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16. Abstract
Continuously reinforced concrete pavement (CRCP) is a major form of highway pavement in Texas due to its increase in ride quality, minimal maintenance, and extended service life. However, CRCP may sometimes experience pavement distress that results in early failure, either due to under-design or the use of poor construction materials. Significant effort has been made to improve the performance of some of these materials (e.g. siliceous river gravel) to achieve an acceptable level of performance but has been unable to provide a practical solution. This research study investigates whether fiber reinforcement may solve some of the problems associated with siliceous river gravel, particularly spalling.

The main objectives of this study were to: (1) Conduct a comprehensive literature review in order to determine the current state of the art regarding CRCP design and behavior as well as the role that fiber reinforcement may have in improving its performance; (2) Perform field investigations in order to verify constructability and workability of fibers in CRCP construction; (3) Perform frequent monitoring to evaluate the effect of fibers on crack spacing, crack width, and spalling development; (4) Perform laboratory testing that validate the effect of fibers on typical concrete paving mixes; (5) Provide TxDOT with recommendations as to possible changes in the construction and design specifications of CRCP, which could serve to reduce or prevent spalling.

Because the manifestation of spalling in CRCP may sometimes take several years, it is difficult to draw firm conclusions from this two-year study. However, based on the findings within the time frame of this project, fiber reinforcement did appear to prevent or limit spalling in the field test sections, when compared to the control sections that did not contain fibers. It is recommended that future monitoring of these test sections be performed to fully characterize the long-term efficacy of fibers in reducing or preventing spalling of CRCP.

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Fiber in CRCP Pavements

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Products

Product 3 (P3) is included in this report as Chapter 6, Summary, Conclusions and Recommendations

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Chapter 1. Introduction

1.1 Research Background

Continuously reinforced concrete pavement (CRCP) is a major form of highway pavement in Texas due to its increase in ride quality, minimal maintenance, and extended service life. However, CRCP may sometimes experience pavement distress that results in early failure, either through under-design or use of poor construction materials. Significant effort has been made to improve the performance of some of these materials (e.g., siliceous river gravel) to achieve an acceptable level of performance, but this has not resulted in a practical solution. This research study investigates whether fiber reinforcement may solve problems associated with siliceous river gravel, particularly spalling.

1.2 Research Objectives

This research study has the following objectives with respect to the prevention of spalling in CRCP:

1. Conduct a comprehensive literature review in order to determine the current state of the art regarding CRCP design and behavior, as well as the role that fiber reinforcement may have in improving its performance.
2. Perform field investigations to verify constructability and workability of fibers in CRCP construction.
3. Perform frequent monitoring to evaluate the effect of fibers on crack spacing, crack width, and spalling development.
4. Perform laboratory testing that validates the effect of fibers on typical concrete paving mixes.
5. Provide TxDOT with recommendations for possible changes in the construction and design specifications of CRCP, which could serve to reduce or prevent spalling.

1.3 Scope of Report

In order to realize the benefits of fibers in CRCP, it is important to first understand certain aspects of each element individually. Chapter 2 of this report gives a detailed background of CRCP with regard to materials, design, construction, and performance. Spalling is considered the most detrimental materials-related distress in the state of Texas, especially in the Houston area. Consequently, the main focus has been placed on trying to solve this problem with the implementation of fibers. A comprehensive review was also compiled that focuses on fiber-reinforced concrete. This primarily pertained to the different types of fibers and their effects on mix design, fresh concrete properties, hardened concrete properties, and constructability.

Chapter 3 summarizes a comprehensive laboratory research program aimed at quantifying the benefits of using steel or synthetic fibers in concrete. These efforts culminated in the selection of materials and mixture proportions that were used in two full-scale field trials, discussed in detail in Chapters 4 and 5. Lastly, Chapter 6 summarizes the key project findings and provides recommendations and guidance for future use of fiber reinforcement in CRCP applications.

Chapter 2. Literature Review

Continuously reinforced concrete pavement (CRCP) is a major form of highway pavement in Texas due to its high ride quality, minimal maintenance requirements, and long service life. However, CRCP may experience pavement distress that results in early failure due to under-design or the use of poor construction materials. Significant effort has been made to improve the performance of some of these materials (e.g., incorporating siliceous river gravel) to achieve an acceptable level of performance. This report evaluates the potential benefits that fibers may provide in CRCP.

In order to realize the benefits of fibers in CRCP, it is important to first understand certain aspects of each element individually. In this chapter, the different types of materials and mixture proportions that are most commonly used in CRCP construction are discussed first. Included are discussions on cement type, water-to-cementitious ratio, aggregate type, chemical and mineral admixtures, steel reinforcement, and sub-base material. Each is evaluated to determine its influence on pavement performance and durability.

Certain design guidelines have been established to ensure quality pavements. Most CRCP construction in the U.S. takes place in Texas and Illinois. Due to the amount of construction, Illinois and Texas use modified versions of the AASHTO provisions for slab-thickness design that reflect their individual design needs. The main design differences between Texas and Illinois include slab thickness, longitudinal steel amount, depth of steel, allowable crack width, and concrete design strength. This report summarizes the differences in each of these design variables. General design variables that relate to CRCP performance are also discussed.

CRCP is often affected by various distresses, resulting in premature repair and rehabilitation. This report discusses some of the most common distresses that occur in CRCP. In previous decades, many of these distresses were directly related to design inadequacies, but often they did and still do occur as a result of poor construction materials or methods. The main distresses that affect CRCP performance are spalling, edge punchout, and widened transverse cracking. It is important to understand the development and prevention of these distresses to allow CRCP to reach its full potential.

The methods currently used for constructing and placing CRCP are not intended to include fibers. Therefore, this report evaluates the most common types of paving equipment to determine whether fibers would affect the paving operation. The main types of paving equipment include slipform machines, self-propelled form-riding machines, and concrete spreaders.

There are several different types of accelerated pavement testing (APT) facilities that provide the capability to test fibers in CRCP. This report discusses some of the most common types of APT, which include test roads, circular test tracks, linear tracks, and other similar testing configurations. The main variations among each of the configurations pertain to loading, pavement configurations and materials, instrumentation, data collection, and analysis procedures.

The next section of this report focuses on fiber-reinforced concrete. Fiber-reinforced concrete is widely used in various concrete applications including industrial floor slabs, shotcrete, jointed concrete pavements, and thin bonded overlays. It is generally understood that fibers are effective in reducing plastic shrinkage and cracking, and in increasing toughness. Depending on the fiber type and dosage, additional benefits may also be obtained. This report focuses on two different types of fibers: steel and synthetic. Each has certain advantages and disadvantages that dictate its applications. This report also discusses the effects that fibers have on a given mix design, as well as the fresh and hardened concrete properties. There are several test methods used to quantify the effects of fibers. The predominant test method in the U.S. is American Society for Testing and Materials (ASTM) C 1018. It is used to measure the amount of toughness that fibers provide concrete.

There are several aspects of CRCP performance that may be improved by fibers. These include a reduction in crack width, an increase in toughness, and a reduction in plastic shrinkage. There are also some challenges that fibers present to CRCP, such as constructability, ride ability, and an increase in cost. It is difficult to implement fibers into the specifications and design standards due to the inability to quantify their effects. There have been several studies conducted with steel fibers used in thin bonded concrete overlays that have shown them to be excellent supplements to concrete pavements. This report concludes with a summary of key issues and recommendations for future research.

2.1 Continuously Reinforced Concrete Pavement

Continuously reinforced concrete pavement (CRCP) is a portland cement concrete that contains no expansion or contraction joints. Inducing cracks at closely spaced intervals that accommodate the volumetric changes eliminates the joints. The concrete strength, amount of steel, and environmental conditions control crack spacing. The main benefits of CRCP are an increase in ride quality, minimal maintenance, and a longer service life (Gagnon et al. 1998).

CRCP has been in existence since 1921 with thousands of miles of pavement completed throughout the U.S. It has been heavily constructed since the early 1960s during the vast expansion of the U.S. Interstate System construction program. There are currently more than 28,000 miles of CRCP that have been paved in the U.S. alone. Most of the pavements have provided a service life in excess of 30 years without the need for major rehabilitation. When designed and constructed properly, CRCP can truly remain maintenance-free throughout its service life. However, there are cases where the pavement has been under-designed or constructed with poor materials, resulting in significant pavement distress and premature failure. There has been considerable research and development of CRCP design and behavior. Design methods were initially based on incomplete data and did not address many variables associated with pavement distresses such as aggregate type. Currently, there is a much better understanding of CRCP behavior and performance that has led to a more sophisticated design procedure.

2.2 Materials and Mixture Proportions

The performance of continuously reinforced concrete pavement has been studied and researched for many years. It has been well documented that material selection has great influence over performance and durability of the pavement. As a result, there are guidelines that restrict the use of certain materials to ensure good performance. This section discusses some limitations of construction materials used in CRCP. These materials include cement type, water-to-cementitious ratio, aggregate type, the use of chemical and mineral admixtures, steel reinforcement, and sub-base material.

2.2.1 Cement Type

There are several different types of cement that can be used in CRCP. The initial time, material cost, and durability issues generally govern the type of cement selected. According to Item 421 of the Standard Specifications, the State of Texas allows the use of Type I, IP, II, or III cements for concrete pavements. Type I cement is normally chosen due to its lower cost. The downfall for this cement type is that it has no special properties for early age strength or resistance to durability issues. Type IP cement is a blend of Type I cement and a supplementary cementing material, usually fly ash. It may be used to lower water demand, reduce permeability, increase long-term strength, and reduce costs. Type II cement is used in applications where additional resistance to sulfate attack is desired; it also helps moderate the heat of hydration. Type III cement is generally used in fast-track paving applications due to its high early strength. Typical dosages for cement in the state of Texas are in the range between 5 and up to 6 sacks per cubic yard (Texas Department of Transportation 1994).

2.2.2 Water-to-Cementitious Ratio

The water-to-cementitious-materials ratio (w/cm) varies depending on the amount of admixtures used, the type and size of aggregate, and the desired air content. These values of w/cm are selected with the idea of achieving a 1- to 3-in. slump (TxDOT 1994) for slipforming placement and sufficient strength to ensure durability.

