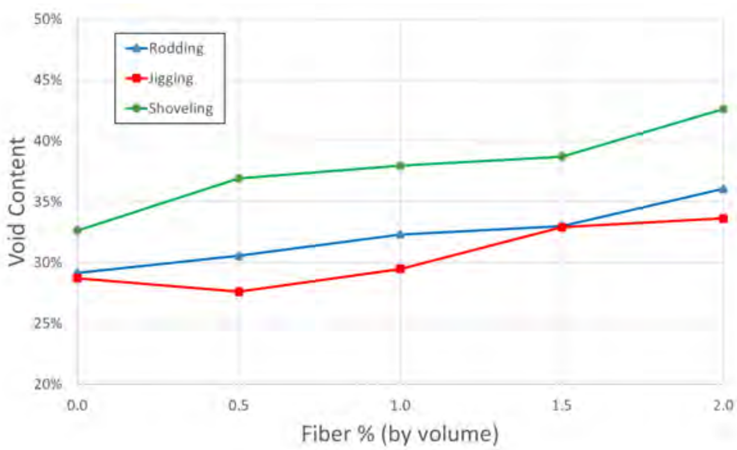


Figure 4.9 Las Vegas T5 fiber void content

Furthermore, since the test must be conducted in the dry state, an SSD correction factor is applied to relate the void content to the SSD state. To determine this factor, it is necessary to calculate the dry aggregate quantities using the values from Step 1 and the absorption percentage of each aggregate. The equation for the factor is detailed below the table. The results for Las Vegas are displayed in the table below.

Table 4.6 Modified ASTM C29 fiber-aggregate void content results – Las Vegas

Method		LV-Ref	LV-0.5	LV-1.0	LV-1.5	LV-2.0
Jigging		28.70%	27.58%	28.51%	32.05%	32.37%
Rodding		29.14%	30.54%	31.37%	32.14%	34.85%
% Void Ave.		28.92%	29.06%	29.94%	32.10%	33.61%
Bulk Density, lb/ft^3		117.56	118.96	117.44	113.84	111.28
Dry Agg. Weight, pcy	#67	1666.85	1658.52	1650.18	1641.85	1633.51
	#89	178.41	177.52	176.63	175.73	174.87
	FA	1217.54	1211.46	1205.37	1199.28	1193.19
Dry Agg. and Fiber Volume, ft^3	#67	9.97	9.92	9.87	9.82	9.77
	#89	1.10	1.10	1.09	1.09	1.08
	FA	7.45	7.41	7.37	7.34	7.30
	Fiber	0.00	0.14	0.27	0.41	0.54
Combined Dry Aggregate and Fiber Volume, ft^3		18.52	18.57	18.60	18.66	18.69
Moisture Correction Factor		0.97	0.97	1.00	1.05	1.09
Corrected Void %		28.02%	28.28%	29.99%	33.69%	36.65%
Difference from Reference Mix		N/A	0.26%	1.97%	5.67%	8.64%

The correction factor is calculated with the following equation:

$$CF = \frac{(W_{CA1}+W_{CA2}+W_{FA}+W_{fiber})/\rho_{blend}}{27-V_{SSD}+V_{dry}} \tag{4.1}$$

where *CF* is the moisture correction factor, W_{CA1}, W_{CA2}, W_{FA}, and W_{fiber} are the dry weights of the coarse aggregates, fine aggregate, and fiber, respectively, and ρ_{blend} is the bulk density of the blend. The denominator, which essentially replaces the SSD aggregate volume with the dry aggregate volume, uses the variables V_{SSD} and V_{dry} as the volume of SSD aggregates and dry aggregates, respectively. This equation serves to relate as the bulk dry volume of the aggregates and fiber relative to the mix design with dry aggregates. V_{SSD} is simply the summation of the SSD aggregate volume while V_{dry} is calculated with the following equation:

$$V_{dry} = \frac{\left(\frac{W_{CA1}}{SG_{CA1}}+\frac{W_{CA2}}{SG_{CA2}}+\frac{W_{FA}}{SG_{FA}}\right)}{uw_{water}} \tag{4.2}$$

where SG_{CA1}, SG_{CA2}, and SG_{FA} are the specific gravities of the coarse aggregates and fine aggregate. Note that fiber is not a part of this equation. This is because the denominator of Equation 4.1 is only accounting for ingredients that can hold moisture. If fiber were included, it would just be added back in its entirety. Therefore, it is not necessary to include it.

Using LV-0.5 as an example of these calculations, the summation of the dry aggregate and fiber (66 lbs from Table 4.5) weights is 3113.5 lbs. Dividing this by the bulk density (118.96 lb/ft^3) gives the value 26.17 ft^3 in the numerator of the equation above. As for the denominator, the SSD aggregate volume quantities from Table 4.5 (10.01 ft^3, 1.11 ft^3, and 7.41 ft^3 for #67, #89, and FA, respectively), are subtracted from 27, which returns 8.47 ft^3. The volume quantities of the dry aggregate – calculated using Equation 4.2**Error! Reference source not found.** – returns a volume of 18.43 ft^3. Plugging both values into the denominator results in 26.90 ft^3. Thus, the SSD correction factor for LV-0.5 is calculated as 26.17 ft^3 over 26.90 ft^3, or 0.97.

Continuing with Table 4.6, the corrected void percentage is the average void percentage multiplied by the correction factor. The difference from the reference mix is the key parameter in determining how much fine aggregate to add. It is important to note, however, that the amount of fine aggregate that can be added is limited by the void content of the fine aggregate itself. Thus, the following equation is used specifically to determine the amount that can be added before operating at a loss:

$$FA_{add\%} = (1 - FA_{void\%}) * (\%_{ref}) \tag{4.3}$$

where $FA_{add\%}$ is the necessary additional fine aggregate, $FA_{void\%}$ is the fine aggregate void percentage, which is determined using ASTM C1252 (Uncompacted Void Content of Fine Aggregate), and $\%_{ref}$ is the difference in void percentage from the reference mix (the bottom row of Table 4.6).

Table 4.7 below shows the results of the calculated $FA_{add\%}$, the additional fine aggregate weight in pounds per cubic yard, and the remaining void content not filled by the fine aggregate addition.

Table 4.7 Amount of fine aggregate to be added and remaining void content

Las Vegas FA Void Content: 38.24%	**LV-0.5**	**LV-1.0**	**LV-1.5**	**LV-2.0**
Percentage of FA to be Added	0.16%	1.22%	3.50%	5.33%
Weight (pcy)	7.10	53.69	154.58	235.41
Remaining Void Content	0.10%	0.75%	2.17%	3.30%

The remaining void content is what will be filled with paste in the following step. Also, any additional paste beyond that is considered excess paste. In other words:

$$EP = \%_P - \%_{void} \tag{4.4}$$

where EP is excess paste, %P is the paste volume percentage, and %void is the corrected void percentage.

The diagram in Figure 4.10 is a graphical representation of the effect of additional fines on the void content of the concrete matrix. Note the high amount of white space (representing void content) still remaining.

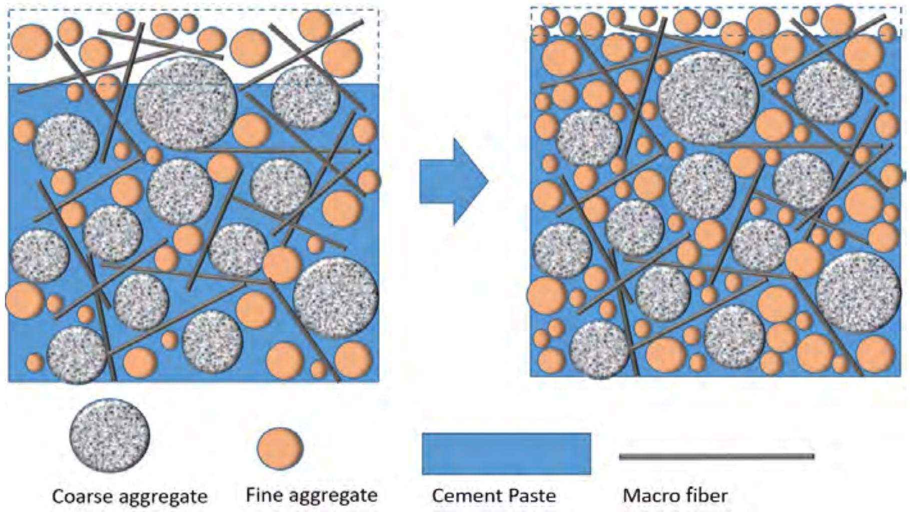


Figure 4.10 Effect of additional fine aggregate on the concrete matrix

The mix designs following the addition of fine aggregate are shown below.

Table 4.8 Mix designs after Step 2 of excess paste adjustment method

Mix ID	Unit	Cement Type V	Class F Fly Ash	Water	#67 C.A.	#89 C.A.	F.A.	T5 Fiber	Excess Paste
LV-Ref	pcy	501	125	250	1682	181	1218	0	**3.01%**
	cf	2.55	0.88	4.01	10.06	1.12	7.45	0.00	
LV-0.5	pcy	498	124	249	1670	180	1216	66	**2.56%**
	cf	2.53	0.87	3.99	9.99	1.11	7.44	0.13	
LV-1.0	pcy	490	122	245	1644	177	1244	131	**0.41%**
	cf	2.49	0.86	3.92	9.83	1.09	7.61	0.27	
LV-1.5	pcy	477	119	238	1600	172	1308	192	**-4.04%**
	cf	2.42	0.83	3.82	9.57	1.06	8.00	0.39	
LV-2.0	pcy	466	116	233	1564	168	1355	251	**-7.62%**
	cf	2.37	0.81	3.73	9.35	1.04	8.29	0.51	

Step 3: Adding paste and adjusting remaining ingredients accordingly

The third and final step involves eliminating the void content through the use of additional paste. Simply filling the voids with paste is not sufficient. However, as an extra amount of paste is necessary to coat the aggregate and fibers and maintain a high degree of workability. The excess paste is, therefore, the driving factor in how much paste is added. This step combines the results of the modified ASTM C29 standard found in Step 2 with the excess paste value of the reference mix (3.01% for LV).

There are three key variables in this step: paste, total aggregate content, and excess paste. Each of these variables is encompassing of other elements (e.g., the paste is made up of cement, fly ash, water, air, and admixtures), and thus adjusting them as a whole means adjusting the components as a whole (air content and admixtures are not adjusted, however). In other words, when adjusting paste, for example, the ratios within the paste stay the same. This conserves the water-to-binder ratio used in the reference mix. This is also true for the aggregates. This both simplifies the process and maintains proportions similar to those of the references mix.

Details of this step's calculations are described under the following table:

Table 4.9 Adjusting paste and total aggregate to match excess paste

		LV-Ref	LV-0.5	LV-1.0	LV-1.5	LV-2.0
Aggregate Content		68.98%	68.67%	68.62%	68.91%	69.10%
Paste Content		31.02%	30.83%	30.40%	29.64%	29.03%
Target EP		3.01%	3.01%	3.01%	3.01%	3.01%
Admixtures and Air Content		N/A	3.47%	3.47%	3.47%	3.47%
Adjustable Paste [(Paste Content) – (Admixtures & Air)]		N/A	27.82%	29.53%	33.23%	36.19%
Paste Content (NEW)		N/A	31.29%	33.00%	36.70%	39.66%
Aggregate Content (NEW)		N/A	68.21%	66.02%	61.86%	58.44%
Paste Ratios	Cement	N/A	0.343	0.343	0.343	0.343
	Fly Ash	N/A	0.118	0.118	0.118	0.118
	Water	N/A	0.540	0.540	0.540	0.540
Aggregate Ratios	#67	N/A	0.539	0.531	0.514	0.501
	#89	N/A	0.060	0.059	0.057	0.055
	F.A.	N/A	0.401	0.410	0.429	0.444
Adjusted Paste Volumes	Cement	N/A	2.57	2.73	3.07	3.35
	Fly Ash	N/A	0.88	0.94	1.06	1.15
	Water	N/A	4.05	4.30	4.84	5.27
Adjusted Aggregate Volumes	#67	N/A	9.92	9.46	8.58	7.90
	#89	N/A	1.10	1.05	0.95	0.88
	F.A.	N/A	7.39	7.32	7.17	7.00
	Fiber (unchanged)	N/A	0.13	0.27	0.39	0.51

The aggregate content percentage and paste content percentage are taken from Step 2 and are simply the ratio of the aggregate and paste to the overall mix. With the target excess paste value set by the reference mix, the paste is increased first, and the aggregate is adjusted accordingly. This is how the new paste percentage is determined. As was stated before, it is important to maintain admixture content and air content throughout this adjustment process. Therefore, only the cement, fly ash, and water were adjusted, hence the name "adjustable paste" in the table. This step is done for several iterations (this example utilized Microsoft Excel) until the excess paste is equal for each mix.

There are some additional details of note in this step. For example, the "paste ratios" row has the same values for each degree of fiber. The aggregate ratios change slightly with each interval of aggregate due to the slight increase in fine aggregate from Step 2. The ratios are the same, though, when compared to the mix designs from Step 2. The adjusted paste volumes and adjusted aggregate volumes are calculated by taking each respective ratio and multiplying by the paste percentage and 27 ft^3. The diagram in Figure 4.11 offers a graphical representation of the final step of the process. The space between particles is largely filled in, leaving only the desired air content.

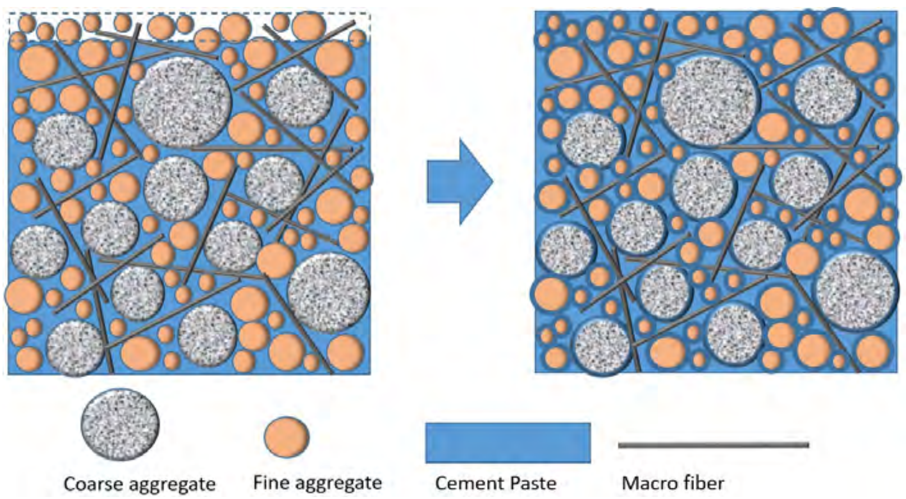


Figure 4.11 Effect of excess paste on the concrete matrix

The final mix designs are seen below in Table 4.10.

Table 4.10 Mix designs after Step 3 of excess paste adjustment method

Mix ID	Unit	Cement Type V	Class F Fly Ash	Water	#67 C.A.	#89 C.A.	F.A.	T5 Fiber	Excess Paste
LV-Ref	pcy	501	125	250	1682	181	1218	0	**3.01%**
	cf	2.55	0.88	4.01	10.06	1.12	7.45	0	
LV-0.5	pcy	506	126	253	1659	178	1208	66	**3.01%**
	cf	2.57	0.88	4.05	9.92	1.1	7.39	0.13	
LV-1.0	pcy	537	134	268	1581	170	1196	131	**3.01%**
	cf	2.73	0.94	4.3	9.46	1.05	7.32	0.27	
LV-1.5	pcy	604	151	302	1434	154	1172	192	**3.01%**
	cf	3.07	1.06	4.84	8.58	0.95	7.17	0.39	
LV-2.0	pcy	658	165	329	1321	142	1145	251	**3.01%**
	cf	3.35	1.15	5.27	7.9	0.88	7	0.51	

Figure 4.12 presents a graphical summary of the overall adjustment process and the effect on each element of the concrete.