2.2.3 Aggregate Type

There are a variety of aggregates that can be used in concrete pavement applications, including CRCP. These vary according to porosity, gradation, shape and surface texture, bond, elastic modulus, and coefficient of thermal expansion (CTE). The aggregate type and size has a considerable effect on the concrete strength, CTE, and amount of shrinkage (Mehta and Monteiro 1993). According to Item 421 of the Standard Specifications, the nominal size used in Texas conforms to a grade 2 or 3, which has a maximum size of 1.5 inches. The two main aggregate sources for Texas are crushed limestone and siliceous river gravel. Each of these aggregates has distinct characteristics that dictate the performance of the concrete pavement. Field performance has shown that pavements constructed with crushed limestone generally perform better than those constructed with siliceous river gravel (Dossey and McCullough 1999).

2.2.4 Chemical Admixtures

There are a number of chemical admixtures that can improve the performance of concrete pavements. These include air-entraining, water-reducing, accelerating, and retarding admixtures. Air-entraining admixtures are extremely effective in providing resistance to freeze-thaw damage. The volume of entrained air that is needed for good durability is dependent on the severity of the environment and the concrete’s maximum coarse aggregate size. Typical amounts of air-entraining agents are 4.5 to 7.5 percent. It is important to find an optimal dosage that does not reduce the early- and long-term strength of the concrete (American Concrete Pavement Association 1994).

Water-reducing admixtures are often used to reduce the water-to-cementitious ratio and increase the workability of the concrete mix. In addition, they can be used to increase early strength in fast-track concrete paving applications. Popovics (1979) explains that this early strength boost is the result of a lower number of cement particle agglomerations and an increased dispersion of the cement particles. Depending on the type of water reducer, the dosage rates will vary from 3 to 18 fl. oz./cwt. Mid-range water reducers are usually applied in the range of 3 to 15 fl. oz./cwt, and high-range water reducers are applied in the range of 4 to 18 fl. oz./cwt.

Accelerating admixtures are often used to aid in strength development and to reduce initial set times. Accelerators become very important in cold-weather construction applications; as they allow for earlier finishing, which helps to lower construction costs. Conversely, retarding admixtures are primarily used in hot-weather construction applications to allow laborers sufficient time to finish the concrete when finishing it may not otherwise have been practical (ACI Committee 325 1997).

2.2.5 Supplementary Cementing Materials

Supplementary cementing materials (SCMs), such as fly ash and ground-granulated blast-furnace slag, provide many benefits for concrete pavement applications in addition to reducing costs. SCMs react with the chemical products of portland cement during cement hydration to increase long-term strength gain and durability (Kosmatka and Panarese 1988). The addition of fly ash helps lower water demand, improves workability, reduces permeability, and increases long-term strength. Fly ash is generally added in dosages of 20 to 35 percent as a substitute for cement. Slag can also help to increase the long-term strength and improve the finishability of concrete. Typical amounts of slag are 30 to 50 percent.

2.2.6 Reinforcing Steel

The basic design premise for reinforcement varies slightly between continuous and jointed pavements. The amount of stress that is allowed to form in the reinforcing steel is equivalent to 75 percent of the steel yield strength. The reinforcement may be either reinforcing bars or deformed wire fabric. Reinforcing bars are generally selected due to their enhanced performance and ease of construction.

The primary reinforcement in CRCP is longitudinal steel that extends continuously throughout the pavement. Longitudinal reinforcing steel bars serve to minimize transverse cracks widths. The tighter the transverse crack widths, the better the load transfer is between adjacent slabs. Creating tighter transverse crack widths is accomplished by increasing the percentage of longitudinal reinforcing steel. This reduces the spacing between transverse cracks which in turn decreases the transverse crack widths. This creates better load transfer between adjacent slabs and reduces the amount of dirt and salt that can penetrate down through the crack. Its principal purpose is to develop a large number of transverse cracks as a result of environmental loading and hold them tightly closed, as opposed to having a small number of cracks with large crack widths. As volumetric changes occur within the concrete, the longitudinal steel must resist the restraint that is created between the concrete slab and the sub-base material. A balance between the properties of the concrete, sub-base, and reinforcing steel must be achieved to enable the pavement to perform satisfactorily. The main parameters for reinforcing steel are percentage of steel reinforcement, bar diameter, yield stress, modulus of elasticity, and thermal coefficient (Hudson et al. 1988). The size of the reinforcing bar varies with each application. Since it also influences crack development in CRCP. For a given steel percentage, the pavement with a smaller bar size gives a larger steel surface area for the concrete to bond. Therefore, a larger number of small reinforcing bars may carry the same amount of stress as a smaller number of large bars but have a greater surface area to distribute that stress to the concrete. Thus, the transfer of stress between the steel and concrete is enhanced for smaller bars, resulting in shorter crack spacing and smaller crack widths. No. 5 and No. 6 bars are most commonly used. In order to ensure a sufficient bond between the concrete and reinforcing steel, it is recommended that the bar size not exceed that of a No. 6 bar. The main consideration in selecting deformed wire fabric is that the wire diameter be large enough that corrosion will not significantly reduce the cross section diameter (AASHTO 1993).

The steel percentage has a significant effect on crack spacing. The optimum percentage of longitudinal reinforcement produces stress-relieving cracks that are held tightly together to prevent water penetration from the surface to the sub-base. The steel is used to restrain cracks from opening as volumetric changes in the pavement occur. This enables the concrete to maintain sufficient aggregate interlock, load transfer, and stiffness at cracks.

The main controlling factor for steel percentage is crack width. An increase in steel percentage is directly related to a decrease in crack spacing, crack width, and steel stress. It is the restraint of the concrete from steel reinforcement and sub-base friction that ultimately causes the concrete to crack. Therefore, a proper balance between the concrete, steel, and sub-base must be attained that produces satisfactory results (Dossey and McCullough 1999). The percentage of longitudinal steel varies among state highway agencies but is typically in the range of 0.5 to 0.8 percent.

Transverse reinforcement is intended to control the width of any longitudinal cracks that may form. It mainly serves to restrain the lateral movement and prevent longitudinal cracks from opening excessively, thus maintaining load transfer and minimizing water entry. The size and type of reinforcing bars is similar to longitudinal reinforcement, and transverse reinforcement is generally placed below and squared with the longitudinal bars. Some countries have experimented with placing the bars obliquely to the longitudinal bars in a skewed orientation. This decreases the chance of a weakened plane forming in the concrete that may form an undesired crack. However, studies in the U.S. have found minimal success with this technique (Dossey and McCullough 1999). Transverse reinforcement may be excluded from the pavement design if past experience indicates longitudinal cracks will not form (AASHTO 1993).

2.2.7 Sub-base Material

The sub-base is the layer of material located between the subgrade and the surface of the pavement. Several different materials are commonly used for sub-base, such as graded granular materials, lean concrete porous layers, and materials stabilized with suitable admixtures. The main function of the sub-base is to provide a uniform, stable, and permanent support, which helps prevent subgrade erosion and pumping (AASHTO 1993). The TxDOT Pavement Design Manual (2001) describes several different materials that are considered acceptable for rigid pavement applications. Asphalt concrete pavement or an asphalt-stabilized sub-base is generally used in a layer of 4 inches. A 1-in. asphalt concrete bond breaker may also be applied on top of 6 in. of a cement-stabilized sub-base.

The vertical location of the longitudinal steel has an effect on the crack pattern. There has been some debate among state highway agencies as to the optimum location for steel placement. Several states have chosen to place the steel near the surface of the pavement where volumetric strains are greatest to reduce the surface width of cracks. This method succeeds in restraining induced movements, resulting in an increase in the number of transverse cracks. However, the downfall of this approach is that it tends to develop an irregular cracking pattern. The other widely accepted steel placement (the one adopted by Texas) is at the mid-depth level of the pavement. This level was chosen because it limits the total vertical movement from wheel loads and the localized steel stress at cracks due to temperature differential and wheel loads (CRSI 2001).

A related topic to the depth of steel placement is the utilization of two-layer placements. Texas has adopted this method for pavements 13 in. or more in order to maintain sufficient spacing between the reinforcing steel to allow larger aggregates to pass. Using two layers gives a reduced depth of cover and results in a greater degree of volumetric restraint. Two layers of transverse steel are also required to balance the configuration.

The existing AASHTO design procedure for slab thickness is based on the interaction of several design variables, which are represented in a design nomograph. There is some flexibility within this design format, which includes crack width, crack spacing, steel stress, and depth of steel. These items can have significant influence over the selected level of pavement thickness. Depending on the pavement application, Illinois generally designs a slab thickness of 10 to 13 inches. Texas, however, has a wider range of slab thickness ranging from 8 to 15 inches. (CRSI 2001).

A downfall of this approach is that the increased steel reduces the concrete cross-section and thus may create a weakened plane for transverse cracking (Concrete Reinforcing Steel Institute [CRSI] 2001).

2.3 Other Design Concepts and Issues

There are more than 35 states in the U.S. using continuously reinforced concrete pavement (CRCP). Most CRCP construction in the U.S. takes place in Texas and Illinois, so these states will be discussed in the greatest detail. The adopted design procedure in the U.S. is the 1986/1993 AASHTO code. However, due to large amounts of construction, Illinois and Texas use modified versions of the AASHTO provisions, which reflect a slightly different design philosophy. These design requirements include slab thickness, longitudinal steel amount, depth of steel, allowable crack width, and concrete design strength. It is important to understand the basis of these different design strategies to have a better fundamental understanding of CRCP design and performance.

2.3.1 Allowable Crack Width

The maximum allowable transverse crack width is generally limited to increase load transfer efficiency. Tight cracks are also important in preventing corrosion in regions of the country where de-icing salts are used. Texas requires a maximum crack width of 0.025 in., whereas all other states recommend a maximum of 0.04 in. (CRSI 2001).

2.3.2 Slab Thickness

The procedure for thickness design has evolved throughout the history of CRCP. Originally it was argued that CRCP need not be as thick as jointed reinforced concrete pavement (JRCP) to provide the same structural capacity. Therefore, early pavement designs incorporated CRCP thicknesses that were 70 to 80 percent that of JRCP. This was warranted by the structural continuity of the pavement and the localization of the maximum stresses along transverse cracks and not along the free edge of the slab. It was also justified with stress calculations and field experiments. Many countries, notably France and Spain, still use a reduction for thickness in CRCP designs. However, the 1981 revision to the AASHTO Interim Guide for Design of Pavement Structures recommended that CRCP have the same thickness as JRCP unless local experience has shown that thinner slabs perform satisfactorily. It is difficult to conclude which view is correct because so many factors affect slab performance (Permanent International Association of Road Congresses [IARC] 1994).

The thickness of the slab is a function of the sub-base, traffic loading, steel reinforcement percentage, and composition of concrete. The main concern in thickness design is to develop a crack pattern that does not result in premature failures. Also, present design methods determine how much reinforcing steel is required to achieve desired crack spacing for a given set of temperature conditions. A larger slab thickness also increases resistance to critical bending stresses and load transfer, resulting in fewer punch outs and a smoother pavement. In addition, Buch et al. (1999) have stated that although the current design methodologies for slab thickness address the issue of crack development, they do not adequately embrace crack width requirements. They further recommend that a better relationship should be provided that links pavement thickness, load transfer, crack width, and the percentage of reinforcement for a given crack spacing.

2.3.3 Aggregate Type

One of the most important factors in crack development is aggregate type. Although many design and construction techniques have attempted to compensate the effects of aggregate, there has been little success. Pavements constructed with a siliceous river gravel perform significantly worse than those with crushed limestone. The difference in performance is directly related to the thermal coefficient value.

There have been several attempts to minimize the effects of a high thermal coefficient. These include controlling the season during which concrete is placed, blending the mix with a less expansive aggregate, increasing the amount of longitudinal reinforcement, and inducing cracks by the use of saw cuts. The most successful adjustment for improving crack spacing has proven to be placing the concrete during winter months, although discussions with TxDOT suggest that this may increase the chances of spalling. An increase in steel percentage also has been shown to enhance the pavement’s performance. Blending aggregates have performed well in some cases but were disappointing in others. Lab studies have demonstrated that the concrete properties are proportional to the composition of aggregates. However, field results have not always supported this, lending doubt as to the validity of the lab tests (Dossey and McCullough 1999).

The aggregates available in each state generally govern the type of aggregate placed in the pavement. Texas predominantly uses crushed limestone and siliceous river gravel aggregate with sizes ranging from 0.75 to 1.5 inches. Crushed limestone generally performs better in field conditions than siliceous river gravel. Illinois has traditionally used gravel, crushed gravel, stone, concrete slag, or sandstone aggregate with a maximum size of 1.5 in., although crushed limestone is currently used due to a shortage of quality gravel and potential D-cracking problems.

2.3.4 Sub-base

The sub-base is the layer of material located between the subgrade and the bottom of the concrete layer. Several different materials are commonly used for sub-base, such as graded granular materials, lean concrete porous layers, and materials stabilized with suitable admixtures. The main function of the sub-base is to provide a uniform, stable, and permanent support. This prevents subgrade erosion and pumping, which lead to more detrimental distresses (AASHTO 1993).

Sub-bases are also used to increase the modulus of subgrade reaction, to minimize the damaging effects of frost action, and to provide a working platform during construction. The type of sub-base is characterized by its ability to prevent loss of support (LS). Ranges for LS vary from 0.0 to 3.0, depending on the type of sub-base selected (AASHTO 1993). A sub-base must also be selected that minimizes the friction with the base of the concrete pavement to prevent excessive resistance during environmental loading.

2.3.5 Subgrade

The amount of subgrade support may influence the required pavement thickness. It is defined by the modulus of subgrade reaction, k. AASHTO (1993) provides several design tables that assist in the development of an effective modulus of subgrade reaction.

2.4 Performance Issues

Continuously reinforced concrete pavements have often been affected by various distresses, resulting in premature repair and rehabilitation. This section describes some of the most frequent distresses pertaining to CRCP performance. These distresses are typically a result of poor construction materials or methods. Most generally occur within 3 to 10 years of construction, and they negate the main benefits of CRCP because it is intended to remain maintenance-free throughout its service life. Therefore, it is important to understand the development and prevention of the various distresses to allow CRCP to reach its full potential.

2.4.1 Spalling

Spalling is the most detrimental distress associated with CRCP made with siliceous aggregates within the state of Texas. The spalling is characterized by surface pop outs of a slab immediately adjacent to either side of a transverse crack. In the longitudinal direction, the spall is 2 to 6 inches away from the transverse crack. The deepest part of a spall will be ½ to 1 ½ inches. Even though the spalling causes a rough ride, the functionality of the pavement is not reduced.

A spall may continue to grow in the longitudinal direction as the summation of traffic increases. It is not clear if the additional longitudinal spall area, sometimes another 6 more inches, prorogates at a time soon after construction, or at a much later date.

Field performance in the state of Texas has shown that pavements constructed with crushed limestone generally perform much better with respect to spalling than those constructed with siliceous river gravel (Dossey and McCullough 1999). It has also shown that pavements constructed in the winter exhibit the most severe cases of spalling. There are currently several theories that are being considered at TxDOT to explain this phenomenon. It is believed that winter placement induces cracks that remain in the upper portion of the slab as a result of the lower temperature gradient. As the temperature later increases, the crack propagates deeper into the slab. The way in which the crack propagates is dependent on the type of aggregate. For pavements constructed with river gravel, the crack has a tendency to travel around the aggregate because of the aggregate's weak bond to the cement. Several cracks may form to accommodate rapid thermal expansion at the top of the slab. This may also result in horizontal cracking at the mid-depth and is believed to induce the spalling mechanism. For pavements constructed with limestone, the bond between the aggregate and the cement is very strong and encourages the crack to propagate directly through the aggregate. This generally results in a single crack that extends the entire depth of the slab and generally does not cause distress. Concrete placed in the summer does not exhibit such severe spalling. It generally has larger initial temperature gradients that induce nice smooth cracks extending the entire depth of the slab. River gravels tend to exhibit higher elastic modulus, higher CTE, and weaker bond than crushed limestone. Exactly which of these parameters most affects spalling is still a topic of debate.

Slight spalling, which pertains to the mortar in the concrete matrix, is considered merely a cosmetic problem. However, severe spalling generally leads to structural distress and thus requires maintenance and repair. It generally forms at a greater depth before it begins to widen, enabling more damage to be done. Spalling may also be related to the corrosion of reinforcing steel with less than 3 in. of concrete cover. However, this phenomenon most often occurs in northern states, which apply chemical de-icers, and is therefore not a problem associated with Texas (Gagnon, Tayabji, and Zollinger 1998). Figure 2.1 illustrates the damage that may result from spalling.

Figure 2.1: Severe Spalling (Won 2001)

2.4.2 Punchouts

Edge punchout is defined as a pavement section enclosed by two closely spaced transverse cracks, a short, intersecting longitudinal crack, and a free pavement edge. Punchouts have contributed to a substantial number of CRCP rehabilitations and are very expensive to repair. Punchouts were once considered to be the most detrimental distress associated with CRCP. However, sub-base improvements and tying of the pavement to the shoulder have significantly reduced the damage caused by punchouts.

The punchout process generally begins with the loss of pavement support due to moisture accumulation, pumping, and erosion. This initiates the loss of load transfer at closely spaced transverse cracks. The pavement section, formed by closely spaced transverse cracks and pavement edge, then begins to act as a cantilevered beam. This provokes a longitudinal crack to form approximately 2 to 5 ft from the pavement edge. Ultimately, the section punches downward into the sub-base and subgrade material, usually causing the steel to rupture. The result presents a serious hazard to motorists and has the potential to expand to neighboring cracks if not repaired (Huang 1993). Figure 2.2 illustrates the damage that a punchout may cause. It should be noted that punchouts are rarely observed in CRCP recently constructed in Texas, due to extending the CRCP edges into the shoulder, punchouts have been eliminated, even when close crack spacing is present.

Figure 2.2: Edge Punchout (Miller 1993)

2.4.3 Widened Transverse Cracks

Transverse cracks primarily develop perpendicular to the pavement centerline. The basic premise of CRCP is the formation of a controlled transverse crack spacing that eliminates the need for transverse joints. Therefore, transverse cracks are inevitable and must be properly planned to ensure high-quality performance.

Widened transverse cracks usually form from partially corroded reinforcing steel that has a reduced cross-sectional area. This results in a localized loss of tensile capacity and may lead to yielding or rupturing of the reinforcing steel, allowing the crack to open excessively. This may induce other pavement distresses such as faulting, spalling, and edge punchout. Widened transverse cracks may also develop from an inadequate lap of reinforcing steel, resulting in a lack of continuity. Ensuring that cracks are properly spaced most effectively prevents transverse cracks. Transverse cracks with greater spacing generally develop a wider crack opening and are thus more susceptible to corrosion (Gagnon, Tayabji, and Zollinger 1998). Figure 2.3 shows the type of transverse cracking that can occur in CRCP.

Figure 2.3: Transverse Cracking (Miller 1993)

2.4.4 Longitudinal Cracks

Longitudinal cracks are cracks that extend parallel to the pavement centerline. Unlike transverse cracks, longitudinal cracks are not expected or desired in CRCP. They are generally caused by a combination of foundation instabilities, heavy load repetition, and inadequate construction joints (Gagnon, Tayabji, and Zollinger 1998).

According to Gagnon, Tayabji, and Zollinger (1998), there are two types of longitudinal cracking generally associated with CRCP. The first type is classified as a wandering uncontrolled cracking. It usually develops within 3 ft on either side of the centerline joint or lane joint and is caused by late or inadequate sawing of weakened-plane longitudinal contraction joints. The effects of this type of crack are generally cosmetic and do not pose a threat to the integrity of the pavement.

The second type of longitudinal cracking is related to foundation settlement. The effects of this type of cracking are much more severe. It is initiated at the localized areas of foundation settlement where concentrated stresses form that may exceed the flexural strength of the concrete. As a result, a crack may materialize that allows water penetration and accelerates crack deterioration. Figure 2.4 shows a typical longitudinal crack that can occur in CRCP. The uses of stabilized base and proper saw-cutting practices have helped to reduce longitudinal cracking in Texas.