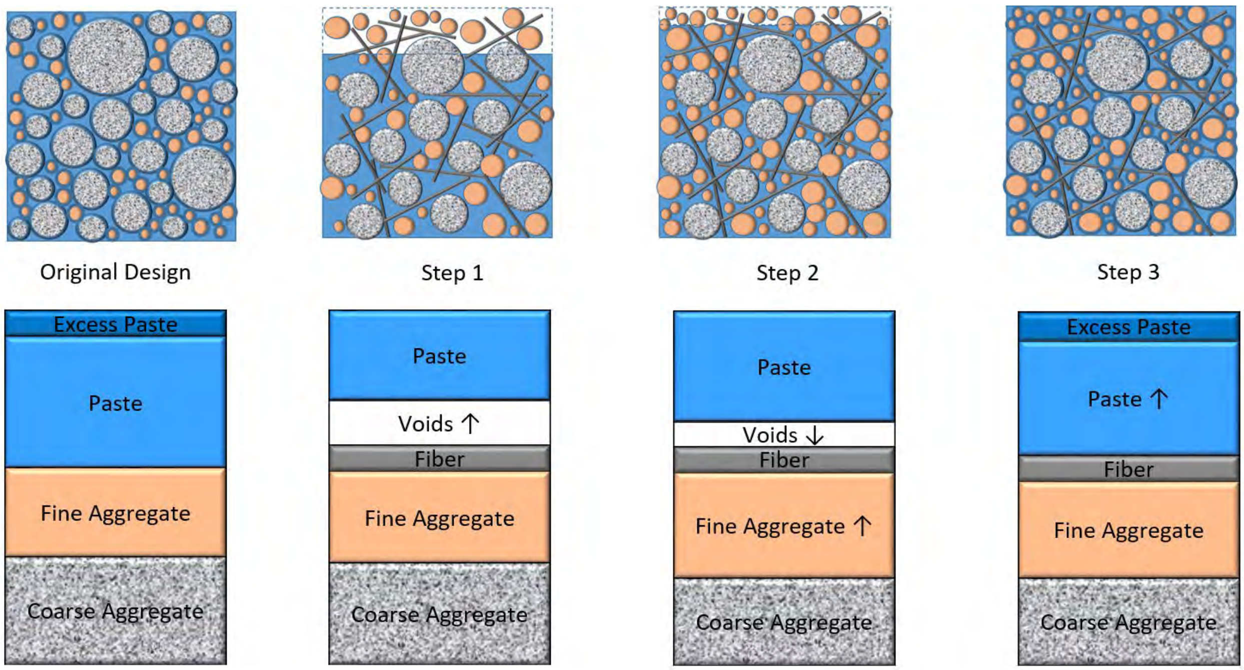


Figure 4.12 Graphical summary of excess paste adjustment method

This adjustment method can be expanded upon to relate excess paste to the aggregate. Some work was done with an excess paste-to-aggregate ratio, which would have provided slightly more relevance to a particular set of materials, but results varied considerably when attempted with Nevada and Lincoln mixes. This is a potential topic of future study, as it could build upon the excess paste method to deliver a more adaptable mix design method. However, the excess paste adjustment method yielded positive results for Las Vegas, Reno, and Lincoln materials, all of three of which have unique properties to one another.

4.3.2 FRC Mix Designs

As was stated at the beginning of Section 4.3.1, the original adjustment method (based on maintaining the paste-to-fine aggregate ratio) was unsuccessful in that it had insufficient paste to preserve the workability of the reference mixes but provided insight to the best option for fiber volume. Four degrees of fiber (0.5%, 1.0%, 1.5%, and 2.0%) allowed for comparisons to the previous phase, and the highest compatibility between workability, flexural performance, and cost was found with 1.5% of fiber. It was then the best option to move forward with 1.5% of fiber for the remainder of Phase II. The other mix designs are seen in the appendix.

The mix-ID for the mix designs throughout the remainder of the report will be the city followed by “Ref” or “FRC” for the reference and high-performance fiber reinforced concrete mixes, respectively.

The Las Vegas mix design can be seen in Table 4.11Figure 4.14. Binder content is increased by approximately 130 pcy, while total aggregate content drops approximately 320 pcy. The listed HRWR is at the top of the manufacture recommended range, and the regular water reducer is at the midpoint of the range.

Table 4.11 Las Vegas FRC mix design using the excess paste adjustment method

Mix ID	Unit	Type V Cement	Class F Fly Ash	Water	#67 C.A.	#89 C.A.	F.A.	T5 Fiber
LV-Ref	pcy	501	125	250	1682	181	1218	0
	cf	2.55	0.88	4.01	10.06	1.12	7.45	0
LV-FRC	pcy	604	151	302	1434	154	1172	192
	cf	3.07	1.06	4.84	8.58	0.95	7.17	0.39
w/b: 0.41 Air Content: 1.5-4.5% Excess Paste: 3.01% Admixtures → HRWR: 11 fl oz/cwt WR: 8 fl oz/cwt								

The Las Vegas mix design can be seen in Table 4.12. Binder content is increased by approximately 200 pcy, and total aggregate is decreased by approximately 430 lbs. The HRWR, VMA, and AEA are near the middle of the manufacture recommended ranges, while the stabilizer is near the bottom of the recommended range.

Table 4.12 Reno FRC mix design using the excess paste adjustment method

Mix ID	Unit	Type I/II Cement	Class F Fly Ash	Water	#67 C.A.	#89 C.A.	F.A.	T5 Fiber
Reno-Ref	pcy	521	173	258	1381	247	1187	0
	cf	2.65	1.08	4.13	8.52	1.52	7.31	0.00
Reno-FRC	pcy	673	224	332	1074	192	1117	188
	cf	3.42	1.40	5.33	6.62	1.18	6.88	0.38
w/b: 0.38 Air Content: 4.0-7.0% Excess Paste: 8.96% Admixtures → HRWR: 10 fl oz/cwt VMA: 6 fl oz/cwt Stabilizer: 5 fl oz/cwt AEA: 1 fl oz/cwt								

The Lincoln mix design can be seen in Table 4.13. The binder content increases by approximately 60 pcy while the aggregate decreases by approximately 180 pcy. Both the WR and AEA are dosed at the high end of the manufacture recommended range. Fibers do much less to disrupt the matrix of Nebraska's state mix due to the 70-30 fine aggregate-to-coarse aggregate ratio. This is why there is a relatively small increase in fine aggregate content and a much smaller increase in the binder. Fine aggregate largely fills the voids that are caused by fiber before much additional aggregate needs to be added.

Table 4.13 Lincoln FRC mix design using the excess paste adjustment method

Mix ID	Unit	Type IP Cement	Water	#57 C.A.	Sand & Gravel	T5 Fiber
Lincoln-Ref	pcy	657	255	867	1993	0
	cf	3.52	4.08	5.22	12.19	0.0
Lincoln-FRC	pcy	721	280	785	1887	194
	cf	3.87	4.48	4.73	11.54	0.40
w/b: 0.39 Air Content: 6.0-8.5% Excess Paste: 9.38% Admixtures → WR: 6 fl oz/cwt AEA: 1 fl oz/cwt						

4.3.3 Results

Table 4.14 shows the results of the excess paste adjustment for Las Vegas, Reno, and Lincoln materials. Multiple mixes were done for Las Vegas and Reno FRC using the same mix design, but with small changes in HRWR. Furthermore, multiple mixes were necessary to perform all the hardened concrete tests.

Table 4.14 Phase II fresh concrete test results

Mix ID	Mix #	Slump before Fiber (in.)	Slump after Fiber (in.)	Fresh Unit Weight (pcf)	Air Content	V-L-Box Test t_4 (sec)	V-L-Box Test t_6 (sec)	VSI	Finishability
LV-Ref	M1	4.5	N/A	146.2	3.4%	6.25	15.0	1	Good
Reno-Ref	M1	5.5	N/A	144.0	3.3%	3.75	5.5	0	Good
	M2	7.0	N/A	142.0	4.6%	N/A	N/A	0	Good
Reno-FRC	M1	6.0	2.25	146.5	3.6%	4.5	5.75	0	Good
	M2	8.0	4.0	142.4	5.0%	N/A	N/A	0	Good
Lincoln-Ref	M1	5.0	N/A	140.9	6.5%	N/A	N/A	0	Good
Lincoln-FRC	M1	6.0	4.0	N/A	N/A	15.75	22.5	0	Good

In general, the fresh concrete results illustrate the conservation of workability and concrete appearance. A loss of flowability was expected for both Nevada mixes, but the Reno mix actually had virtually no loss under vibration. Due to the higher paste and fine aggregate content, the Visual Stability Index was 0 (good cohesion and stability with limited segregation) for all mixes with the exception of the Las Vegas Reference mix. Other Las Vegas reference mixes, utilizing more vibration, looked much more stable. Considering stability following vibration is an important concept with fiber reinforced concrete, the often-prolonged period of vibration during the L-Box test can display how stable a particular concrete is. Figure 4.13 shows the reference and FRC Las Vegas mixes for a comparison of their appearance. The FRC mix clearly has more paste present. This is, of course, the result of the longer period of vibration but also an increased amount of paste overall from the excess paste design adjustment. The Lincoln FRC mix was of high viscosity, potentially due to the higher fine aggregate content versus the other mixes. This could be the cause of the longer L-Box t_4 and t_6 times.

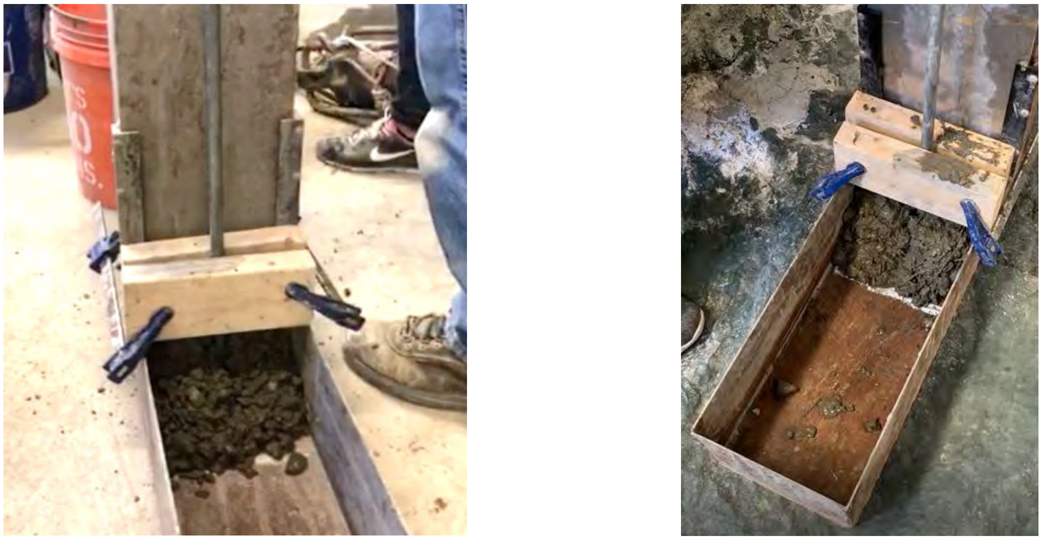


Figure 4.13 Comparison of LV-Ref and LV-FRC stability during vibrated L-Box test

Lincoln mixes had very comparable appearances, with little difference beyond the few fibers protruding from the concrete. For example, see Figure 4.14 below.

Figure 4.14 Comparison of Lincoln-Ref and FRC mix stability and appearance

The excess paste adjustment method improved the results of nearly all the hardened concrete tests in Phase II. Furthermore, with the influence placed on excess paste content, the hardened appearance of the FRC was expected to be comparable to that of the reference mixes. The 4x8 specimens seen in Figure 4.15 were vibrated externally for 8-10 seconds, and, overall, these specimens share a very similar or improved appearance to the reference mixes.

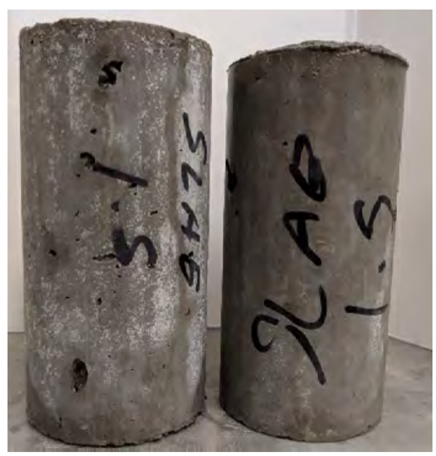
LV-Ref

LV-FRC

Reno-Ref

Reno-FRC

Lincoln-Ref



Lincoln-FRC

Figure 4.15 Comparison of hardened concrete specimen appearance

The paste-to-fine aggregate adjustment method would often result in concrete with significant voids, indicating that an appropriate amount of paste is very important to maintaining a concrete that has a smooth and filled out the surface. See Figure 4.16 below for an example of a Las Vegas mix using this adjustment method and note the particularly large voids covering the surface.

Figure 4.16 Example casted cylinder specimen with inadequate paste

Compressive strength, though not typically improved by fibers (many Phase I mixes lost strength), was improved for all three sets of mixes for Phase II. Results showed that fiber reinforced concrete was noticeably stronger but specimens over their reference mix counterparts. However, this is very likely due to the increased paste content, rather than a function of the fibers. The results are presented in Figure 4.17.

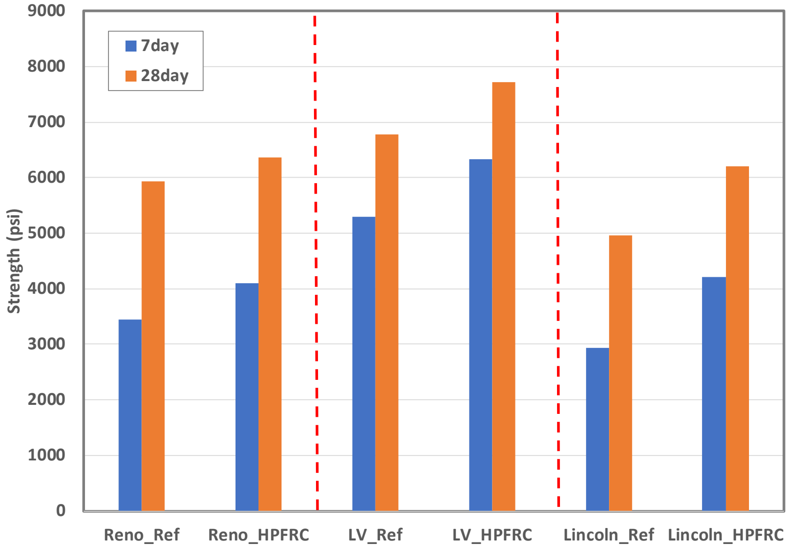


Figure 4.17 Phase II compressive strength results

Failure under compressive loading was typically much less dramatic for FRC mixes versus normal concrete. Exterior cracks were often very small or not visible at all, and there were rarely large fragments of concrete. Flaking after the failure occurred but was not frequent. Figure 4.18 shows two samples of FRC with typical crack patterns.

Figure 4.18 Typical compression cracks of FRC cylinders after failure

The flexural performance was aided significantly by fibers. The Reno FRC had a modest improvement over the reference mix in modulus of rupture (MOR), and Las Vegas FRC improved considerably. With toughness not being a typical parameter of normal concrete but being a very important one for fiber, the toughness value was obviously very much improved. Note: the toughness values should not be compared to Phase I values considering Phase I tests did not use LVDT's and are therefore less accurate.

The flexural strength test results from Las Vegas mixes are presented in Figure 4.19, comparing the FRC mix to the reference mix. Both the modulus of rupture and toughness were significantly improved. After the initial crack, the abrupt jumps in the graph indicate that the beam cracked and, where a normal beam would fail, was rather "caught" by the fibers. Load decreased in essentially a linear trend to the end of the test at 0.12" of deflection. The FRC load at the end of the test was barely below the load sustained by the normal concrete before breaking.

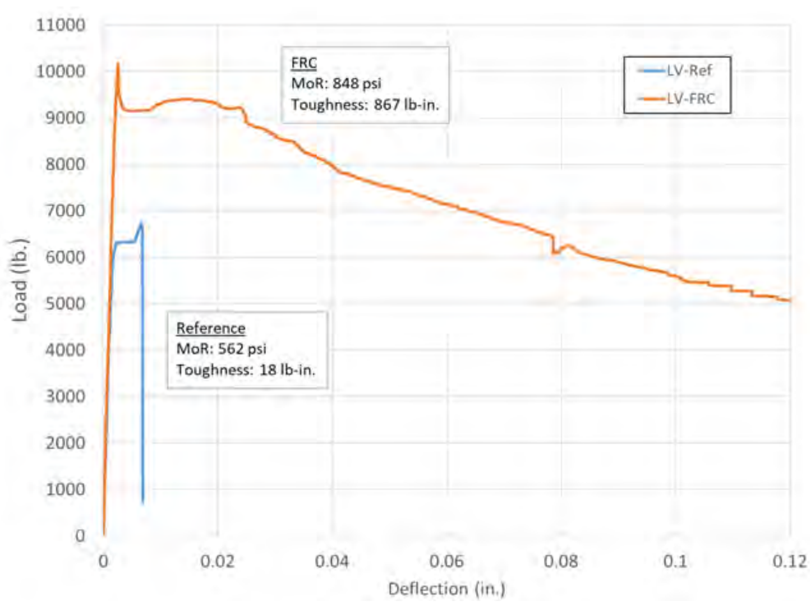


Figure 4.19 Phase II Las Vegas flexural graph

The Reno flexural graph is presented in Figure 4.20. The modulus of rupture was improved while toughness was again significantly improved. Upon initial cracking, the beam decreased in load tolerance until it plateaued at approximately 7500 psi. It maintained this value to the end of the test, which is a sign of highly stable fiber reinforced concrete.

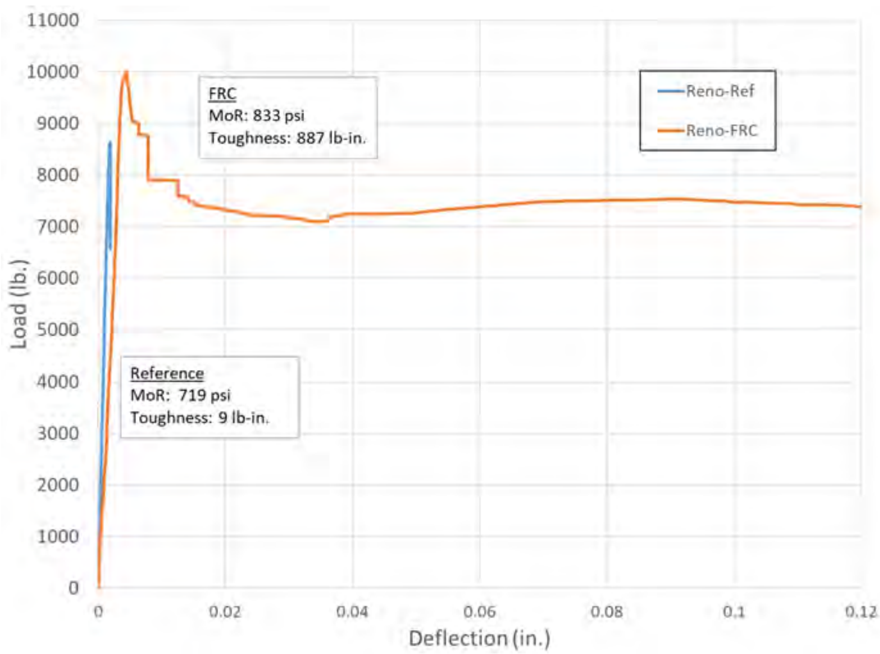


Figure 4.20 Phase II Reno flexural graph

A typical crack at the end of a test can be seen in Figure 4.21. Cracks would frequently extend to near the top surface. Multiple smaller cracks propagating from the main crack were a common occurrence but were held tight by fibers and not to be considered major cracks during this test. Crack widths were not measured as it not practical to get a measurement with the beam in place and were inaccurate once the beam was moved.

Figure 4.21 Typical FRC crack at conclusion of test

Splitting tensile strength was another parameter that was assumed to be improved with fiber reinforced concrete. However, the splitting tensile test is not necessarily a quality test for the tensile strength of FRC. Once the cylinder cracks, the cross-section can be deformed to an oval shape as the fibers arrest the cracks. With that being said, there was still a substantial increase in strength, and the results at 7 days and 28 days are displayed in Figure 4.22.

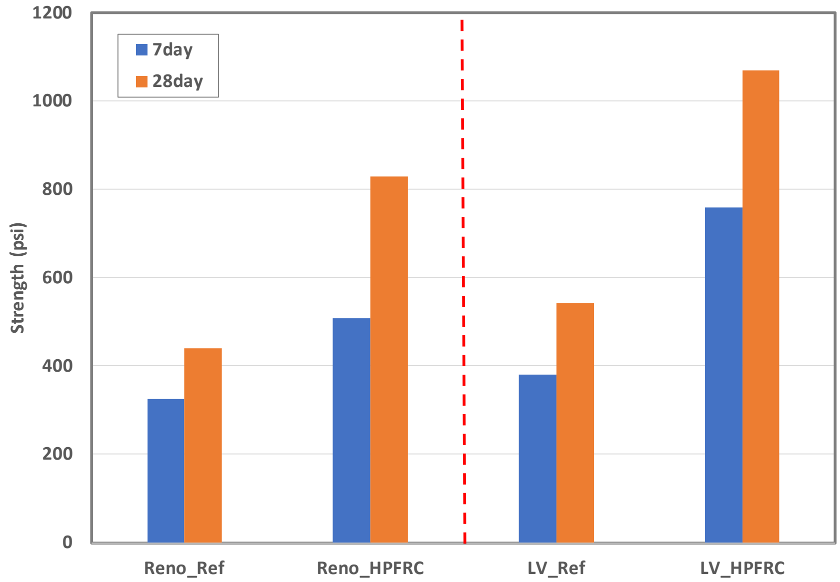


Figure 4.22 Phase II splitting tensile strength results

4.4 Phase III Study (Performance Evaluation Phase)

4.4.1 Mix Designs

The same mix designs from Phase II apply for the performance evaluation phase. Mixes were performed with 10% of the water on reserve and all HRWR on reserve. In almost every case for Las Vegas mixes, all the reserved water and HRWR were added back in. FRC Mixes for Reno did not typically need much HRWR to be workable, and thus very small dosages were used.

4.4.2 Results

The free shrinkage results were conflicting for mixes with Las Vegas and Reno materials. A higher degree of shrinkage was expected, given the increased cement content. However, with Las Vegas materials, the additional shrinkage of the FRC versus the reference mix was almost negligible. Reno, on the other hand, showed a considerable shrinkage increase. The two graphs in Figure 4.23 show the shrinkage results.

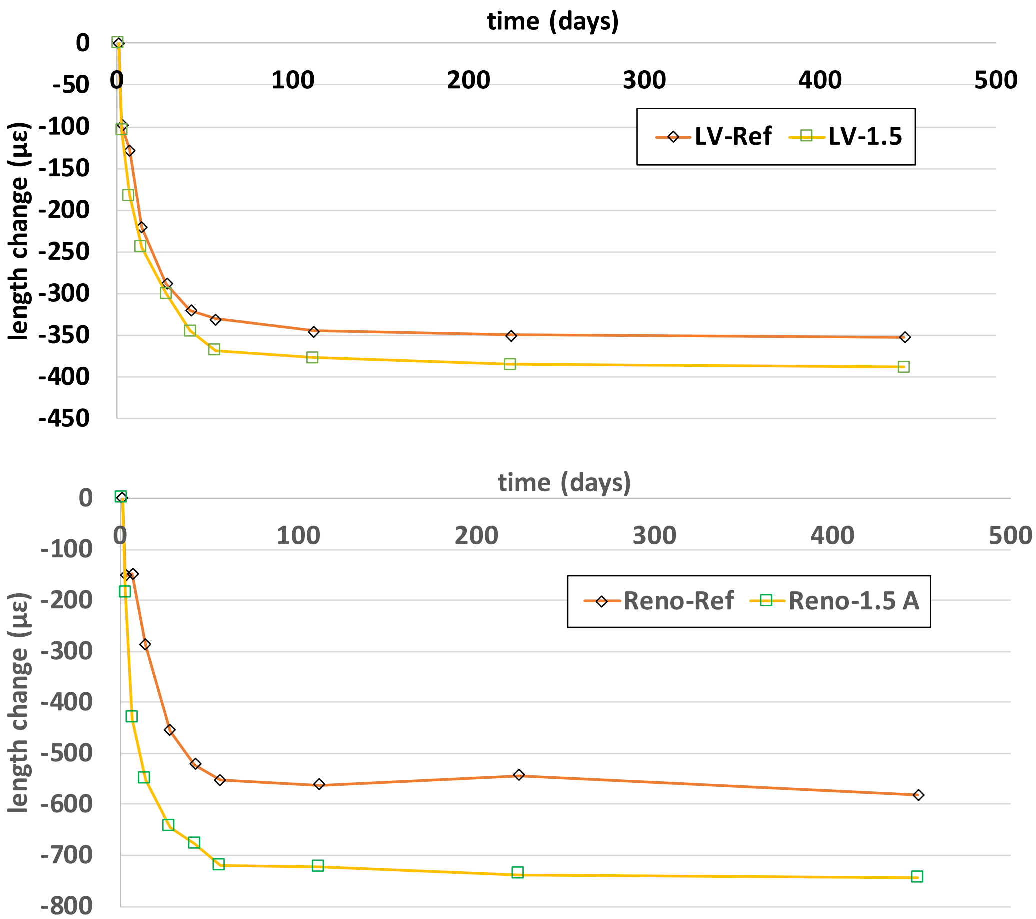


Figure 4.23 Free shrinkage results

Restrained shrinkage results were only obtained for Reno as the Las Vegas rings had several faulty readings or no reading at all. The results of the Reno-Ref versus Reno-FRC mixes can be seen below. The reference mix cracked at 8.5 days. Even with a higher rate of strain (likely a product of the higher cement content), the FRC mix did not show any notable cracking. It's highly possible that micro-cracks formed, hence the small local fluctuations in the strain. With normal concrete, these micro-cracks would immediately turn into major cracks. With fibers, however, the micro-cracks are bridged and not allowed to propagate further. In Chapter 5, which deals with SHPC, the restrained shrinkage ring does have a notable crack, and the fibers act in a similar manner to a flexural test in which they repeatedly bridge the crack and prevent crack widening. See section 5.5.2 for the graph.

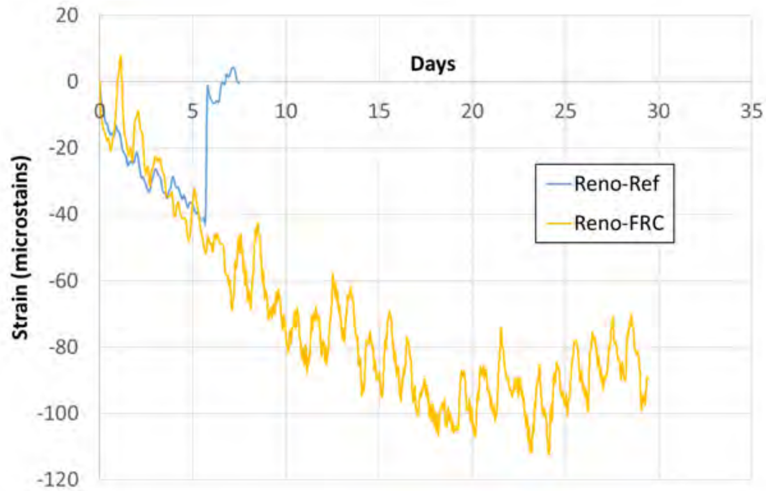
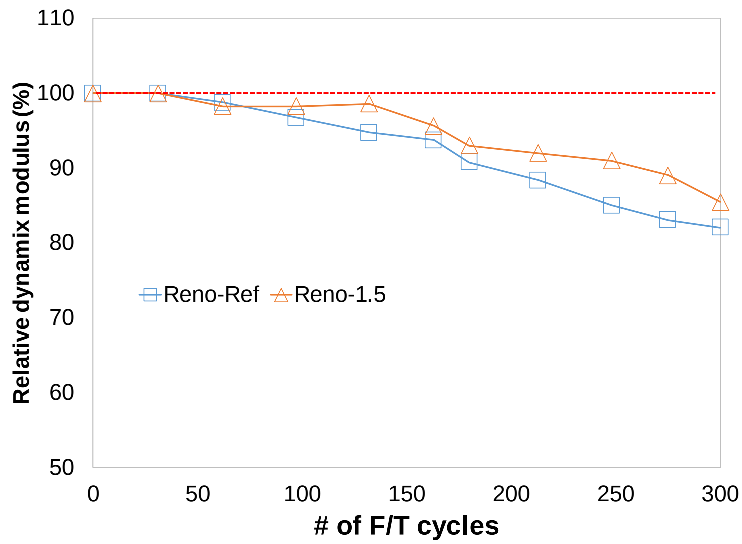
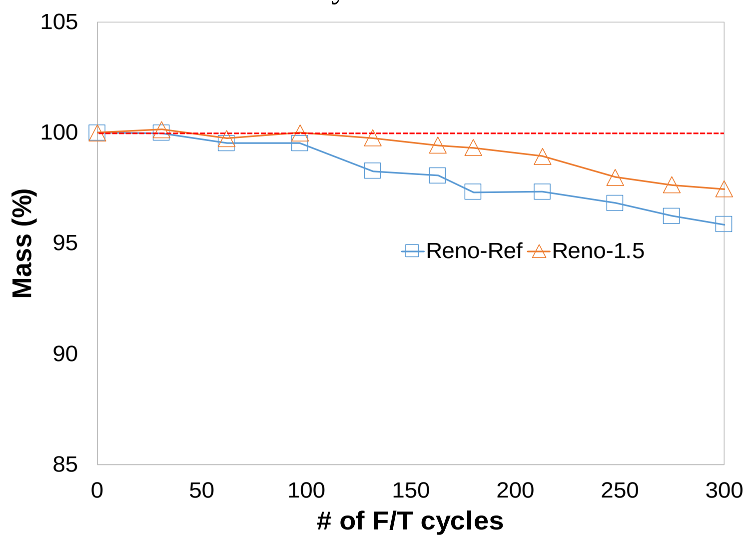


Figure 4.24 Restrained shrinkage results

Freeze/Thaw tests were conducted for Reno-FRC and the reference mix. The results of the F/T testing can be seen in Figure 4.25. Through 300 cycles, less than 20% loss was seen in the relative dynamic modulus (RDM), with the FRC mix has slightly less RDM loss. The mass losses were at approximately 4.1% and 2.5% for the FRC mix and reference mix, respectively.



Relative dynamic modulus



Mass

Figure 4.25 F/T testing results

CHAPTER 5. DEVELOPMENT OF SUPER HIGH-PERFORMANCE CONCRETE

5.1 Introduction

Super High-Performance Concrete (SHPC) is a special type of concrete developed using materials from Reno and Las Vegas, Nevada, and local materials from Omaha, Nebraska. The goal of this new concrete is to provide an alternative to ultra-high performance concrete (UHPC). In applications in which high-performance concrete (HPC) is not sufficient, SHPC could potentially be employed rather than UHPC with benefits to both cost and production. Besides the prohibitive cost, UHPC is known for requiring high shear mixing and cannot be produced in a traditional drum-type mixer such as ready-mixed trucks. SHPC, on the other hand, significantly cuts back on cost and is compatible with a drum mixer while also providing far better workability, strength, and crack resistance than HPC.

SHPC is self-consolidating, has a low water-to-binder ratio, and incorporates silica fume and fibers. Several initial designs were tested using typical silica fume based concrete design methods. These designs were based on steel fiber reinforced self-consolidating concrete designs presented in a study by Ferrara et al. in 2007, but with a portion of the cement replaced with silica fume. Reno materials were used throughout this initial stage. This phase, called the preliminary phase, was performed until promising results were found for a particular mix. Once a mix design was determined, the second phase (called the SHPC Adjustment Phase) involved an adjustment very similar to the method described in Section 4.3.1. This adjustment was carried out to convert the Reno design to Las Vegas materials and Omaha materials. This chapter will detail the design, adjustment, test methods, and results of SHPC. Chapter 6 includes a pair of lab-scale beam connections constructed with Reno and Omaha SHPC.