Figure 2.4: Longitudinal Cracking (Miller 1993)

2.4.5 Crack Width

Crack width can have a commanding effect on the behavior and performance of CRCP. The basic premise of CRCP is that many hairline cracks will develop that must be held closely together by sufficient longitudinal reinforcement. Excessive crack width can lead to the loss of load transfer, the penetration of incompressible material, and water infiltration. It can also progress to punchouts, spalling, blowups, and reduced foundation support. Therefore, it is important to generate a narrow crack width that provides sufficient aggregate interlock and impedes the infiltration of water. Design guides generally recommend that crack widths not exceed 0.04 in. (AASHTO 1993). The main factors that affect the development of crack widths include the placement season, percentage and depth of reinforcing steel, and coarse aggregate type (Dossey and McCullough 1999).

2.4.6 Crack Spacing

The main parameters associated with crack spacing are devised from consideration of localized failures such as punchouts and spalling. Most of transverse cracking that occurs in CRCP is a function of the percentage of longitudinal reinforcement, concrete strength, traffic load application, and slab/base interface friction. It is also strongly influenced by environmentally induced strains such as drying shrinkage and thermal strains during the first few days after placement.

In CRCP, the crack spacing controls the slab behavior. Narrow crack spacing produces critical concrete stresses in the transverse direction. This increases the probability of longitudinal cracking, resulting in punchouts. Conversely, crack spacing greater than 3.5 ft causes the slab to act as a longitudinal beam, significantly reducing the chances of longitudinal cracking. However, if the crack spacing becomes too great, the transverse crack width must increase to accommodate the equivalent expansion, often resulting in spalling (Hudson et al. 1988). Therefore, design guides generally recommend a crack spacing of 3 to 8 ft to minimize localized failures. CRCP containing river gravel tends to result in closer crack spacing than CRCP containing crushed limestone.

2.4.7 Coefficient of Thermal Expansion

The coefficient of thermal expansion (CTE) for concrete is directly related to the volumetric change that hardened concrete experiences due to temperature change. Therefore, CTE has a great influence on the crack development in CRCP. Since approximately 40 percent of concrete pavement's volume is composed of coarse aggregate, the coarse aggregate type is the main contributor to differences in CTE. Extensive research has revealed that the thermal coefficient value is directly related to the silica content in the aggregate. Some river gravel has a CTE approximately 60 percent higher than that of a crushed limestone. However, the effect of variation in coarse aggregate types on CRCP performance has not always been incorporated into the design-construction process (Allison et al. 1993).

The main influence from CTE is the amount of opening and closing of the transverse cracks. This contributes to the stress in the reinforcing steel and loss of load transfer, thus leading to severe distress (Gagnon, Tayabji, and Zollinger 1998). Most CRCP constructed in the state of Texas is composed of siliceous river gravel or crushed limestone. However, many have been designed without regard for aggregate type. Therefore, the pavements that use river gravel often result in premature failure.

2.5 Construction and Paving

The methods for constructing and placing concrete pavement are constantly changing with the development of new materials and paving technology. There are several different types of paving equipment that are used for CRCP construction. These include slipform machines, self-propelled form-riding machines, and concrete spreaders. The subgrade and sub-base preparation has also progressed to produce better-performing pavements.

2.5.1 Paving Equipment

Slipform machines are capable of spreading, consolidating, screeding, and finishing freshly placed concrete in one pass to provide a well-consolidated, homogenous pavement that requires minimal hand finishing that meet surface tolerances. Slipform machines are equipped with automatic controls to control the line and grade from either or both sides of the machine. They also contain vibrators that consolidate the concrete for the full width and depth of the pavement being placed. "Gang-mounted spud" type internal vibrators usually supply vibration. Slipform pavers should be operated with a nearly continuous forward movement that mixes, delivers, and spreads concrete with uniform progress. They should also have a constant, uniform amount of concrete ahead of the strikeoff device to submerge internal vibrators and equalize the depth of concrete placed by the spreader. The slump should be maintained to an absolute minimum when using slipform pavers (ACI 325 1997). A typical slipform machine is shown in Figure 2.5.

Figure 2.5: Slipform Paving Machine (Leica Geosystems 2002)

Self-propelled form-riding machines are capable of consolidating and finishing the concrete with minimal hand finishing. They come equipped with immersed tube or multiple spud vibrators. The vibrators are attached to the spreader, the finishing machine, or a separate carriage that guides the finishing machine. One important consideration when using this type of machine is ensuring that it does not displace the fixed side forms (ACI 325 1997).

Concrete spreaders may be used on smaller jobs to reduce the cost of placement. A typical concrete spreader is shown in Figure 2.6. The concrete is placed using a conveyor system that uniformly spreads the concrete across the section. The concrete is then manually vibrated to ensure proper depth for consolidation and finishing. A roller screed is typically used to strike off excess concrete. A bull float or a trowel may then be used to finish the surface of the concrete pavement (ACI 325 1997).

Figure 2.6: Concrete Spreader

The pavement surface should contain both fine and coarse textures. The sand in the cement-mortar layer produces the fine texture, and the ridges of mortar left by texturing create the coarse texture. The method of texturing selected should be compatible with the environment, speed, traffic volume, and pavement topography. Skid resistance is generally provided to concrete pavements by burlap drag, brooming, wire combs, rug backing, or plastic combs used to tine the pavement (ACI 325 1997).

2.5.2 Roller Screeding

A typical roller screed performed the screeding on this project. A typical roller screed is capable of screeding across a single lane of pavement, is self-propelled, and rides on the side-forms. Specifically, a roller screed has three round aluminum tubes that reach across a paving lane, and rides trackless on top of the forms. The first tube screeds the concrete by spinning in the opposite direction of the paving. The second and third tubes spin in tandem, as controlled by the operator, propel the machine forwards, stopped or backwards. The roller screed moves backwards to re-screed an area, and turn in the direction of the machines movement. Handheld vibrators are used near the screed to consolidate the concrete. A typical roller screeder is shown in Figure 2.7. Unlike a slip form paver, the roller screed does not need to maintain a constant forward speed.

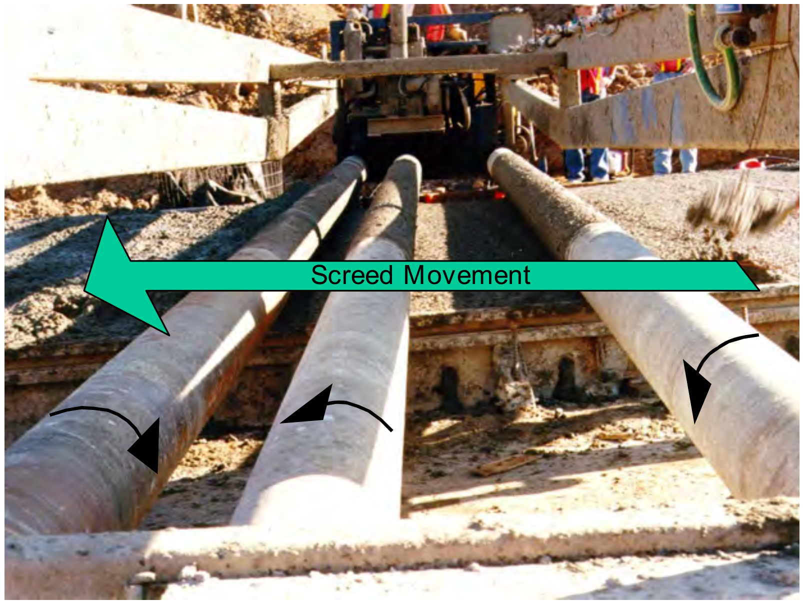


Figure 2.7: Roller-Screeding

2.5.3 Subgrade/Sub-base

It is important that the quality of a subgrade be adequate for a given pavement. A sub-base is generally required when heavy traffic or serious frost problems are expected. There are several pavement characteristics that should be considered when preparing a sub-base. The main function of the sub-base is to provide a uniform, stable, and permanent support for the pavement in order to prevent subgrade erosion and pumping. Most CRCP is placed on a treated sub-base unless a suitable sub-base is present. It is not permitted for the concrete to be placed on a frozen subgrade or sub-base. It is also important to make certain preparations to the subgrade before the concrete is placed. These include fine grading, adjusting the surface of the subgrade or sub-base, adding moisture, recompacting any disturbed material, and preparing the final finished surface to match the specified grade and cross-section (ACI 325 1997). A sub-base that has good stability will enhance the pavement smoothness. It is also recommended that the sub-base be 2 ft wider than the pavement lane when constructed with a slipform paver to accommodate the slipform tracks.

The sub-base for these Houston District Construction Projects for the trial sections evaluated in this research consisted of 5 inches of asphalt base as per TxDOT Standard Specifications Item Number 345 in the 1993 Book of Texas Standard Specifications.

2.5.4 Placing Reinforcement

The reinforcement is placed on supports called *chairs* before the concrete is placed. The reinforcement should have a cover of 2 inches. This is typically not an issue within the state of Texas because the design philosophy at this time places the reinforcement at mid height. Specifications also state that the reinforcement should not fall below the mid-depth of the pavement. It is important that the steel be placed on supports capable of maintaining the steel in its specified position while the concrete is placed. It is also important to provide an adequate splice between connected longitudinal reinforcement to prevent failure of the splices at early ages. Splices should be in a skewed or staggered pattern, and the splice length should be longer than 30 bar diameters and also longer than 16 inches.

2.5.5 Curing Methods

There are several different methods that can be used to cover and cure concrete pavement. Regardless of the curing method selected, the curing should be applied immediately after the finishing operations have been completed and the bleed water leaves the surface. The most widely used curing methods are membrane curing, cotton mats or burlap, waterproof paper, and white polyethylene sheeting. These construction projects used membrane curing according to TxDOT Standard Specifications.