In summary, SHPC is defined as a self-consolidating concrete with a 28-day compressive strength of at least 10,000 psi, a 28-day modulus of rupture of at least 1,000 psi, and a 28-day toughness (calculated according to ASTM C1609). It is typically reinforced at 1.5-2% of steel fibers by volume. Silica fume is also typically present, and a high fine-to-coarse aggregate ratio is used to improve particle packing.

5.2 Mixing Procedure

The mixing procedure for SHPC differs from that of standard FRC partly due to the addition of silica fume. The overall process for SHPC is similar to that of silica fume concrete but with the incorporation of fiber. The procedure begins with coarse aggregate being mixed with approximately 75% of the water (with the air-entraining agent) for 30 seconds. Silica fume is then added while the mixer is running (Figure 5.1).

Figure 5.1 Loading silica fume

The silica fume mixing period is 1.5 minutes, and the silica fume should be added during the first 30 seconds. Next, cement and fly ash are added while the mixer is at rest. The mixer is then turned on for another 1.5-minute mixing period. Fine aggregate is added to the stationary mixer, and the remaining 25% of the water (with a high-range water reducer, mid-range water reducer, and viscosity modifier) is added to a rotating mixer at the beginning of a 5-minute mixing period. This is followed by a 3-minute resting period. Another 5 minutes of mixing is then carried out before determining the slump-flow before the fiber is introduced. Once the slump flow test (ASTM C1611) is finished, and the material has been returned to the mixer, another 5-minute mixing period is started. In the same manner, as with fiber reinforced concrete, fiber is introduced slowly at the beginning of this period, but with the entirety of the fiber ideally loaded in the first minute. Following the conclusion of the 5-minute period, another slump-flow test is performed. All samples that were cast were cured in the same manner as their normal concrete and FRC counterparts, as described in Section 3.4.3.

5.3 Test Methods

Many of the same tests used for FRC outlined in Section 3.4 are used for SHPC. However, given that SHPC is designed to be self-consolidating, a slump flow test replaces the slump test as the standard measurement of workability. The slump flow test (ASTM C1611) involves filling an inverted slump cone, lifting the cone, and allowing concrete to flow out, recording the time for the concrete to spread to 500mm, and lastly, measuring two diameters of the concrete once it has stopped flowing. The first diameter should be the visibly longest diameter, and the second should be perpendicular to the first diameter. A commonly accepted range of flow spread for SCC is 18”-30”. See Figure 5.2 for the testing procedure.

Figure 5.2 Slump flow test

All other tests for SHPC are described in Section 3.4. These tests included visual stability index, compressive strength, flexural strength, slant shear bonding strength, and restrained shrinkage. In addition, two beam connections were cast using SHPC mix designs.

5.4 Phase I Study (Preliminary Phase)

An important concept of SHPC was to maintain a low water-to-binder ratio (0.25) and a high degree of workability with minimal segregation without solely relying on a viscosity modifying admixture. With UHPC, particle packing is very important to the qualities that make UHPC the most impressive variation of concrete. SHPC uses high cementitious material content to achieve a similar result. The excess paste is significantly higher in the case of SHPC than the FRC mixes, and the early issues when designing SHPC largely dealt with determining the appropriate amount that prevented segregation. Silica fume content is a key component here as well given that it tends to make cement paste far “stickier”, i.e., more viscous, which limits segregation. Figure 5.3 shows a comparison of SHPC to other concrete matrices.

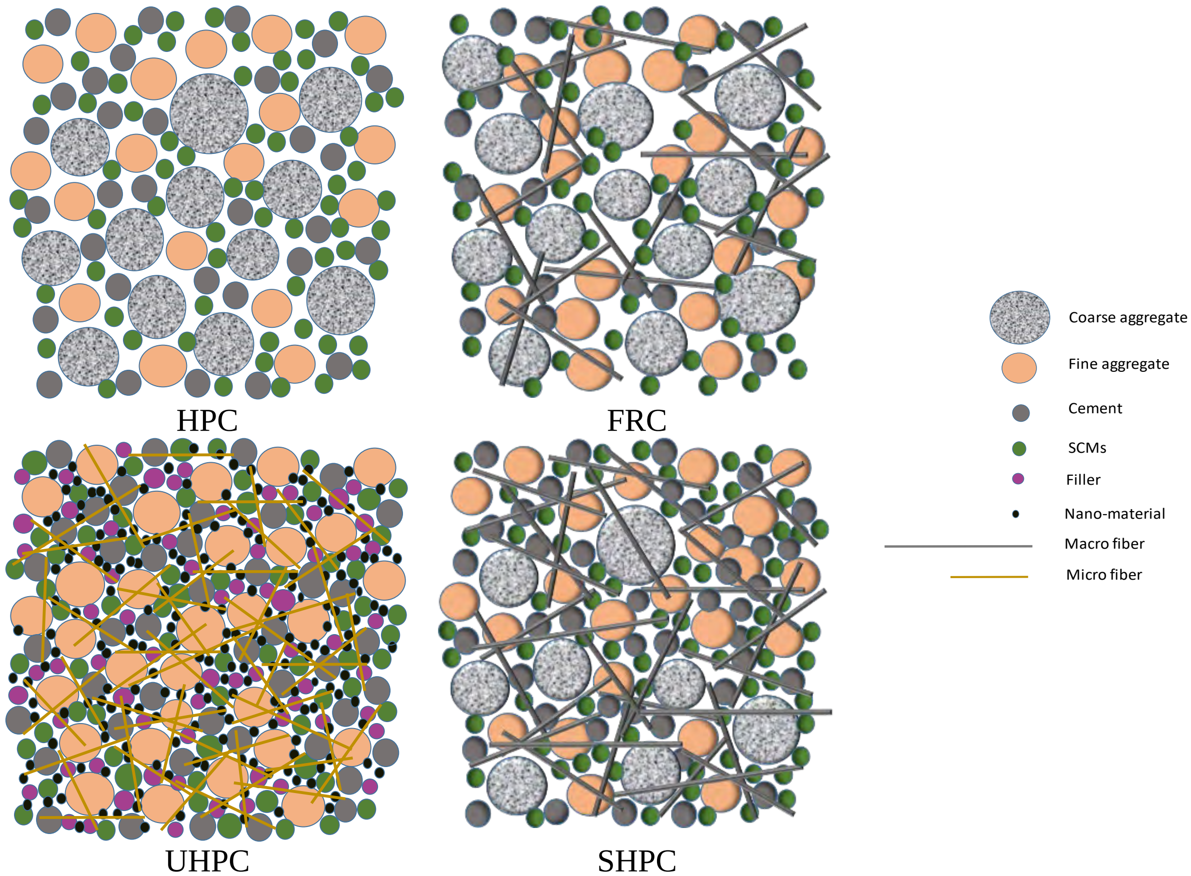


Figure 5.3 Comparison of SHPC matrix to other concretes

5.4.1 Mix Designs

Selected preliminary mixes using Reno materials that were conducted for SHPC can be seen in Table 5.1. SHPC uses 2% of fiber by volume, a high-range water reducer, viscosity modifying admixture, stabilizer, and air entraining agent (the same admixtures used in Reno-Ref and Reno-FRC).

Table 5.1 Mix design for SHPC (Reno materials)

Mix ID	Type I / II	Silica Fume	F Fly ash	Water	#67	#89	Sand	T5 Fiber	Gl. 7500 (fl oz/cwt)	Paste vol%	Excess Paste vol%
SHPC1	796	68	264	276	1205	215	858	258	12.1	46.13%	15.30%
SHPC2	871	75	289	302	992	177	940	258	12.1	49.96%	19.13%
SHPC3	909	78	301	315	885	158	980	258	12.1	51.88%	21.05%
SHPC4	946	81	313	328	779	139	1021	258	13.2	53.82%	22.99%
For all mixes	w/b: 0.25 Air Content: 6.0% VMA 362: 3.6 fl oz/cwt Delvo Stabilizer: 3.6 fl oz/cwt Master Air 200: 1.2 fl oz/cwt										

5.4.2 Results

The fresh concrete results for each of the first four mixes are presented in Table 5.2 below.

Table 5.2 Fresh concrete properties of SHPC mixes (Reno Materials).

Mix ID	Slump flow before fiber, in.	T_{500} before fiber	Slump flow after fiber, in.	T_{500} after fiber	VSI	Fresh unit wt, pcf
SHPC1	26.75	N/A	17.625	N/A	2	142.7
SHPC2	28.25	7 seconds	24.375	22 seconds	1	147.6
SHPC3	31.00	N/A	27.625	N/A	1	146.6
SHPC4	30.50	12 seconds	29.75	17 seconds	0	147.8

In terms of workability, SHPC4 was the most impressive as the slump flow loss was minimal, and the T_{500} time was just 5 seconds longer after the addition of fiber. Though none of these mixes were particularly poor, the first three mixes were clearly more impacted by fiber. The consistency of SHPC4 before and after the addition of fiber can be seen in Figure 5.4.

Figure 5.4 SHPC4 consistency before and after adding fibers

The compressive strength of the mixes can be seen in Figure 5.5.

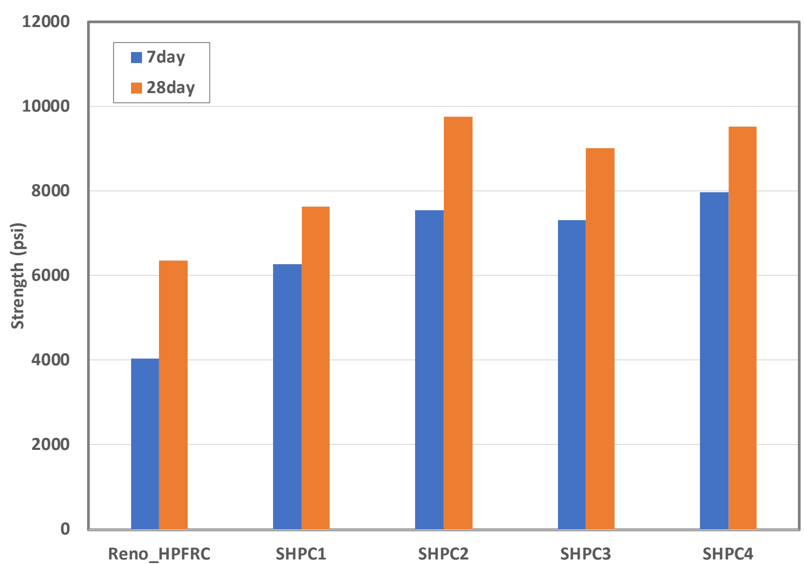


Figure 5.5 Phase I SHPC compressive strength results

The Reno FRC mix is included in the chart for comparison considering the SHPC mixes used the Reno materials. SHPC2 had the highest compressive strength at 28 days, but SHPC4 had the highest 7-day strength. They are comparable enough, however, for the difference to be mostly negligible.

The graphs from the flexural strength tests can be seen in Figure 5.6.

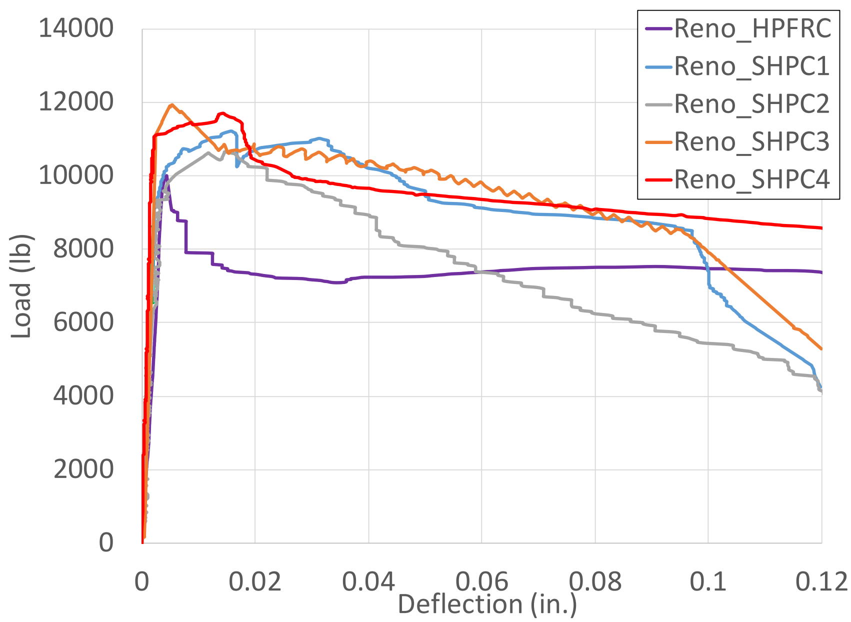


Figure 5.6 Flexural strength graph for initial SHPC mixes

SHPC3 and SHPC4 had the best combination of peak load and toughness. The modulus of rupture and toughness can be seen in Table 5.3.

Table 5.3 Flexural results for initial SHPC mixes

Mix ID	Modulus of Rupture (psi)	Toughness (lb-in.)
Reno_FRC	833	887
Reno_SHPC1	934	1069
Reno_SHPC2	888	898
Reno_SHPC3	995	1111
Reno_SHPC4	976	1060

When comparing the mixes, SHPC4 was ultimately chosen as the most promising, given its high performing workability properties and mechanical properties. This version of SHPC was then referred to as Reno-SHPC for the remainder of this study.

5.5 Phase II Study (SHPC Adjustment Phase)

SHPC is adjusted in a very similar manner to FRC. The excess paste is again the control parameter. With SHPC, however, the adjustment is not from a mix with to fiber to a mix with fibers. Rather, it is an adjustment to an entirely new set of materials (with fibers and admixtures remaining the same). The adjustment was made for both Las Vegas and Omaha materials. The Omaha materials, which included a comparable aggregate gradation to Reno and Las Vegas, involved a Type IP cement. In this adjustment process, the initial step is to match the water-to-binder ratio. Then, the paste is increased, and aggregate is adjusted to the point that excess paste content is the same as the Reno mix. Thus, the second step is different from the FRC excess paste method as it does not involve adding fine aggregate but rather is focused on matching water-to-binder ratio.

5.5.1 Mix Designs

The mix designs after adjustment for Reno, Omaha, and Las Vegas materials are presented in Table 5.4.

Table 5.4 SHPC final mix designs

<table>
<tr><th>Mix ID</th><th>Unit</th><th>Cement Type I/II</th><th>Silica Fume</th><th>Class F Fly Ash</th><th>Water</th><th>#67 C.A.</th><th>#89 C.A.</th><th>F.A.</th><th>T5 Fiber</th></tr>
<tr><td rowspan="2">Reno-SHPC</td><td>pcy</td><td>946</td><td>81</td><td>313</td><td>328</td><td>779</td><td>139</td><td>1021</td><td>258</td></tr>
<tr><td>cf</td><td>4.81</td><td>0.59</td><td>1.95</td><td>5.25</td><td>4.80</td><td>0.86</td><td>6.28</td><td>0.53</td></tr>
<tr><td colspan="10">w/b: 0.26 Excess Paste: 22.99% Air Content: 6.0% VMA: 3.6 fl oz/cwt
HRWR: 13.2 fl oz/cwt Stabilizer: 3.6 fl oz/cwt AEA: 1.2 fl oz/cwt</td></tr>
<tr><th>Mix ID</th><th>Unit</th><th>Cement Type V</th><th>Silica Fume</th><th>Class F Fly Ash</th><th>Water</th><th>#67 C.A.</th><th>#89 C.A.</th><th>F.A.</th><th>T5 Fiber</th></tr>
<tr><td rowspan="2">LV-SHPC</td><td>pcy</td><td>864</td><td>75</td><td>255</td><td>293</td><td>883</td><td>153</td><td>1130</td><td>258</td></tr>
<tr><td>cf</td><td>4.40</td><td>0.54</td><td>1.78</td><td>4.69</td><td>5.28</td><td>0.95</td><td>6.91</td><td>0.53</td></tr>
<tr><td colspan="10">w/b: 0.26 Excess Paste: 22.98% Air Content: 6.0% VMA: 4.3 fl oz/cwt
HRWR: 12.0 fl oz/cwt Stabilizer: 4.3 fl oz/cwt</td></tr>
<tr><th>Mix ID</th><th>Unit</th><th>Cement Type IP</th><th>Silica Fume</th><th>Water</th><th>#67 C.A.</th><th>#89 C.A.</th><th>Sand & Gravel</th><th colspan="2">T5 Fiber</th></tr>
<tr><td rowspan="2">Omaha-SHPC</td><td>pcy</td><td>920</td><td>79</td><td>244</td><td>981</td><td>176</td><td>1283</td><td colspan="2">258</td></tr>
<tr><td>cf</td><td>4.93</td><td>0.57</td><td>3.91</td><td>5.93</td><td>1.06</td><td>7.77</td><td colspan="2">0.53</td></tr>
<tr><td colspan="10">w/b: 0.26 Excess Paste: 23.00% Air Content: 7.25% VMA: 4.3 fl oz/cwt
HRWR: 15.0 fl oz/cwt Stabilizer: 4.3 fl oz/cwt AEA: 2.2 fl oz/cwt</td></tr>
</table>

The different materials resulted in quite different designs following the adjustment, but all with the same excess paste percentage and water-to-binder ratio. The volume of silica fume was relatively constant as well.

Reno-SHPC was tested for compressive strength and flexural strength. One of the two SHPC beam connections that are detailed in Chapter 6 used Reno-SHPC. LV-SHPC was tested for compressive strength, flexural strength, slant shear bond strength, and restrained shrinkage. Omaha-SHPC was tested for compressive strength and was used in the second SHPC beam connection.

5.5.2 Results

Table 5.5 shows the fresh concrete properties of the SHPC mixes.

Table 5.5 Fresh concrete properties of SHPC

Mix ID	Slump flow before fiber, in.	T_{500} before fiber	Slump flow after fiber, in.	T_{500} after fiber	Fresh unit wt, pcf
Reno-SHPC	30.50	12 seconds	29.75	17 seconds	147.8
LV-SHPC	30	5 seconds	27.75	10 seconds	143.28
Omaha-SHPC	29.25	13 seconds	24.75	27 seconds	154.3

The slump flow test yielded similar values for all three sets of materials. All three mixes were still well above the minimum for SCC (18”) after fiber was introduced. Omaha-SHPC did flow much slower, but still had a high spread.

The compressive strength results can be seen below.

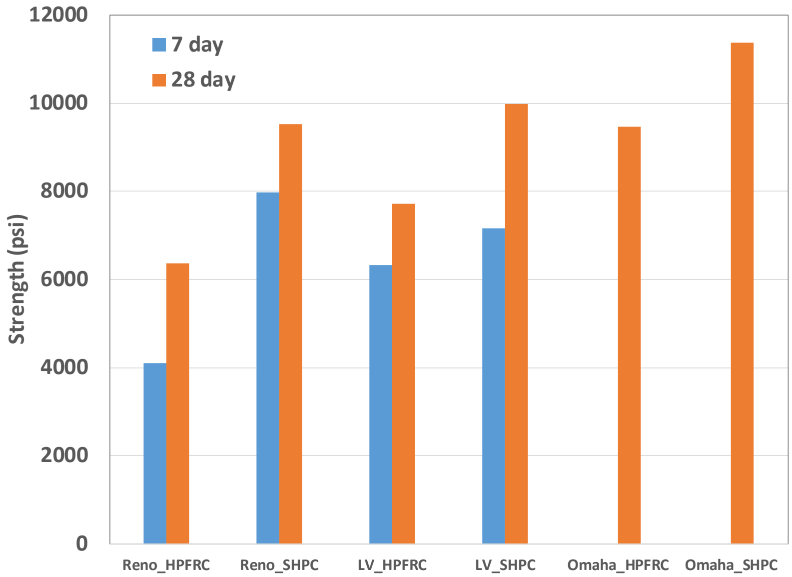


Figure 5.7 Phase II SHPC compressive strength results

Compared to the Reno and Las Vegas FRC mixes, the compressive strength was significantly increased for SHPC. With a compressive strength goal for 10,000 psi at 28 days, two out of the three mixes met the criteria. Reno-SHPC, though not at the 10,000 psi value, is still approximately 3,200 psi stronger than its FRC counterpart. Note: There was no Omaha FRC mix, hence why there is only SHPC data in the chart for Omaha.

The results of the flexural strength tests can be seen below.

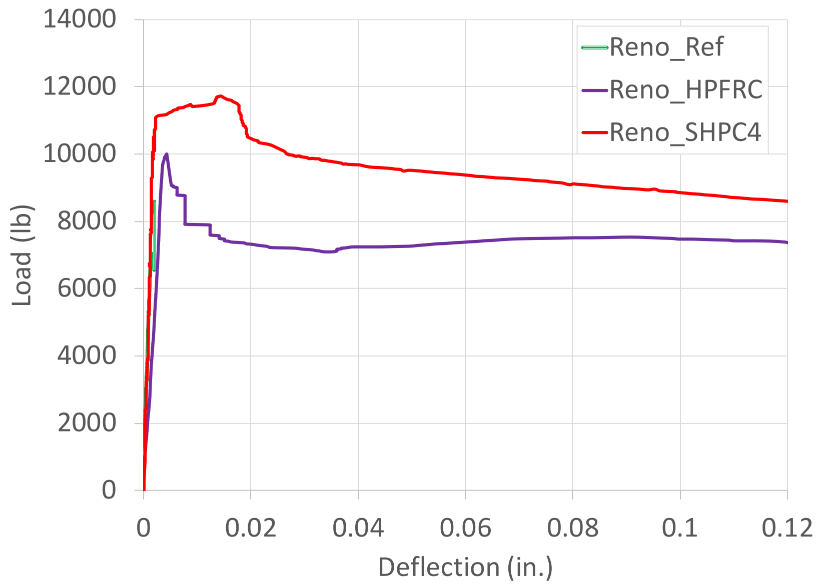


Figure 5.8 Reno SHPC flexural results

Reno-SHPC has a modulus of rupture of 976 psi versus the 833 psi value for the FRC mix. The toughness is also considerably higher at 1060 lb-in compared to 867 lb-in.

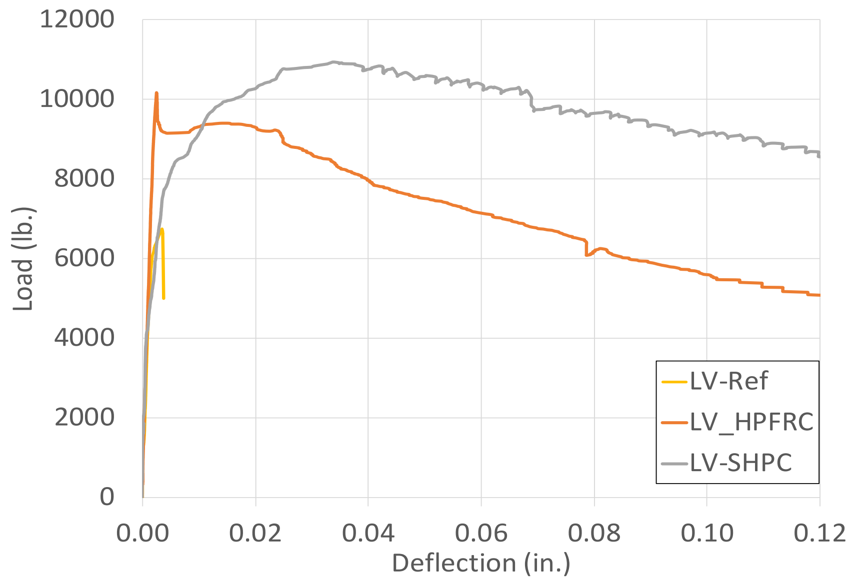


Figure 5.9 LV SHPC flexural results

LV-SHPC exhibited a minor increase in modulus of rupture in comparison to the FRC mix, with the value increasing from 848 psi to 912 psi. The toughness, however, increased considerably with the FRC toughness being 867 lb-in versus the 1240 lb-in toughness of LV-SHPC.

Using the same Nebraska DOT bridge deck mix (47BD) as the base material, the slant shear bonding also improved for SHPC compared to FRC. The results of this test are presented below.

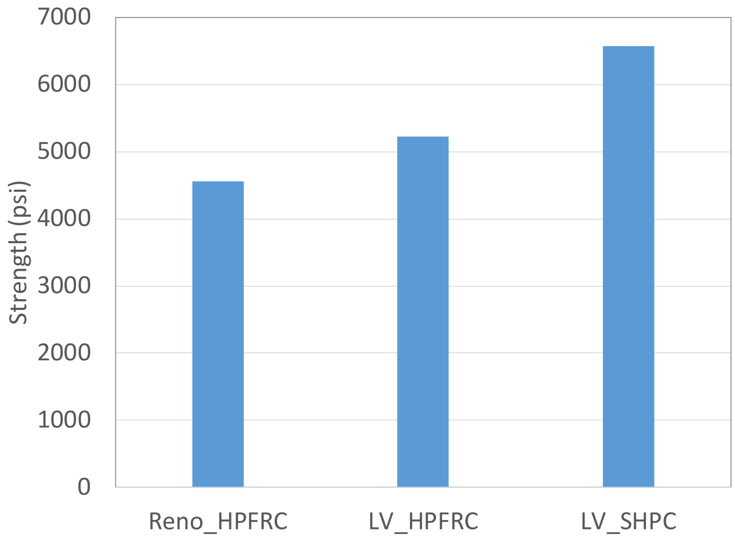


Figure 5.10 SHPC slant shear bonding results

The Reno and Las Vegas FRC specimens broke at the bonding interface. The bonding of LV-SHPC, however, held out, and the base concrete broke first. A comparison of the fracture types can be seen in Figure 5.11. Bonding on a larger scale was tested for Reno and Omaha SHPC in a beam connection. See Section 6.3 for more detail.

LV-FRC LV-SHPC (bottom layer)

Figure 5.11 Comparison of bond strength for FRC versus SHPC

The interface of the FRC mix is smoothly detached, indicating a clean bonding failure. This contrasts with the SHPC mix, which had no cracking and improved bonding.

The SHPC restrained shrinkage ring performed well when considering its repeated cracking. The graph in Figure 5.12 shows more than one instance of cracking, particularly at 7.5 days and 23.75 days. In both of these cases, the strain did not return to zero indicating that the fibers acted in a manner similar to a flexural test in which they bridge the crack and limit propagation.

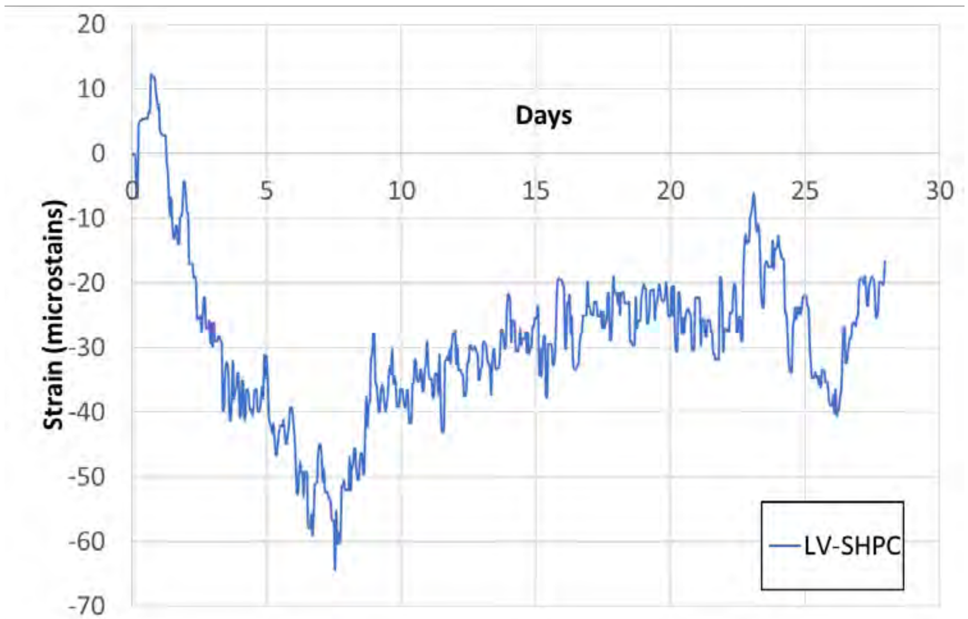


Figure 5.12 LV-SHPC restrained shrinkage result

5.6 Summary

The developed SHPC results show, at the very least, a starting point for a concrete that could be viewed as an alternative to UHPC. The cost is also much cheaper than UHPC. A full breakdown of material costs of FRC, SHPC, and UHPC is shown in the table below.

Table 5.6 Unit cost of materials

Material	Unit cost	Unit
I/II cement	$130	Ton
#67 Coarse Aggregate	$25	Ton
#89 Coarse Aggregate	$40	Ton
Reno Fine Aggregate	$18	Ton
#10 Sand	$20	Ton
T5 Fibers	$1,300	Ton
UHPC Fibers	$2,600	Ton
Water	$2.5	Ton
HRWR	$20	Gallon
VMA	$18	Gallon
Stabilizer	$18	Gallon
Air entraining agent	$7	Gallon

A cost analysis revealed the cost of SHPC to be approximately 50% of the cost of non-commercial UHPC developed form Nebraska DOT and 16% of the cost of a commercially available UHPC. The table below shows the cost comparison. Details of the non-commercial and commercial UHPC mixes can be found in Mendonca et al. (2020).

Table 5.7 Cost comparison – FRC, SHPC, UHPC

Mix Type	Cost ($/yd^3)
FRC	$211
SHPC	$325
UHPC (Developed with Nebraska Materials)	$656
Commercial UHPC	~$2000

A potential measure to further improve the compressive and flexural strengths and really separate this concrete from HPC would be to introduce a higher amount of silica fume into the mix design. Silica fume will lower the workability parameters of SHPC, but with the results presented in this study showing a very flowable concrete, there is room to cut back on the flowability for the sake of improving strength. A secondary measure could be to add more HRWR. This will not only serve to mitigate some of the flowability loss caused by the additional amount of silica fume, but may also increase mechanical properties as well.

CHAPTER 6. LAB-SCALE SLAB AND CONNECTION TESTS

6.1 Introduction

Two large scale slab pours, and two beam connection castings were carried out to analyze the performance of the FRC and SHPC mixes in a more realistic setting. The slabs were cast for the purpose of observing the constructability of the FRC mixes. Two beam connections were cast to evaluate the constructability of the SHPC mixes and determine the bonding strength between SHPC and a pair of HPC T-beams.

6.2 FRC Slab Construction and Testing

Eight slabs were cast to analyze the constructability and workability on a larger scale than typical lab specimens. Two pours, one without fiber and one with fiber, were conducted using the Nebraska state bridge deck mix design and its excess paste adjusted FRC design. In each casting, four slabs of dimensions 16’x3’x8” were made using rebar spacings of 6”, 9”, 12”, and 18”, respectively.

6.2.1 Mix designs

The same Lincoln mix designs seen in Chapter 4 were used for the slab pours. They can be seen below in Table 6.1 for reference.

Table 6.1 Mix designs for slabs

Mix ID	Unit	Type IP Cement	Water	#57 C.A.	Sand & Gravel	T5 Fiber
Lincoln-Ref	pcy	657	255	867	1993	0
	cf	3.52	4.08	5.22	12.19	0.0
Lincoln-FRC	pcy	721	280	785	1887	194
	cf	3.87	4.48	4.73	11.54	0.40
w/b: 0.39 Air Content: 6.0-8.5% Excess Paste: 9.38% Admixtures → WR: 6 fl oz/cwt AEA: 1 fl oz/cwt						

6.2.2 Formwork and specimen design

Wooden formwork was built for the first pour and reused for the second pour. Two forms, each holding two slabs, were constructed. The schematic for the formwork is shown in Figure 6.1.