Membrane curing requires that a liquid membrane be applied with a mechanical spray machine at a rate of at least 1 gal per 150 ft^2 of surface immediately after the water film has left the pavement surface. This method is illustrated in Figure 2.8. It is important to provide uniform consistency and dispersion of the curing material. This is accomplished by agitating the liquid in the supply container immediately before and during application. The sides of the pavement should be coated within 1 hour of the removal of forms. Monomolecular coatings are generally applied in undesirable drying construction conditions to retard surface evaporation.

Figure 2.8: Curing Application

2.5.6 Chemical Admixtures

Various materials are available to improve the performance and constructability of concrete pavements. These include chemical admixtures, supplementary cementitious materials, and curing materials. Accelerated admixtures are added to concrete to reduce the time of setting and accelerate the early-strength development. Air-entraining admixtures are added to improve both the durability and workability of the concrete. They are especially important in areas where freeze-thaw is a concern. Water-reducing admixtures are added to reduce the total water content and water-to-cementitious-materials ratio. This enables the concrete to improve its compressive strength, flexural strength, and durability, while decreasing its permeability, shrinkage, and creep. Retarding admixtures may also be used to reduce the setting of concrete when rapid setting is not desired (ACI 325 1997).

2.5.7 Sampling and Testing of Materials

A quality control program should be in effect to ensure that the concrete meets the requirements of the specifications. The contractor, concrete producer, and supplier are generally responsible for the program. The type and amount of material sampling depends on the project and governing agency. The majority of states generally monitor compressive and flexural strength of concrete. Fresh properties are also often monitored to verify the quality of the concrete as it arrives at the construction site (ACI 325 1997). A typical quality control program is shown in Figure 2.9.

Figure 2.9: Quality Control Program

2.5.8 Opening to Public Traffic

The main factor that indicates when the pavement can be opened is the concrete strength and not the time of placement (American Concrete Pavement Association [ACPA] 1994). Most states use flexural strength as the criterion for evaluating load capacity. This gives an accurate assessment of the tensile strength at the bottom of the pavement where tensile stresses are produced. The amount of load capacity required to open the pavement depends on several factors. These include the type of pavement, slab thickness, foundation support, edge support condition, location of loads on the slab, and the type, weight, and number of anticipated loads during the early stages of the concrete (ACPA 1999).

2.6 Service Life

Continuously reinforced concrete pavement (CRCP) has been heavily constructed in Texas for many years. The majority of these pavements have provided a service life longer than 30 years without the need for major rehabilitation. The TxDOT Pavement Design Manual (2001) currently requires a minimum performance life of 30 years. When designed and constructed properly, CRCP requires no maintenance whatsoever throughout its service life. However, there are many cases of CRCP distress and premature failure due to improper design or poor construction materials.

2.6.1 Failure Modes

There are two modes of failure that CRCP may experience. They are classified as functional and structural failures. A functional failure occurs when pavement is no longer able to provide acceptable ride quality, and a structural failure occurs when one or more of the pavement's structural components fail. Functional failure is generally characterized using the present serviceability index (PSI). A disadvantage of this index is that it does not always capture the distress of the pavement. As a result, a distress index is often used in addition to the PSI to assess the functionality of the pavement.

2.6.2 Performance Issues

The service life for CRCP is affected by several factors. One of the major factors is traffic load. It is difficult to predict the volume and axle weights of traffic that a pavement may experience. Predictions are usually made from highway department truck studies. The environment is also a factor for the service life. Temperature and moisture conditions produce non-uniform stress gradients as the pavement tries to restrain them. These conditions may also change the shape of the slab, diminishing the degree of subgrade support. The boundary conditions also affect the pavement's service life. These include subgrade friction, load transfer at cracks, and load position. The final factor relates to the support conditions. The subgrade may be affected by pumping, densification, and displacement of the subgrade (ACI 215 1992).

CRCP has an inherent capability for withstanding heavy loads while maintaining its durability and strength. Unlike jointed reinforced concrete pavements (JRCP), CRCP allows expansion through the opening and closing of tightly held cracks, thus eliminating the need for expansion and contraction joints. Joints are the main causes of distress in JRCP. CRCP has few weaknesses in regard to the wear and tear that a pavement must endure, thus giving it the longest service life of any pavement.

2.6.3 Fatigue Performance

Concrete pavements are constantly subjected to fatigue loadings caused by traffic and cyclic environmental conditions. Because the amount of vehicle loading is always increasing, the need for pavements to handle fatigue has become ever more apparent. Most high-volume pavements experience much larger daily traffic loads than were initially anticipated in the design. Although fatigue loading may eventually cause cracking, the pavement should remain serviceable if load transfer is maintained. Clemmer (1923) found that an induced flexural stress could be repeated indefinitely without causing rupture of the concrete if the stress did not exceed 50 percent of the modulus of rupture. This serves as the basis for the guidelines of concrete pavement design.

CRCP also provides an adequate foundation for repair and rehabilitation. Overlays are often added directly to the top of CRCP to extend service life. The absence of joints in CRCP generally reduces the amount of reflective cracking that is typically associated with overlays, thus increasing the effectiveness of pavement repair.

2.7 Fiber-reinforced Concrete

This section addresses the properties of fiber-reinforced concrete. Within the scope of this project, fibers were used simultaneously with conventional amounts of reinforcing steel.

Plain portland cement concrete is an inherently brittle material, with low tensile strength and strain capacity. While the traditional means of overcoming these inherent flaws has been to add steel reinforcing bars at specified locations in the matrix, during the past century there have been developments to use randomly oriented, discrete fibers to remedy these weaknesses. This is known as fiber-reinforced concrete. It is important to recognize that the purpose of both traditional reinforced concrete and fiber-reinforced concrete is to improve the behavior of plain concrete, but the applications for each are very different. It was initially believed that fiber-reinforced concrete could be used to improve the static properties of plain concrete (compressive strength, tensile strength, flexural strength) in the manner of traditional reinforcing bars. Therefore it could be used as a replacement for steel reinforcing bars. Despite these initial hopes, it has been well documented through research that fiber-reinforced concrete offers insignificant strength gains and cannot be used in the same manner reinforcing bars are used (Mindess 1995).

What fiber reinforcement does offer is the ability to control crack widths and carry significant stresses after the initial cracking of the concrete. This improvement in post-cracking behavior gives the concrete a pseudo-ductility that is known as toughness. Fiber reinforcement also provides plain concrete with improved dynamic properties such as fatigue strength and behavior under impact loading (Mindess 1995).

Currently, fiber-reinforced concrete has been limited to nonstructural applications because of its inability to relate toughness to any parameters typically used in structural design. These applications include slabs, pavements, bridge decks, thin bonded overlays, shotcrete, etc. These are all applications in which dynamic performance is critical, which is why the introduction of fiber reinforcement can be easily justified.

The understanding of fiber-reinforced concrete continues to evolve and improve each year. The current state-of-the-art of various fiber-reinforced concrete topics will be investigated. These topics include fiber types, effects on mix designs, effects on fresh and hardened concrete properties, test methods, and applications.

2.8 Fiber Types

Fiber reinforcement has been implemented in various forms of construction for thousands of years. Its roots can actually be traced back to Roman times, when straw was used as reinforcement for sun-dried clay bricks. This early form of fiber reinforcement was used to improve ductility in the same manner as modern fiber reinforcement. Despite these basic similarities, a significant amount of research has been conducted to improve the performance of modern fiber reinforcement.

There have been a variety of materials developed as potential fiber reinforcement over the past century. These materials include natural fibers (straw, horse hair), asbestos fibers, glass fibers, carbon fibers, steel fibers, and synthetic fibers (nylon, polypropylene, polyethylene). Each of these materials has had varied success. This report will focus on steel and synthetic fiber reinforcement, because these are the materials that have become the standard in FRC construction. Various aspects of these two materials will be covered, including production, fiber geometry, mechanical properties, typical dosage rate, and applications.

2.8.1 Steel Fiber Reinforcement

Steel fiber reinforcement has gone through many changes throughout its history before becoming what it is today. Typically, modern steel fiber reinforcement is produced from high tensile strength steel (greater than 130 ksi). It is generally a cold-drawn wire that is deformed to a desired shape and then cut. While various deformation geometries have been developed to produce better anchorage of the fiber in the matrix, the two most common geometries used today are corrugated fibers that are sinusoidal in shape and hooked-end fibers. Corrugated fibers are generally a manufacturing byproduct and therefore have lower quality control than hooked-end fibers, which are produced solely for the purpose of fiber-reinforced concrete. Hooked-end fibers are also typically collated into small bundles with an adhesive that breaks down during mixing and allows the fibers to distribute throughout the matrix. Other steel fiber types that have been developed include straight, machined chip, and melt extract steel fiber reinforcement. These steel fiber types have not had much success (ACI Committee 544 1997). Steel fiber geometries are shown in Figure 2.10.