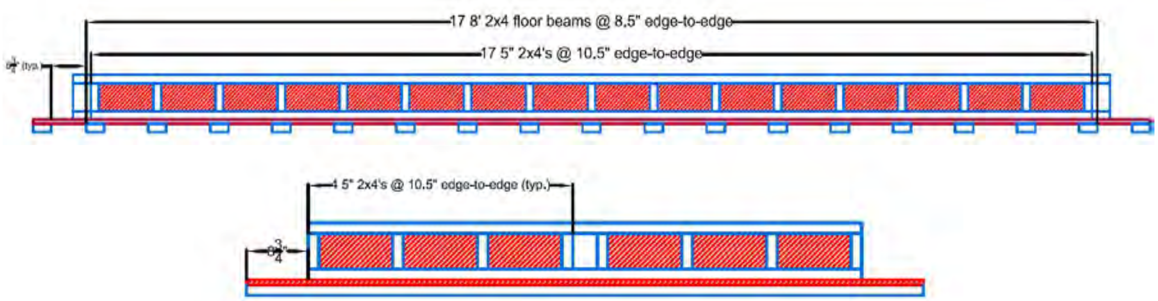
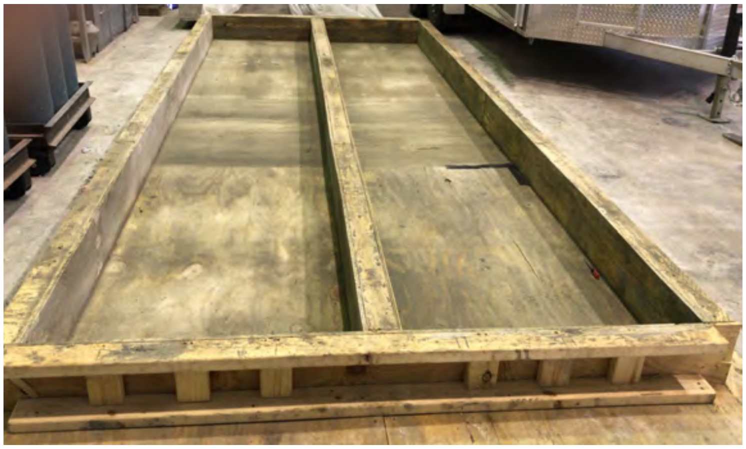


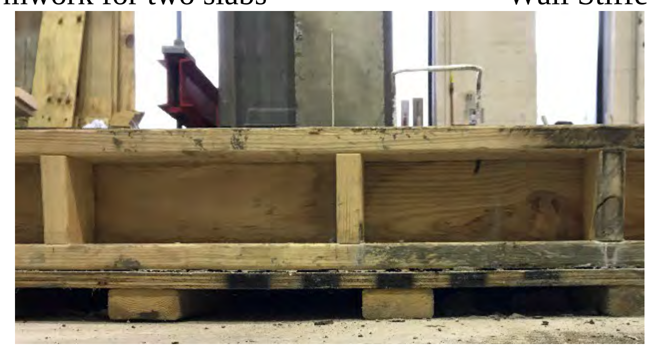
Figure 6.1 Side and end views of formwork schematic

Seventeen 2x4 floor beams were used with ¾” plywood on top as the bottom surface. 2x4’s were used to stiffen the walls, which were ½” thick plywood. Images of the formwork before placing rebar can be seen below.



Formwork for two slabs

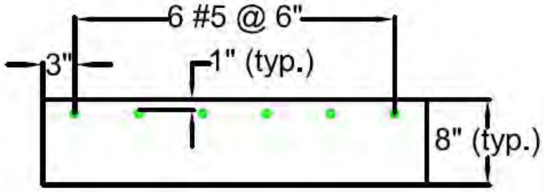
Wall Stiffeners



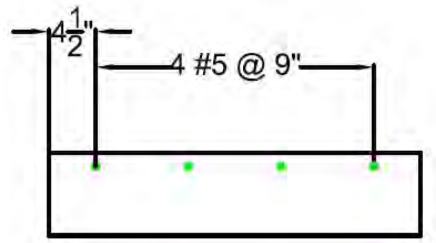
Floorbeam Setup

Figure 6.2 Formwork setup

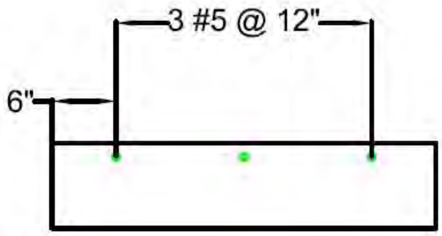
#5 rebar was placed using 6”-high rebar chairs (giving a concrete cover of 1⅜”) with different rebar spacings for each slab. Rebar was spaced at 6”, 9”, 12”, and 18”, with 9” and 12” being the most common spacings seen in bridge decks. A slab cross-section sketch of the rebar plan, with lifting inserts and chair positioning, is presented in Figure 6.3.



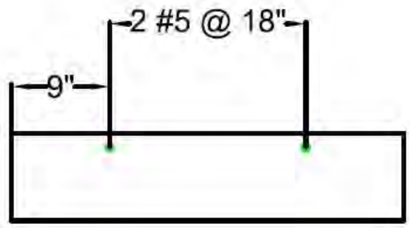
6” spacing



9” spacing



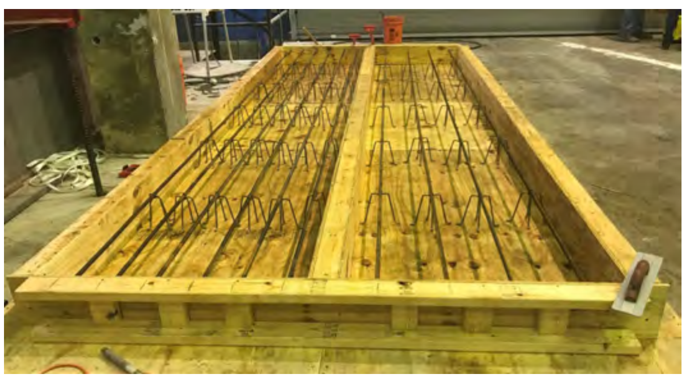
12” spacing



18” spacing

Figure 6.3 Rebar spacing sketch

The rebar alignment can be seen below in Figure 6.4.



6” and 9” rebar spacing

12” and 18” rebar spacing

Figure 6.4 Formwork with rebar alignment

Formwork was oiled thoroughly before casting.

6.2.3 Mixing and Casting – Normal Concrete

The standard state mix was cast first. The batch volume was 6.0 cubic yards. This mix had a slump of 6.0”, which was excellent for comparison to the FRC mix. A picture of the slump test result, which shows a concrete with very high consistency, can be seen in Figure 6.5.

Figure 6.5 Slump test for non-FRC slab

The slabs were cast in two layers, with the bottom layer being cast for all four slabs before returning to cast the second layer. Before vibrating, concrete was spread out with shovels. The vibration was then applied to the concrete while walking on top of the formwork (as seen above), and three evenly spaced locations were vibrated at approximately one-foot intervals. Figure 6.6 shows the difference in the appearance of the poured concrete versus the vibrated concrete in the first layer.

Figure 6.6 Slab concrete before and after vibration

The same procedure of vibration was applied to the second layer as well. A screed was then used to level off the slab before being finished with a bull float (Figure 6.7).

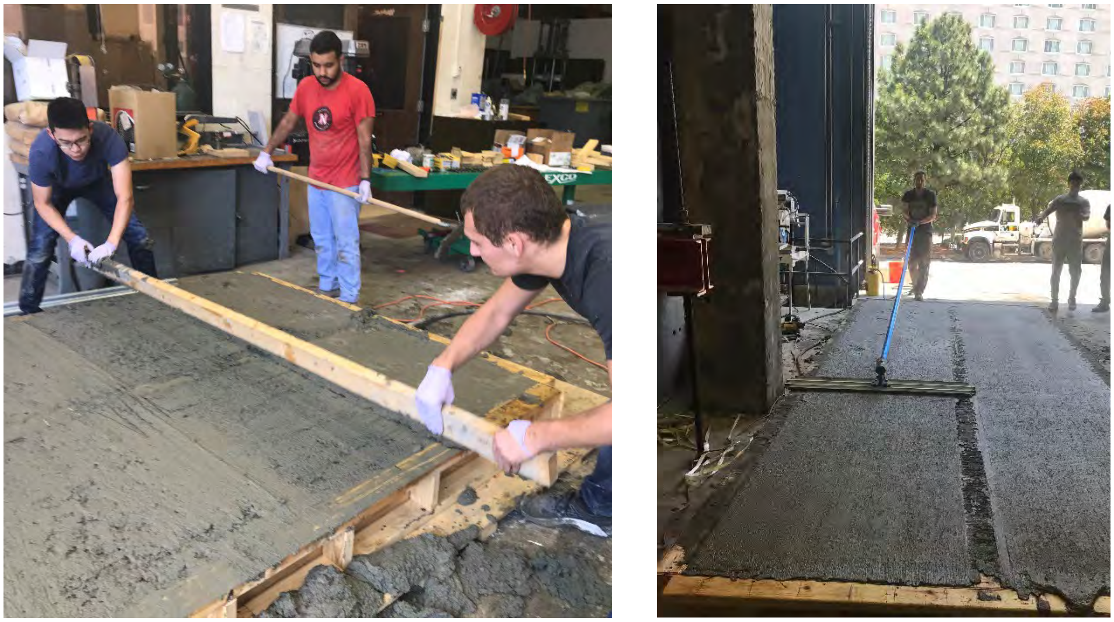


Figure 6.7 Non-FRC slab finishing with screed and bull float

Lifting inserts were placed in the concrete after the concrete had reached a more plastic state. The final product, with inserts in place, and the burlap and plastic wrap for curing, are shown below. The slabs were cured under the wet burlap and plastic wrap for 7 days.

Slab with inserts Burlap and plastic wrap for curing

Figure 6.8 Non-FRC slabs inserts and curing

6.2.4 Mixing and Casting – FRC

A common concern with fiber reinforced concrete is whether it will have good finishability in the field. Fibers may protrude out from the surface and can be pulled up when doing finishing. This function of concrete constructability is vital and was the primary focus of the FRC slab casting.

Fibers were added to the mixing truck just before the slab pour. Before this, however, a slump test was performed. After the slump test yielded a 5.5” to 6” slump, fibers were loaded into the truck. For the 5.5 yd3 batch, approximately 1065 lbs of fiber were loaded manually with buckets. Each bucket was poured slowly into the revolving mixing truck. Figure 6.11 shows fiber being introduced to the mixer from above.

Figure 6.9 Introducing fiber into mixing truck

The mixer was then run at mixing speed for an additional four minutes. After this, another slump test was performed (4”). The comparison of the slump before and after the introduction of fiber can be seen below.

Figure 6.10 FRC slab slump test before and after fiber

The concrete before and after fiber was very comparable in appearance. Vibration had a significant impact on making the concrete very workable and compatible. See Figure 6.11 for an image of the concrete before and after vibration.

Figure 6.11 Impact of vibration on FRC workability

Though this concrete did have a high slump for FRC, vibration greatly benefitted the FRC and essentially made it indistinguishable from the non-FRC mix. As was stated before, however, finishing FRC is often a concern. Using the same screeding and bull float process as before, the concrete was finished and showed no sign of difficulty in doing so. No fibers were visible at all on the surface. It is believed that the higher paste content provided by the excess paste adjustment method resulted in concrete of high workability, finishability, and overall constructability. The FRC slabs being finished, and the final product can be seen below in Figure 6.12.

Bull float finishing Finished slabs with inserts

Figure 6.12 FRC slab finishing

A visual examination was performed on the appearance of the slabs after the formwork was removed. As shown in Figure 6.13, all the FRC slabs appear to be well-consolidated with no sign of honeycomb fiber that was observed on the surface of the side of the slabs. A very small amount of fiber was found at the corner of the formwork.

Figure 6.13 FRC slab final product

6.2.5 Test setup and procedure

To evaluate the structural behavior of the concrete slabs made with plain concrete and FRC, a test set up, as shown in Figure 6.14 was used. The testing setup was designed to load the slabs at the ends producing a constant moment region in the center. Roller support was obtained by placing a 1 in. diameter and 36 in. long steel rod between 0.5×12×36 in. flat steel plates while a pin support was obtained by placing a 1 in. diameter and 36 in. long steel rod between 0.75×12×36 in. steel plates that were grooved 0.25 in. Based on the calculated yield capacities, the load of the specimens was applied in two kips load increment. The load from each ram was monitored by KCB-500kN load cells (±125,000 lb capacity and 1.080 mV/V sensitivity) manufactured from Tokyo Sokki Kenkyujo Co., Ltd. or from Honeywell Inc. Deflections were monitored using ten string potentiometers (two at midspan, two at each support, and two at each loading points) manufactured by UniMeasure Inc. (range and sensitivity listed in Table 5.5). Two string potentiometers were installed at. Load and deflection data were collected through a VISHAY data acquisition System 7000-128-SM and monitored at 0.1-second intervals during testing using VISHAY StrainSmart Ver.4.7.25. The string potentiometers for deflection measurements were calibrated using a Fowler Trimos height gage (Model 600+, 24 in. travel) with an accuracy of 0.00005 in.

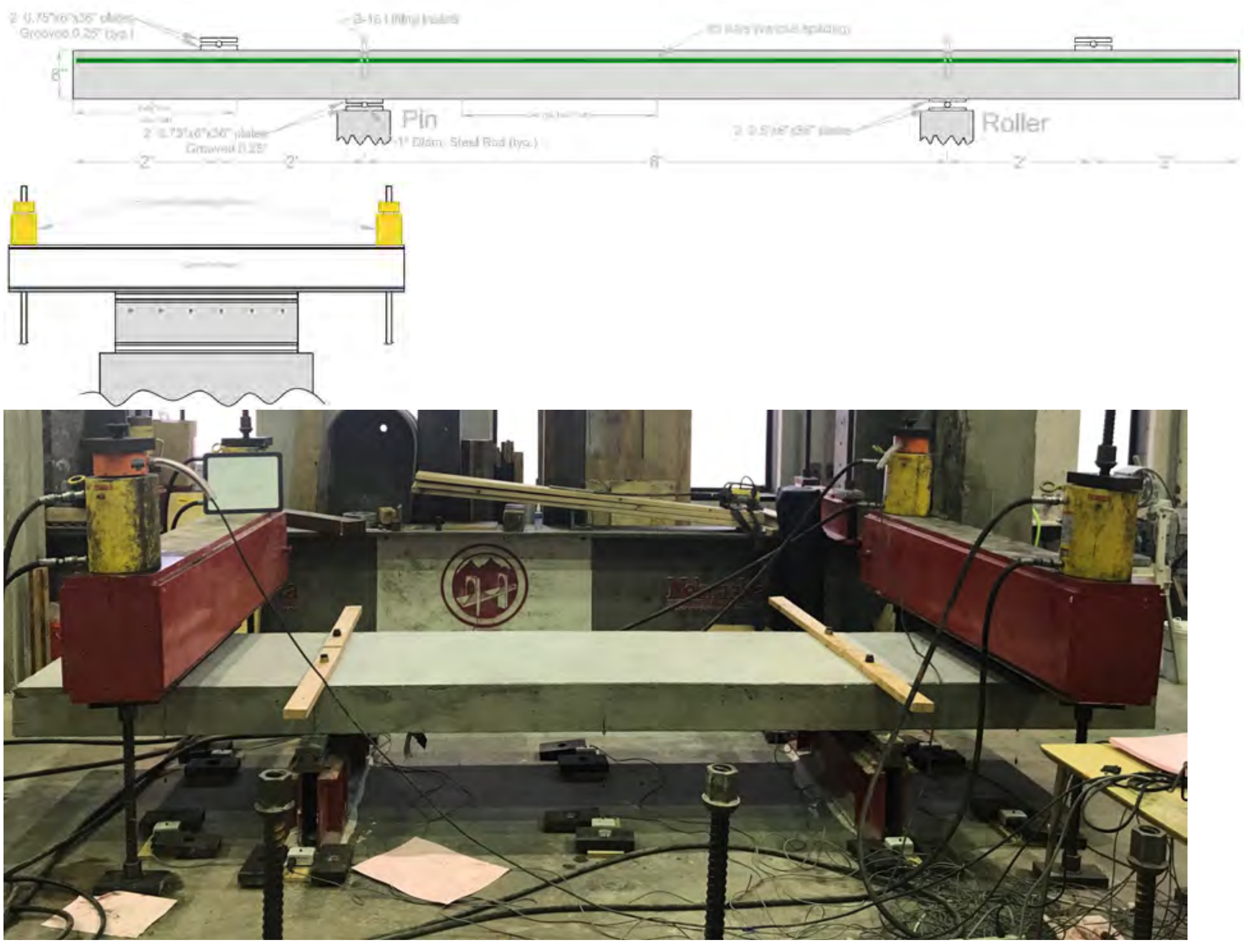


Figure 6.14 Slab specimen test set up

At each load stage, cracks were marked with a permanent parker and crack widths in the constant moment region were measured using a crack width microscope with the magnification of 40x and accuracy of 0.02mm, see Figure 6.15. A crack was considered a primary crack if it connected through the full width in transverse direction. The width of each primary crack was measured in three different locations on the top surface and the average width was reported.