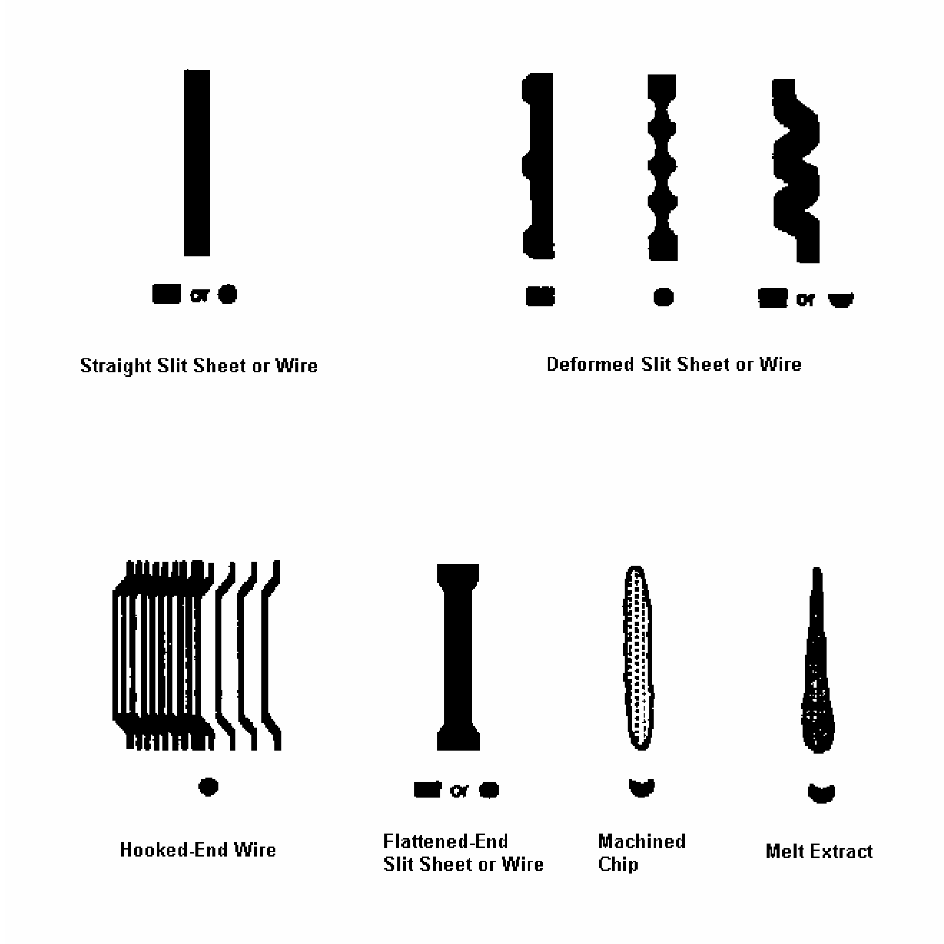


Figure 2.10: Steel Fiber Geometries (ACI Committee 544 1997)

Properties that are important in determining steel fiber performance are strength, stiffness, and aspect ratio. Strength and stiffness ensure proper stress development and crack width control, while the aspect ratio (length/diameter) is an important factor in understanding the bonding properties of the fibers to the matrix. Most steel fiber reinforcement today has an aspect ratio of 45 to 100, with lengths typically ranging from 2 to 3 inches. Fibers of this size typically ensure the desired failure mode of gradual bond loss and pull out of the fiber, as opposed to sudden fracture of the fiber itself. Larger aspect ratios typically are not used because they tend to lead to balling of the fiber reinforcement at higher dosages.

2.8.2 Synthetic Fiber Reinforcement

While various types of synthetics have been implemented for fiber reinforcement, the most commonly used synthetic fibers today are nylon and polypropylene. The more common of the two is polypropylene. There are two types of polypropylene fibers being manufactured, and each of them has been designed for a specific purpose. Fibrillated fibers are small (0.5 to 1.0 in. long) rectangular fibers that are longitudinally scored to split apart during mixing to create a more complex network of fibers throughout the matrix. The other synthetic fiber geometry is similar to that of steel fiber reinforcement and is designed for a similar use. These monofilament fibers come in a variety of diameters, lengths, and geometries. Fiber geometries include straight, corrugated, and cone-ended. Both fibrillated and monofilament fibers are manufactured through an extrusion process where the material is hot drawn through a die. Fibrillated fibers are drawn through a rectangular cross-section die, while monofilament fibers are drawn through a circular cross-section die.

The same properties that are important for steel fibers are also important for synthetic fibers. Polypropylene has a significantly reduced modulus (500 to 700 ksi) when compared to steel fibers (29,000 ksi) and is typically of a lower tensile strength (20 to 100 ksi) than steel fibers (50 to more than 130 ksi) (ACI Committee 544 1997). Despite the generally lower strength of synthetic fibers, the increased flexibility in the material allows for pull out of the fiber to occur before the fiber fractures.

2.8.3 Dosage Rates and Applications

Regardless of fiber type, low-dosage FRC (0.1 to 0.5 percent by volume) and high-dosage FRC (1 to 2 percent by volume) have distinct purposes and applications. Low-dosage FRC is currently used for the sole purpose of controlling plastic shrinkage cracking, as it does not provide any additional toughness. Low-dosage FRC is usually only composed of fibrillated synthetic fibers, because even at low volumes there are large numbers of fibers present in the matrix. High-dosage FRC is used when more significant improvements (toughness, fatigue, crack width control) are desired. These applications include shotcrete, thin bonded overlays, and slabs on grade. Typically, steel fiber reinforcement has been used when high-dosage FRC is desired, but continued development of synthetics has made them popular as well. Specific dosage rates and applications will be discussed in greater detail later in this chapter.

2.9 Effects on Mix Design

As with any traditional mix design, fiber-reinforced concrete needs to be proportioned to ensure it has good workability and finishability in its plastic state as well as the necessary strength and durability characteristics in its hardened state to provide good field performance. Because it is well understood that fiber reinforcement has little impact on strength parameters used in concrete mix design (compressive strength, flexural strength), the major modifications that need to be made for a given mix design, if any, focus predominantly on ensuring the proper workability and finishability of the mix.

Typically, most low-volume FRC requires no change to the mix design because it has little impact on the concrete's workability, but changes may be necessary if higher dosages of fiber reinforcement are desired. The ACI Guide for Specifying, Proportioning, Mixing, Placing, and Finishing of Steel Fiber-reinforced Concrete (1993) does recommend that for mixes containing higher dosages of steel fiber reinforcement, an increased amount of fine aggregate may be needed to maintain a desired fine-to-coarse ratio, which ensures the proper workability, finishability, and packing of the concrete matrix. The guide also recommends limiting the dosages of fiber reinforcement used as the maximum coarse aggregate size increases. This rule of thumb also holds true for higher dosages of monofilament and fibrillated synthetic fibers.

Traditional methods for improving workability can also be used in FRC mix design. Pozzolans can be used in FRC to improve workability as well as durability in the same manner they are implemented in typical concrete designs. Various low- to high-range water reducers, as well as air entrainments, may also be used in FRC, but the dosages necessary to produce the desired slump and air content value can vary significantly. These issues will be discussed in more detail in the fresh properties of FRC.

2.10 Effects on Fresh Concrete Properties

2.10.1 Slump

The slump of fresh concrete is an empirical parameter for determining the workability of a given mix. While slump can be used to determine workability for fiber-reinforced concrete, the methods and interpretation of slump measurements are significantly different compared to plain concrete. Fiber reinforcement increases the cohesion of concrete in its plastic state, which can make it appear to be less workable when the slump is measured using a standard slump cone (Johnston 1994).

The typical method of determining slump, ASTM C143, is a valid method of determining the slump of a given mix only if low dosages of fiber reinforcement are implemented (Folliard and Simpson 1998). Concrete with higher dosages of fiber reinforcement will have little or no apparent slump using the standard methods, even though the mix will be more workable than the slump would lead one to believe. This can again be attributed to the improved plastic stability and cohesion provided by fiber reinforcement. Because of this apparent loss in workability, other determinations of workability must be implemented when using fiber-reinforced concrete that has a measured slump less than 50mm (2 in.) (Folliard and Simpson 1998). The following section will discuss workability in more detail and discuss the alternative test methods to accurately determine workability for a given mix.

2.10.2 Workability

As described in the previous section, it is difficult to evaluate the workability of fiber-reinforced concrete using traditional empirical methods such as the slump cone test. While it is possible to use the standard slump cone test for mix designs containing very low dosages of fiber reinforcement, the ACI 544 state-of-the-art report for fiber-reinforced concrete (1997) recommends two alternatives to ASTM C143 to measure the workability of fiber-reinforced concrete. Both test methods determine workability as a function of concrete flow rate aided by mechanical vibration.

ASTM C995 is an inverted slump cone test that has been developed specifically for determining the workability of fiber-reinforced concrete. This test method is more user-friendly than ASTM C143 because it is simple and does not require any special equipment. Workability is determined by measuring the time required for the mechanically vibrated concrete to pass completely through a standard slump cone that has been inverted. The vibration is applied directly into the concrete using a typical mechanical hand-held vibrator. It should be noted that tests have shown that this method should only be considered valid when the mix has a traditional slump of 100 mm (4 in.) or less (Balaguru and Shah 1992). If this slump is exceeded, the concrete flows through an inverted slump cone much too rapidly to ensure accurate time measurements. In this situation, the impact of fiber reinforcement will be small enough to consider the standard slump cone test as an accurate indicator of workability.

The second method to accurately measure workability of fiber-reinforced concrete is the V-B, or Vebe, test. This test also measures concrete flow subjected to mechanical vibration but uses a V-B consistometer. This test is not widely used in practice because it requires specific equipment considered too large and cumbersome to be used in field applications (Folliard and Simpson 1998).

2.10.3 Air Content

It is recognized that having a well-developed air void network within the matrix is critical in mitigating freeze-thaw durability issues. This especially holds true for fiber-reinforced concrete because most current applications of FRC are flatwork-related (slabs, pavements, bridge deck, thin bonded overlays) and will be exposed to the outdoor environment. To ensure that the air void system is capable of controlling freeze-thaw damage, the voids must not only be the proper size but also be well-distributed through the entire concrete matrix. The air voids must also be capable of maintaining their stability throughout the processing and placement of the concrete.

Studies conducted by Cantin et al. (1995) investigated the effects of steel fibers on the ability to develop the desired air void network for various concrete strengths and steel fiber geometries. It was determined from this study that steel fiber dosages and geometry have little effect on the ability to produce a quality air void system in the matrix, but the dosages of air-entrainment used to develop the air void system may vary significantly. This need for larger dosages of air-entrainment was directly due not to the inclusion of steel fibers in the matrix, but rather to the increase in the dosages of water-reducer and superplasticizer used on account of the loss of slump and workability. The loss of slump and workability was attributed to increases in fiber dosages and decreases in the water-cement ratio when higher concrete strengths were required. It was also found that the type of mixer used would impact the quality of the air void system in the matrix.

Synthetic fibers have also been found to have no impact on the air content of the concrete matrix (Folliard and Simpson 1998, Bayasi 1993). As with steel fibers, the variations in the amount of air-entrainment used to achieve the desired air content can be attributed to the increased demand of water reducers to compensate for the lost slump and workability of the mix.