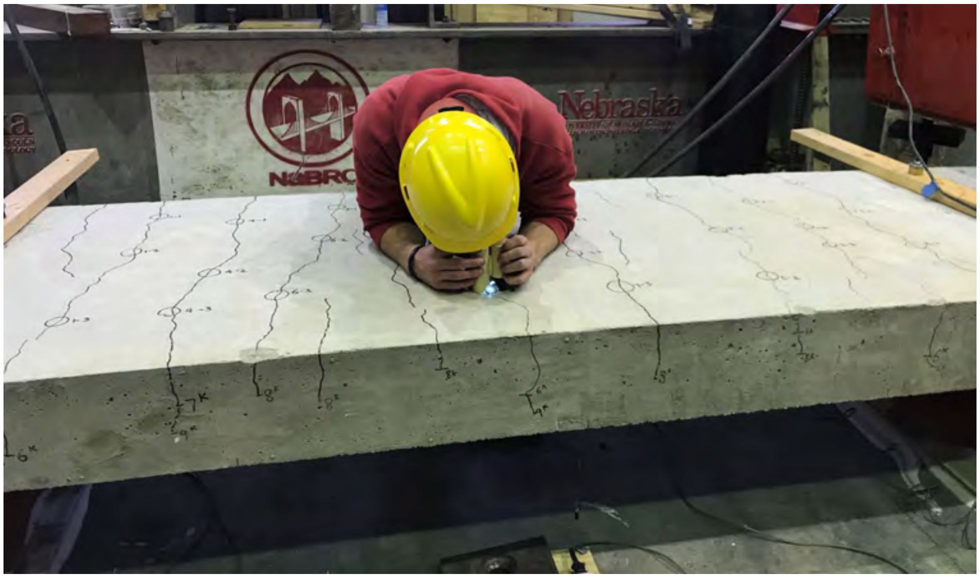


Figure 6.15 Crack width measuring during testing

6.2.6 Test results

The experimental results slab testing are presented to examine the effect of fiber reinforcement of concrete, and bar spacing on crack width and crack spacing. Further analysis was performed to justify the advantages of using fiber reinforced concrete in bridge deck applications.

Figure 6.16 illustrates the load-deflection curves of the slabs with 12 inches of bar spacing. Noted that the graph starts at 7 kips because of the dead load of the supports. The load-deflection behavior was similar except the fiber-reinforced concrete slab experiencing a higher load before the first cracking due to the fact that up to cracking, the stiffness of the slabs is primarily governed by the concrete. After the cracking, when the stiffness is controlled by the steel reinforcement, which was similar for the two specimens, the behavior seems very similar.

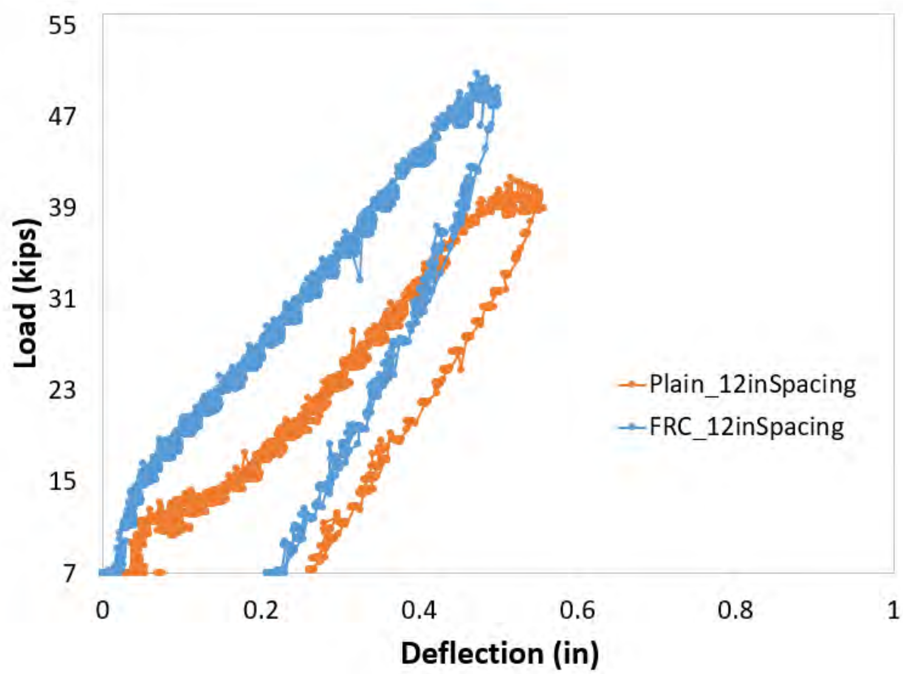
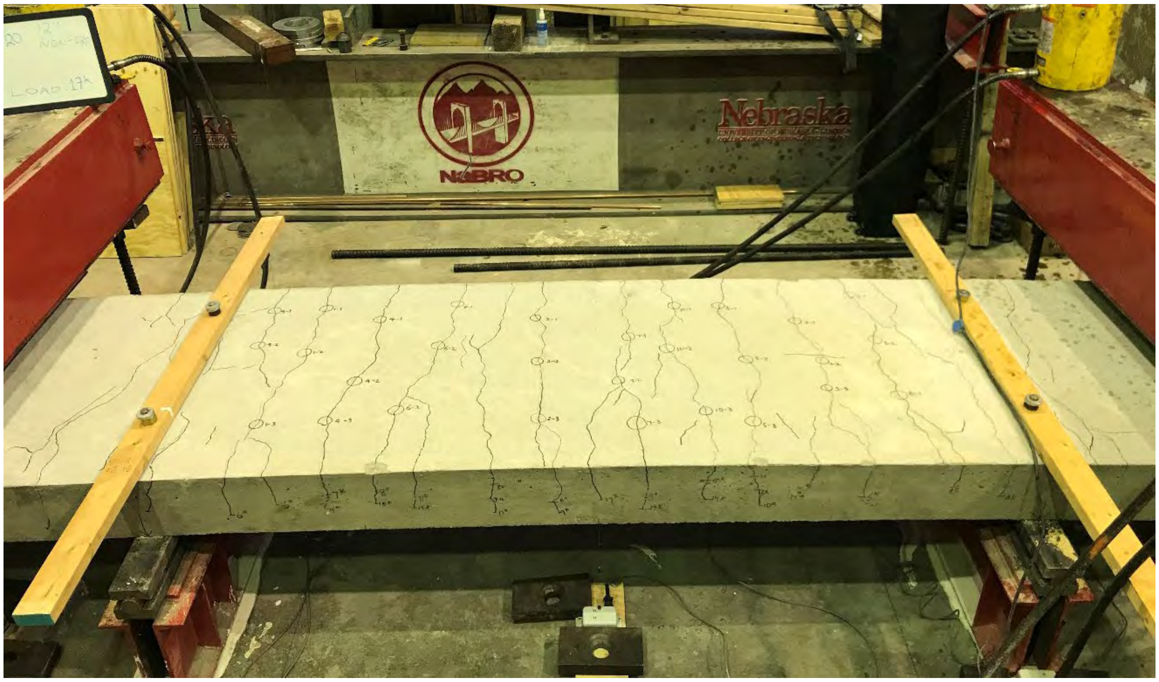


Figure 6.16 Load-deflection curves of the slabs with 12 inches bar spacing

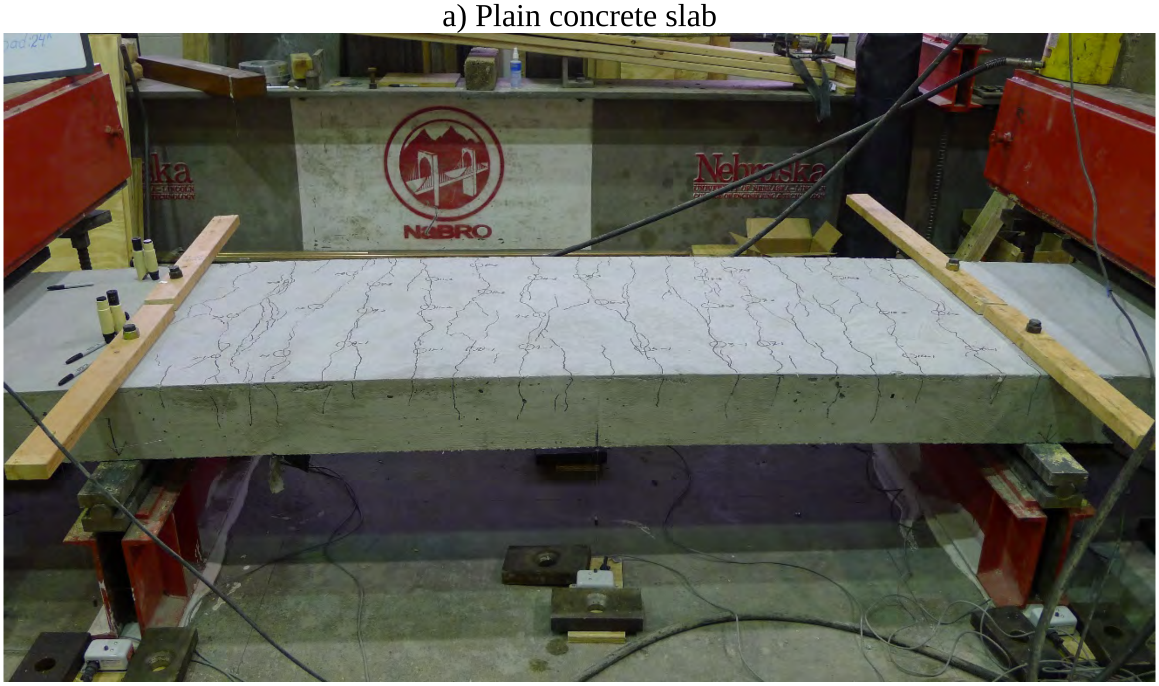
Figure 6.15 shows the cracking patterns for the two specimens. It can be seen that in plain concrete the cracks are mostly singular, whereas in the FRC slab, multiple cracks with multiple branches developed. Table 6.2 summarizes the number of primary cracks and their spacing. FRC slab had two more cracks compared to the plain concrete slab. As expected, when FRC is used, a higher amount but smaller cracks were observed.

Table 6.2 Number and spacing of primary cracks

Specimen	Number of primary cracks	Minimum spacing (in.)	Maximum spacing (in.)	Average spacing (in.)
Plain_12inSpacing	9	5.75	11.00	8.50
FRC_12inSpacing	11	3.50	14.00	8.00



a) Plain concrete slab

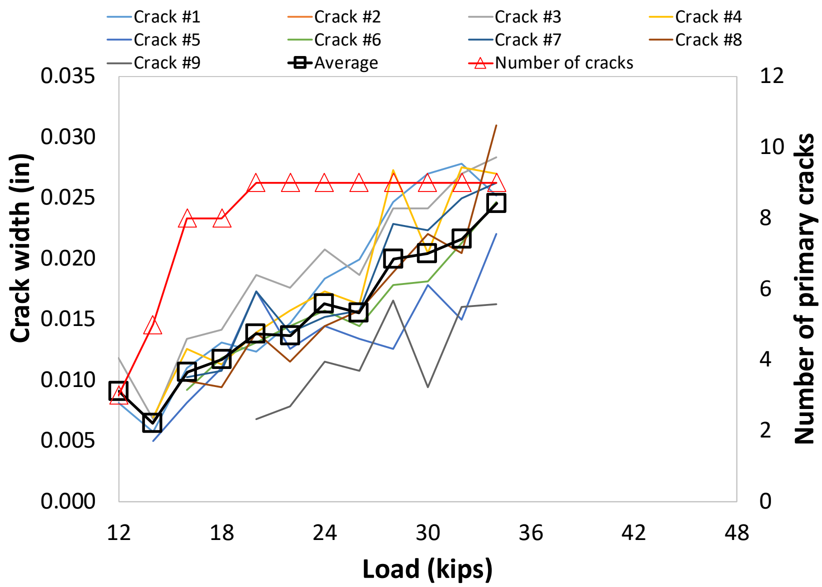


a) Fiber-reinforced concrete slab

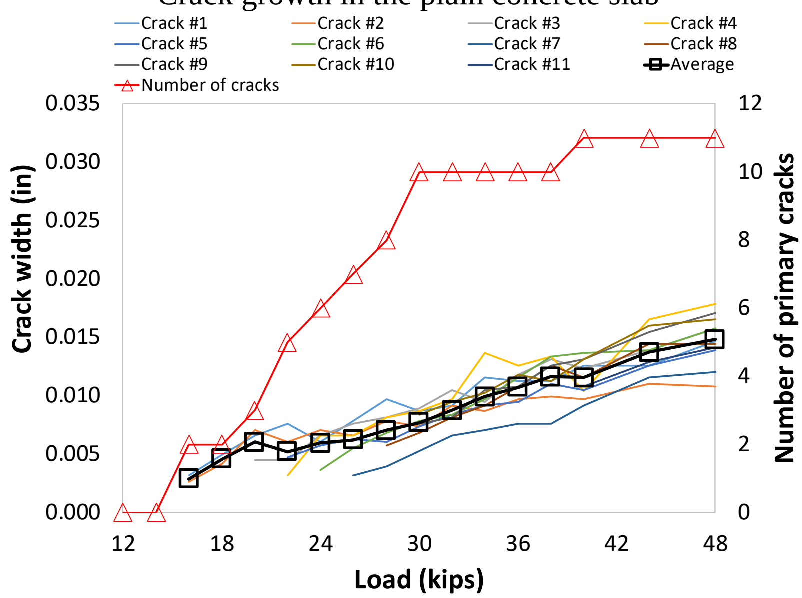
Figure 6.17 Load Crack patterns in the slabs with 12 inches bar spacing

The crack width was measured from three different equally spaced locations for every primary crack. Figure 6.18 demonstrates the growth of each crack, along with the average crack growth. A clear decrease in crack width and their growth can be noticed. The average crack widths for plain and fiber reinforced concrete slabs at ultimate stress were 0.025" and 0.015" respectively, whereas the maximum crack widths were 0.031" and 0.018" respectively. The difference in crack growth can be defined as a slope of the average crack width growth, which is 0.0007 and

0.0004 for plain and fiber reinforced concrete, respectively. It can also be noted that when plain concrete was used, all primary cracks were developed at 20 kips load, and then their number remained unchanged at higher stresses, which is consistent with the previous studies (Hognestad 1962; Kaar and Mattock 1963). While under the same load, FRC slab had only three primary cracks. It is clearly seen that primary cracks keep developing throughout the loading period. The higher energy required to develop primary cracks can be explained by the presence of fibers, which act as micro-reinforcement to bridge cracks and delay their propagation. Besides the reduced crack widths under the same load levels, the developed HPFRC slab also showed higher loading capacity due to the higher strength of the concrete.



Crack growth in the plain concrete slab



Crack growth in the fiber-reinforced concrete slab

Figure 6.18 The difference in crack growth in plain and fiber reinforced concrete slabs

Noted that due to the limitation of equipment availability, full-scale mechanical properties were only performed on the two slabs with 12” rebar spacing. However, it is expected that the same trend, i.e., the presence of fiber can effectively delay crack propagation generated by the applied load can be observed with slabs prepared with other rebar spacings.

6.3 SHPC Beam Connection Construction and Testing

Reno-SHPC and Omaha-SHPC were cast as connections between two T-beams. The T-beams were made of high strength concrete and had rebar extruding at the connection.

6.3.1 Mix Designs

The same mix designs for Reno and Omaha SHPC discussed in Chapter 5 were used for the connections. For reference, they are shown below.