2.10.4 Plastic and Drying Shrinkage

Using fiber-reinforced concrete to control plastic and drying shrinkage cracking is one of its most promising applications. Much of the research that has been conducted relating to FRC has focused on the ability of fibers to control various types of shrinkage. This application for FRC is an economically justifiable implementation of fibers in concrete slabs, because shrinkage control has been good at very low fiber dosages. It has been well documented that using as little as 1.5 lb./cy (0.1 percent by volume) of fibrillated polypropylene fibers has the capability to reduce the total crack area due to plastic shrinkage cracking from 30 percent (Wang et al. 2001) to 70 percent (Berke and Dallaire 1994), depending on the geometry of the fiber used. Small, fibrillated polypropylene fibers have been found to be well suited for plastic shrinkage cracking because of their very small size (usually 0.50 to 0.75 in. in length). This means that even for the small volume of fibers that are used, the actual number of fibers distributed throughout the matrix is large. When the concrete is subjected to an environment that typically induces plastic shrinkage cracking, the fibers control the propagation of micro-cracking in the matrix and therefore prevent the formation of larger macro-cracks. It should be noted that these fibers are capable of controlling shrinkage only at early ages because the tensile strength of the concrete is so low, and the addition of these fibers helps redistribute stresses through the matrix such that the stresses to the matrix as a result of shrinkage are not capable of producing macro-cracks. Steel fibers are generally not considered a good option for controlling plastic shrinkage cracking; although they can reduce crack widths, physically there are not enough fibers in the matrix to control cracks at the micro-structural level at low volume dosages.

Most testing to determine the performance of FRC to control drying shrinkage has been conducted through restrained shrinkage tests. While the use of FRC does not eliminate drying shrinkage cracking in concrete, it does minimize the effects of drying shrinkage by limiting the crack widths. Research conducted by Shah et al. (1994) demonstrated that steel fibers can reduce the maximum crack width by 80 to 90 percent, and polypropylene fibers can reduce the maximum crack width by 70 percent. More recent research conducted by Altoubat and Lange (2001) has also shown that while both steel and polypropylene fibers can be used to control crack widths for restrained shrinkage, polypropylene fibers tend to provide less resistance to controlling cracks widths in similar environments due to their low modulus of elasticity.

2.11 Effects on Hardened Concrete Properties

When modern fiber-reinforced concrete research began, many believed fiber-reinforced concrete would be able to improve every aspect of concrete performance. Over the years, this has not been found to be true. In actuality, fiber reinforcement has been found to have little or no effect on the static properties of concrete. What fiber reinforcement does improve is the dynamic performance of concrete (fatigue, impact) and the post-cracking behavior of concrete by improving toughness and controlling crack widths. The following sections will discuss these various hardened FRC properties in more detail.

2.11.1 Compressive Strength

The compressive strength of fiber-reinforced concrete has been well documented, because no matter which aspect of concrete research is being conducted, the compressive strength is a parameter that is often related to the characteristics being investigated. There is a significant amount of data to show that the compressive strength of fiber-reinforced concrete will not vary significantly from the compressive strength of the same mix design without fibers. At best there may be a slight gain of ultimate compressive strength ranging from 0 to 15 percent when up to 1.5 percent by volume steel fiber reinforcement is used (ACI Committee 544 1997). This slight improvement can be attributed to the steel fibers controlling crack propagation at the micro-structural level, but there is too much variation to take advantage of any strength gains.

It should be noted that research has been done with synthetic fiber reinforcement that has shown there can actually be some losses in compressive strength when higher dosages or longer synthetic reinforcement is used. Research conducted by Naaman and Al-khairi (1995) has shown significant losses in compressive strength when 1 to 2 percent polypropylene fibers were used. This phenomenon was also recorded in compression tests conducted by Tavakoi (1994). Both of these studies attribute this loss of strength to an increased percentage of entrapped air because of the very high quantity of fibers in the matrix. These air issues can most likely be alleviated through proper vibration of the concrete during placement. Furthermore, tests conducted by

Berke and Dallaire (1994) showed little effect of polypropylene fibers at both low and high dosages (up to 2 percent by volume), thus supporting the idea that the losses in compressive strength observed are research-specific.

2.11.2 Tensile Strength

While there is a general consensus regarding the effects of fiber reinforcement on compressive strength, the effects of fiber reinforcement on tensile strength are ambiguous. Most feel that fiber reinforcement in general will only impact concrete performance once cracking has been initiated and thus will not have any impact on tensile strength in concrete. Research conducted by Berke and Dallaire (1994) supports this statement.

On the other hand, there has been research showing up to an 80 percent gain in tensile strength of both steel and synthetic fiber-reinforced concrete (Tavakoli 1994, Shaaban and Gesund 1993). Most likely this phenomenon can be attributed to the higher dosages (2 percent by volume) that were used, which were able to control micro-crack propagation by providing load transfer over the cracks. Another important consideration is that the load demand on the fibers is significantly lower on concrete tested in tension than it is on concrete tested in compression, which also explains the observed improved load carrying capacity in tension. Since the cracks are being held tighter by the fiber reinforcement, the macro-cracks that are typically observed immediately in plain concrete tested in tension are not present in FRC tensile tests. This is actually an increase in ductility and may be misinterpreted as an increase in strength by standard test methods that look for a sudden decrease in load carrying capacity. This misinterpretation becomes more apparent when low dosages of fiber reinforcement are tested in tension because the decreased ductility shows that tensile strength is not really increased. The research conducted by Berke and Dallaire (1994) and Folliard and Simpson (1998) supports this statement.

2.11.3 Flexural Strength

There is no consensus regarding the effect of fiber reinforcement on flexural strength. As with tensile strength, higher dosages of fiber reinforcement will control crack widths and improve load transfer across cracks to give an apparent increase in flexural strength by redistributing stresses. Data recorded in the 1960s and 1970s showed anywhere from a 50 to 70 percent increase in flexural capacity in FRC when compared to plain concrete in 3-point flexure tests (ACI Committee 544 1997). In reality, this is an improvement in the ductile response of concrete, not flexural strength. Tests using low dosages of fiber reinforcement show that there are no significant gains in flexural strength in FRC compared to plain concrete (Folliard and Simpson 1998, Berke and Dallaire, 1994). Other research also supports the claim that fiber reinforcement, even at higher dosages, does not actually improve the true flexural strength of plain concrete (Bentur and Mindess 1990, Balaguru and Shah 1992). This is why understanding the post-cracking flexural behavior of FRC in terms of flexural toughness is much more useful than attempting to determine the flexural strength performance of FRC.

2.11.4 Flexural Toughness

Flexural toughness is the primary parameter used to quantify the improvements that fiber reinforcing imparts on plain concrete. Toughness describes the post-cracking behavior of concrete in flexure through interpretation of the load-deflection plot at a prescribed mid-span deflection. There are various methodologies for interpreting the load-deflection plot to quantify toughness. In North America, toughness is described through toughness indices (I) and residual strength factors (R). Toughness indices compare the area under the load-deflection curve at various multiples past first-crack formation to the area under the load-deflection curve up to first crack. The residual strength factors describe the difference in toughness indices to better understand the post-crack behavior. In Japan, the parameter is merely called toughness (T) and is the area under the load-deflection plot up to prescribed mid-span deflections that are based on a certain fraction of the specimen length. This technique is dependent on specimen geometry, so toughness factors (F) were developed to better interpret toughness independent of specimen geometry. Issues and concerns regarding the validity of these values as a means of determining post-cracking behavior are discussed in more detail in the testing and specifications portion of this report.

It has been well documented that all fiber types at dosage rates greater than 0.25 percent by volume will improve toughness and enable concrete to carry loads well after cracking has begun (Mindess 1995). The actual degree to which toughness can be improved will depend on fiber dosage, fiber material properties, and bonding characteristics between the fibers and the concrete matrix (ACI Committee 544 1997).

For any fiber type, toughness will be increased as the dosage is increased. At higher fiber dosages, a greater percentage of the peak load can be sustained after initial crack formation when compared to lower fiber dosages of the same type. A larger area under the load-deflection curve is obtained, which leads to higher values of calculated toughness. When very high volumes of steel fiber reinforcement are used (4 to 5 percent by volume), it is possible to actually reach load capacities higher than the loading at initial crack formation (Balaguru et al. 1992). However, these dosages of fiber reinforcement are typically too high to be used in practice; therefore, it is considered that fiber reinforcement will not increase the load carrying capacity of concrete.

The stiffness of the fiber used will also have a significant impact on the toughness of the concrete. Polypropylene fibers tend to have a lower toughness than steel fibers at the same dosage level since they have a much lower modulus of elasticity (ACI Committee 544 1997). While polypropylene fibers can achieve the same high levels of ductility as steel fibers, their higher flexibility creates a more significant drop in load carrying capacity (Balaguru and Shah 1992). This leads to smaller areas under the load-deflection curve, resulting in lower values of toughness.

Finally, bonding characteristics are important for evaluating the toughness of FRC. Essential to improving toughness is improving ductility and eliminating the sudden fracture normally experienced by brittle materials. In FRC, this is achieved through a gradual pulling out of individual fibers as deformations increase until failure is eventually reached. Fracturing of individual fibers is not desired and will lead to a more brittle failure of the concrete. This is more difficult to achieve than it may seem, because if the bond between the fibers and matrix is too weak, the fibers will pull out too rapidly and the concrete will not obtain desired ductility and toughness. On the other hand, if the bond between the fibers and matrix is too great, the fibers will fracture and lead to undesirable brittle failure. Typically, as long as the fiber pulls out before critical stresses are experienced in the fiber, good ductility and toughness will be observed.

2.11.5 Fatigue

Fatigue is a very important consideration when designing pavements and slabs, which are typical applications of fiber-reinforced concrete. Improving fatigue performance can dramatically improve the service life for these applications, which can lead to long-term economic savings. Fiber reinforcement is a viable method of improving fatigue performance because it is capable of dissipating energy under dynamic loading and of controlling crack widths and crack propagation much better than plain concrete.