Table 6.3 SHPC beam connection mix designs

<table>
<tr><th>Mix ID</th><th>Unit</th><th>Cement Type I/II</th><th>Silica Fume</th><th>Class F Fly Ash</th><th>Water</th><th>#67 C.A.</th><th>#89 C.A.</th><th>F.A.</th><th>T5 Fiber</th></tr>
<tr><td rowspan="2">Reno-SHPC</td><td>pcy</td><td>946</td><td>81</td><td>313</td><td>328</td><td>779</td><td>139</td><td>1021</td><td>258</td></tr>
<tr><td>cf</td><td>4.81</td><td>0.59</td><td>1.95</td><td>5.25</td><td>4.80</td><td>0.86</td><td>6.28</td><td>0.53</td></tr>
<tr><td colspan="10">w/b: 0.26 Excess Paste: 22.99% Air Content: 6.0% VMA: 3.6 fl oz/cwt
HRWR: 13.2 fl oz/cwt Stabilizer: 3.6 fl oz/cwt AEA: 1.2 fl oz/cwt</td></tr>
<tr><th>Mix ID</th><th>Unit</th><th>Cement Type IP</th><th>Silica Fume</th><th>Water</th><th>#67 C.A.</th><th>#89 C.A.</th><th>Sand & Gravel</th><th colspan="2">T5 Fiber</th></tr>
<tr><td rowspan="2">Omaha-SHPC</td><td>pcy</td><td>920</td><td>79</td><td>244</td><td>981</td><td>176</td><td>1283</td><td colspan="2">258</td></tr>
<tr><td>cf</td><td>4.93</td><td>0.57</td><td>3.91</td><td>5.93</td><td>1.06</td><td>7.77</td><td colspan="2">0.53</td></tr>
<tr><td colspan="10">w/b: 0.26 Excess Paste: 23.00% Air Content: 7.25% VMA: 4.3 fl oz/cwt
HRWR: 15.0 fl oz/cwt Stabilizer: 4.3 fl oz/cwt AEA: 2.2 fl oz/cwt</td></tr>
</table>

6.3.2 Mixing and Casting

The same mixing procedure described in Section 5.2 was used in the connection casting. Considering the mixer was much larger (5.0 ft^3 capacity) than the mixer used for smaller batches (1.7 ft^3 capacity), some HRWR was reserved as a precaution. However, in both cases, all HRWR was used, and the mixes were very consistent with the smaller batches. The slump flow results are shown below.

Table 6.4 Slump flow results for connection mixes

Mix ID	Slump flow before fiber, in.	T_{500} before fiber	Slump flow after fiber, in.	T_{500} after fiber
Reno-SHPC	25.25	N/A	27.75	N/A
Omaha-SHPC	28.25	7 seconds	24.50	22 seconds

In the case of Reno-SHPC, the slump was higher after introducing fiber because the remaining HRWR that was on the reserve was added in at this time as well. The vertical surfaces of the concrete beams at the connection were sprayed with mist just before mixing to improve bonding. The loading of fibers and subsequent product inside the mixer can be seen below.

Figure 6.19 Mixing of SHPC for large batch

As shown in Figure 6.20, the formwork and reinforcement were cleaned and pre-wet prior to the start of the mixing process. Moist towels were used to cover the form to keep them moistened until the mix was ready to be placed. Upon the completion of mixing, concrete was then transported from the mixer to the connection by buckets. The connection setup and pouring of the Reno-SHPC connection are seen in Figure 6.20. The concrete flowed into place with no form of consolidation, and a hand trowel was used for surface finishing. Concrete was cured for seven days using wet towels and plastic wrap that was weighed down to trap moisture.

Figure 6.20 Beam connection preparation and placement

6.4 Test Setup and Test Procedure

In this study, full-scale specimens with the staggered splice joint filled with four different mixes were prepared and evaluated. The four mixes are Reno SHPC (F-R-SHPC), Nebraska SHPC (F-N-SHPC), a commercial UHPC (F-L-UHPC), and locally developed UHPC (F-N-UHPC). The width of the joint was 6 in.

The testing rig was set up for a three-point bending test with a hydraulic ram placed in the middle of the specimen and supported at the two ends of the specimen, as shown in Figure 6.21. However, since two specimens were connected, the stem portion of the precast section were placed on four supports. The hydraulic loads were applied in a small increment during testing until there was a significant drop in load, and the specimens were under rotation. Displacement was measured through string potentiometers placed next to the shear key in both sides at the location of loading point, quarter-point, and at supports.

Figure 6.21 Full-scale slab connection specimen test setup

After the testing was complete, the crack pattern was carefully observed by the research team. And as shown in Figure 6.22, the crack followed a pattern that would typically be observed in yield line analysis of two-way slab specimens. Inclined crack was formed stretching out between the load plate and the supports.

Figure 6.22 Crack mapping and failure of specimen F-R-SHPC

Figure 6.23 shows the load-displacement curve for the slabs prepared with the four different joints. With the Reno and Nebraska SHPC, the specimen was able to reach 84.5 kips, which was comparable with the two UHPC mixes. After testing reached this maximum load, the load did not increase and remained at the peak level while rotation was taking place. Then, the load dropped during rotation, and the test was terminated when cracks on the flange section were observed on the slab, and inclined cracks were observed at the stem of the slab specimen. This was when the steel plates started to punch through the concrete resulting in punching shear failure mode. The initial stiffness varied between these four specimens with F-R-SHPC having the highest initial stiffness, followed by the F-N-UHPC. The F-N-SHPC, and F-L-UHPC specimen had comparable initial stiffness. The displacement at yield was found to be at approximately 0.7 in. when the load was approximately 85 kips for the four specimens. After the specimens reached the yield strength, the T sections started to rotate with an increase in deflection at the loading point without any damage observed at the transverse joint. In addition, the load did not drop until started to unload the specimen for the two UHPC specimen until punching shear failure was observed on the top and bottom of the top flange portion of the T section while there was a sudden drop in load while rotation was taking place for the SHPC specimens, which indicate that the UHPC specimens had better ductility than the SHPC specimen. Although the initial stiffness of these specimens varied, they all reached the yield strength of approximately 85 kips.

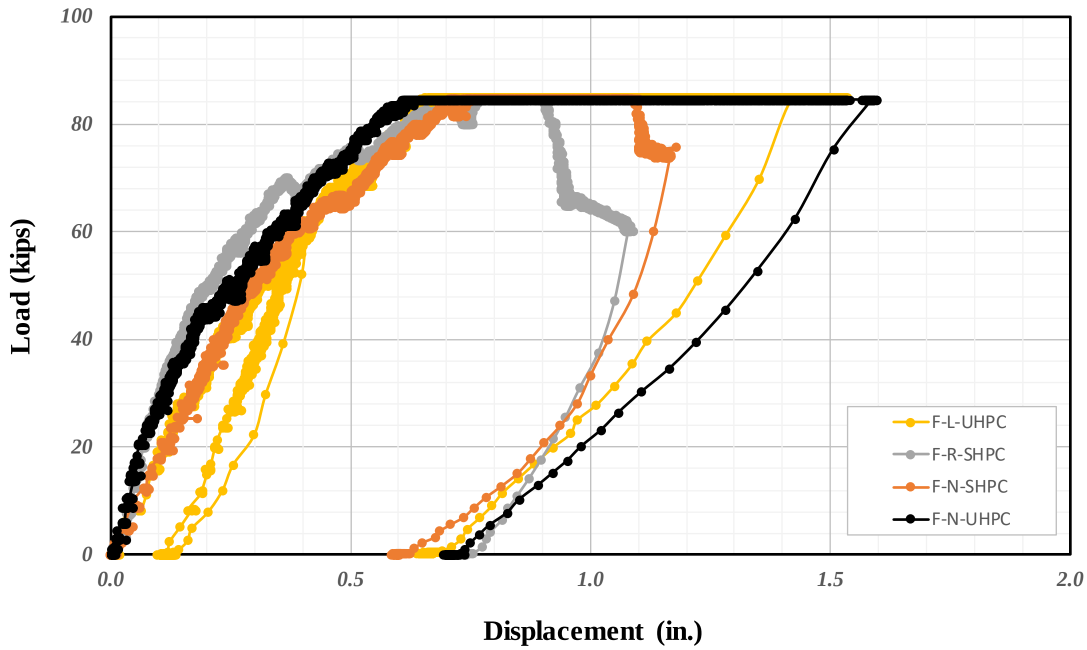


Figure 6.23 Load-Displacement of Specimens with UHPC and SHPC Joints

Overall, both UHPC and SHPC specimens behaved in a desirable manner with good ductility until failure was reached. None of the specimens with SHPC or UHPC joints failed prior to the T section failure. Results demonstrated that SHPC could serve as an excellent alternative to UHPC in bridge deck connection.

CHAPTER 7. RECOMMENDATIONS FOR HPFRC AND SHPC PRACTICE

7.1 Introduction

Based on the comprehensive survey of practices and specifications from state agencies that have experiences of FRC, as well as the experience from the small and large scale laboratory studies in this project, recommendations were presented for HPFRC and SHPC practice. This chapter presents detailed recommendations regarding the mix design, batching and mixing, quality control methods, and casting of HPFRC and SHPC. The information can be further used in the development of specifications for NDOT.

7.2 Mix Design

In general, mix designs for FRC (including both HPFRC and SHPC) typically have the following characteristics: higher cement content, smaller maximum size of aggregate, higher fine aggregate content, and the use of water-reducing admixtures. Based on the design and materials used in a standard bridge deck mix in Nevada, mix design adjustment can be made for FRC to achieve appropriate workability and hardened concrete properties. The aggregate-fiber skeleton void content test is a critical step in the mix design adjustment with the inclusion of fiber.

Based on the extensive study and multiple trial batches, two sets of HPFRC and SHPC mixes (one set based on Reno materials and another based on Las Vegas materials) were is recommended as an economical and feasible mix with good performance to be used in highway bridge decks or connections for NDOT. Table 7. 1 shows the mix design of the recommended mixes.

Table 7. 1 Mix design of recommended FRC and SHPC mixes

Mix	Cement	Fly Ash	Silica Fume	Water	#67 C.A.	#89 C.A.	F.A.	T5 Fiber
Reno-HPFRC	673	224	-	332	1074	192	1117	188
Reno-SHPC	946	313	81	328	779	139	1021	258
LV-HPFRC	604	151	-	302	1434	154	1172	192
LV-SHPC	864	255	75	293	883	153	1130	258

It should be noted that as HPFRC and SHPC are both very sensitive materials, any changes in the source of the raw material could result in different performance. Changes in the mixing procedure or volume of batch also can affect the final performance characteristics. Trail batches are therefore strongly recommended to ensure appropriate fresh and hardened concrete behavior.

7.3 Batching and Mixing

A pre-pour meeting including expected results, weather conditions during placement, batching procedures, fiber loading methods, placing method and sequence, quality control tests methods, and curing methods and procedures are recommended before every batch. For steel fiber, fibers should have no sign of rust prior to batching. Mixer operation, raw materials condition and quantities, formwork condition, and the weather condition is also needed. For bridge deck connection construction, a pre-pour inspection including examining the placement for cleanness (free of debris) and pre-wet to the SSD condition is also recommended.

Compared to conventional concrete production, while there is no major difference in the equipment and mixing method, it is important to have the appropriate fiber loading method to ensure efficient FRC production and prevent fiber balling. The most common method is to wait to load fiber until all other materials have been mixed together. The reason for the popularity of this method is that the fibers are in the mixer for the shortest amount of time and with the entirety of the coarse aggregate, both of which help to prevent fiber balling. It is recommended that steel fibers be added at a rate of approximately 100 lb/min and with a mixing speed of 40 revolutions/min. With steel fibers, it is common practice to load the fibers into the mixing truck directly. Another practice has been to use a conveyor belt to load fibers into the mixing truck so as to increase efficiency and maintain a constant stream of fibers for better dispersion. A screen with a mesh of 1.5” to 2.5” might be used to help prevent fiber balling.

7.4 Quality Control Methods

While a slump test is not always the preferred evaluator of workability for FRC since FRC tends to have a much lower slump then required for construction. In addition, an FRC mix may be workable when vibration is applied despite having a low slump value. Never the less, the slump test proved valuable in determining if the concrete had sufficient workability and consistency before adding fibers and comparing these characteristics with the concrete after adding fibers. A visual examination should be performed to exam the distribution of paste, aggregate, and fibers for appropriate cohesion and stability of the mixture. Test methods such as the vibrated L-box test or mockup test can be used to provide an objective evaluation of fresh concrete behavior prior to placing. For SHPC mixture, standard SCC test methods, including the slump flow and VSI (ASTM C1610) can be used.

For mechanical tests, a compressive strength test (ASTM C39) should be performed to determine if there is a negative impact on strength due to consolidation or fiber balling. The consolidation and no negative impact on strength. Flexural performance (ASTM C1609) should be performed to justify the improvement of the mechanical properties of the developed FRC. Test methods for the durability test and volume stability of the developed FRC mixes are expected to be the same for FRC, as compared to conventional concrete.

7.5 Casting

In typical volume ranges for FRC, the addition of fibers will likely reduce workability. To ensure appropriate consolidation, mechanical vibration is necessary. For surface finishing, top surfaces should be struck off with a screed and the concrete should then be finished with trowels or a bull float. Edging may be necessary to keep fibers from being exposed. The timing of sawing at joints is also critical so that macro fibers are not pulled up.

CHAPTER 8. SUMMARY AND CONCLUSIONS

8.1 Conclusions

Based on results from the experimental study on excess paste-based adjustment for High-Performance Fiber Reinforced Concrete (HPFRC) design and the development of Super High-Performance Concrete (SHPC), the following conclusions can be drawn:

- Adjusting a non-fiber reinforced concrete to incorporate fibers using the excess paste method based on maintaining the same excess paste volume is effective in maintaining workability while improving mechanical properties. The excess paste adjustment method was successful for three completely different sets of materials, all of which exhibited satisfactory workability with no visible segregation, significantly improved moduli of rupture, slightly to significantly improved compressive strength, and high toughness.
- As the traditional slump test is not capable of reflecting the true workability of HPFRC, a vibrated L-Box test was developed to evaluate the workability of HPFRC under vibration. The test assesses the flowability, and passing ability under vibration and was able to effectively measure HPFRC workability.
- Construction and performance of lab-scale slabs prepared with the developed HPFRC demonstrated that not only was the developed mix have sufficient workability to ensure appropriate placing, consolidation, and surface finishing in bridge deck construction, the HPFRC slabs also exhibited superior crack resistance compared to conventional bridge deck concrete prepared with the same reinforcement.
- SHPC is a potential alternative to ultra-high performance concrete (UHPC). Not only can the SHPC prepared with conventional drum-type mixers, but it also exhibits a self-consolidating level of workability, high strength, high toughness, strong bond strength, and excellent durability performance. A similar adjustment based on excess paste was used to convert the SHPC mix design to two other sets of materials with very comparable results.
- Full-scale bridge panel connection test justified that the developed SHPC mixes exhibit excellent constructability and mechanical behavior with good ductility that can serve as a cost-effective alternative to UHPC in bridge connection.

Based on the practices and specifications from state agencies, together with the experience from the experimental study included in this project, detailed recommendations regarding the mix design, batching and mixing, quality control methods, and casting of HPFRC and SHPC were included in the report. The information can be further used in the development of specifications for NDOT.

8.2 Recommendations for Future Studies

The main recommendation for future work would be to expand on the parameter of excess paste as an adjustment method. As it stands, this adjustment method relates the paste content to the void content of a particular aggregate blend. It is recommended that the excess paste parameter be manipulated to account for the common properties of aggregate. For example, using excess paste and its relationship to the particular aggregate surface area may be beneficial in determining a more suitable degree of paste.

Further modification of the vibrated L-Box test as a method for measuring flowability and passing ability of fiber reinforced concrete is recommended. Alterations to the dimensions of the L-box to allow for more concrete in the vertical leg and easier vibration insertion would help to eliminate some human factors associated with the test.

The primary recommendation for Super High-Performance Concrete is to sacrifice some workability and increase strength by including more silica fume. The slump flow values are all well within the range of self-consolidating concrete, and additional silica fume could serve to close the gap between SHPC and UHPC in regards to compressive and flexural strength. A secondary measure could be to add more high range water reducer. This will not only serve to mitigate some of the flowability loss caused by silica fume and maximize the amount of silica fume that could be added but may also further improve mechanical and durability properties.

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