Both steel and synthetic fibers have been found to make significant improvements to plain concrete under fatigue loading. Polypropylene fibers were found to moderately increase fatigue flexural strength and increase the endurance limit (after 2 million cycles) by 15 to 18 percent for dosages ranging from 1.6 to 4.8 lb./cy (Ramakrishnan, Gollapudi, and Zellers 1987). Steel fibers were found to increase fatigue flexural strength by 200 to 250 percent and increase the endurance limit by 90 to 95 percent for dosages ranging from 66 to 100 lb/cy (Ramakrishnan, Oberling, and Tatnall 1987). Steel fibers tend to improve fatigue characteristics better than synthetic fibers because of their much higher stiffness, which enables them to more effectively control crack widths for a given level of loading.

2.11.6 Impact Loading

The ability of fiber-reinforcing to redistribute stresses through the concrete matrix makes it capable of greatly improving the impact performance of plain concrete. While there are various means of measuring fracture energy, ACI Committee 544 (1989) recommends using a drop weight test to determine the number of blows required to either crack or completely fail the test specimen. Tests conducted on synthetic fiber-reinforced concrete have shown significant improvements in impact resistance using the drop weight method as the dosage rate is increased (Bayasi and Zeng 1993, Soroushian et al. 1992). There have also been tests that show significant improvements in the impact resistance of steel fiber-reinforced concrete as the dosage rate is increased (ACI Committee 544 1997).

2.12 Test Methods and Specifications

One of the greatest obstacles of implementing new technologies into standard practice is the difficulty of establishing measurable parameters that can properly quantify the performance of a given technology and establishing testing methods that can reliably measure these parameters. Once this obstacle is overcome, specifications can be created for use in standard codes such as ACI, IBC, etc. This has been one of the greatest hurdles for implementing fiber-reinforced concrete in standard practice. While flexural toughness has been widely accepted as the only viable means of quantifying FRC performance, there have been many developments over the years toward devising methodology that reliably measures flexural toughness. Each proposed test method is a variation of the same basic concept: develop the load-deflection plot for a given flexural specimen and then interpret that data to arrive at the term known as toughness. The following paragraphs investigate the standard test methods that are currently used and provide a critical evaluation of each of these methods.

2.12.1 ASTM C 1018 (North America)

The ASTM C 1018 "Standard Test Method for Flexural Toughness and First Crack Strength of Fiber-Reinforced Concrete (Using Beam with Third Point Loading)" (ASTM 2002) was first published in 1984 and has undergone significant changes leading up to the current fifth edition published in 1997. This test method provides a value of relative flexural toughness, which relates the area under the load-deflection curve at first crack to the area under the same curve at a deflection that is a multiple of the first crack deflection. These values are known as toughness indices (I). From these toughness indices, residual strength factors (R), can be determined by relating various toughness indices, for example, $R_{5,10} = 20(I_{10}–I_5)$. Interpretation of the load-deflection plot to determine toughness and residual strength is shown in Figure 2.11.

Critiques of ASTM C 1018 published by Chen, Mindness, and Morgan (1995) and Johnston (1995) scrutinize the fact that the test method relies on being able to properly determine the *first-crack* deflection at the midspan of the specimen. Determining this value properly is extremely difficult because it is dependent on both the physical setup of the testing apparatus as well as the interpretation of the load-deflection plot. It has been well established that the net deflection (the deflection at midspan minus the deflection at the end supports), and not the nominal midspan deflection, must be calculated to derive the load-deflection plot. Research conducted by Banthia and Trottier (1995) and Chen, Mindness, and Morgan (1993) has documented that there can be significant differences in net deflection and nominal mid-span deflection if the testing apparatus is not sufficiently rigid to control its own deflections or if the concrete at the end supports begins to crush from continued loading. Care must be taken to ensure that the test setup at a given laboratory is able to accurately determine the net deflection of the specimen, which often leads to setups that are too expensive or complex for most typical laboratories to obtain. Recommended experimental setups are shown in Figure 2.12.

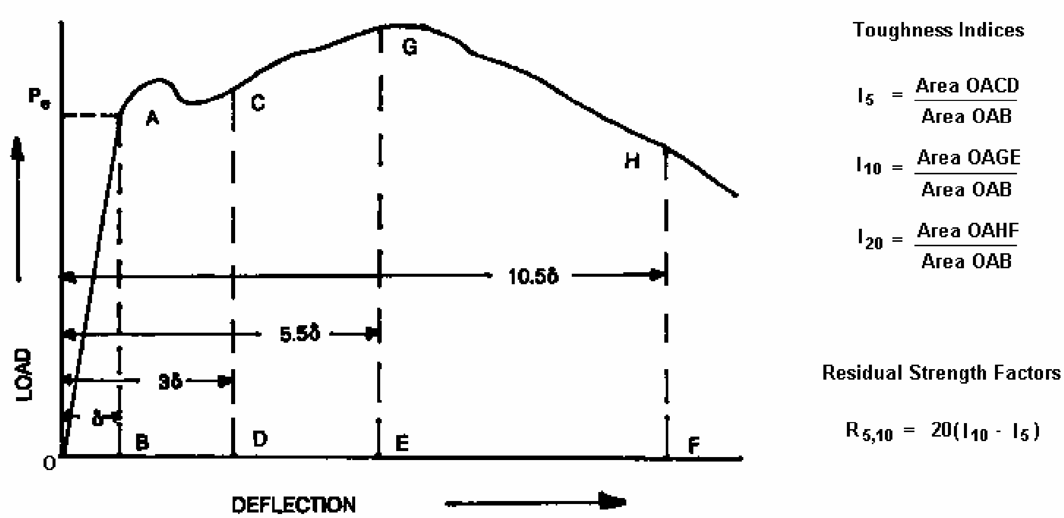
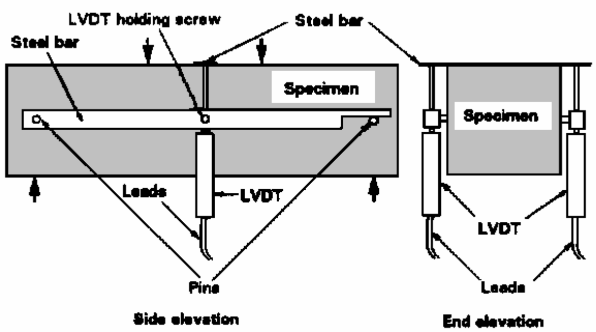
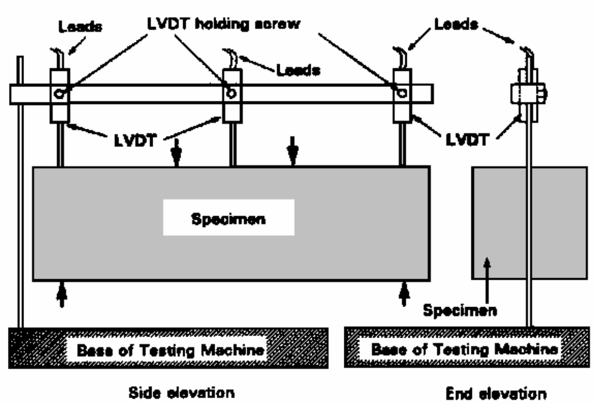


Figure 2.11: ASTM C 1018 Load-Deflection Interpretation (Tatnall and Kuitenbrouwer 1994)

Another issue in determining first-crack deflections is interpreting when the first crack is initially formed. ASTM C 1018 describes first crack as "the point at which the curvature first increases sharply and the slope of the (load-deflection) curve exhibits a definite change." While this definition may seem simple enough, most concrete, especially concrete with large amounts of fiber reinforcement, will not have one clearly defined moment when the specimen cracks but rather a series of microcracks that finally combine to lead to more significant macrocracking and energy dissipation. Chen, Mindness, and Morgan (1995) discussion of ASTM C 1018 provides an excellent example of how variations in the interpretation of first crack can lead to significant deviation in the calculated values of toughness indices and residual strength factors for the same load-deflection plot. The necessity of properly calculating the deflection at first crack is one of the primary weaknesses of using ASTM C 1018 or any other method that relies on a relative approach to determine flexural toughness.



Schematic Illustration of Japanese Yoke Deflection Measuring System



Schematic Illustration of Top LVDT's Deflection Measuring System

Figure 2.12: Recommended Experimental Setups for Flexural Toughness (Chen et al. 1995)

Because first-crack deflection is difficult to measure and interpret from the load-deflection plot, the inherent quality of various toughness indices and residual strength factors can differ significantly. Various research presented by Nemegeer and Tatnall (1995), and Chen et al. (1995) shows that the toughness indices I_5 and I_{10} do not properly distinguish differences in toughness among specimens with different fiber dosages. These toughness indices are calculated from deflections that do not significantly deviate from the first crack deflection (3 times first-crack deflection and 5.5 times first-crack deflection, respectively). Instabilities in the load-deflection curve can arise after first crack is initiated if an insufficiently tough specimen is tested on a test apparatus with insufficient stiffness, due to energy dissipation of both the specimen and the load frame (Chen, Mindness, and Morgan 1995). This region of instability will have a significant impact on the lower toughness indices, but stability is usually regained, making toughness indices that use larger relative deflections (I_{20}, I_{30}, I_{50}) a better indication of toughness (Chen et al. 1995, Nemegeer and Tatnall 1995). It should also be noted that while toughness indices that describe larger relative deflections do indeed provide better indications of toughness, residual strength factors should be viewed as a more viable method of determining toughness because these calculated values are impacted much less by the interpretation of first-crack deflection.

While there are many inherent problems with using ASTM C 1018, it is still the most widely used method to determine flexural toughness. Reliable data can be obtained from this method if the proceeding guidelines are followed.

1. The test apparatus must be sufficiently stiff to ensure minimal end-support deflections.
2. A well-designed, closed-loop data acquisition system should be implemented to properly measure net mid-span deflections and ensure stability of the entire load-deflection plot.
3. Care must be taken to determine first-crack deflection in the same manner for each test.
4. Higher toughness indices should be used to determine the necessary residual strength factors.

2.12.2 JSCE-SF4 (Japan)

The Japanese method of measuring toughness, JSCE-SF4, obtains the load-deflection curve in the exact same manner as ASTM C 1018; therefore, the same issues in regards to the test apparatus and data acquisition must also be addressed when conducting a JSCE-SF4 test. The critical difference between JCSE-SF4 and ASTM C 1018 is in the interpretation of the load-deflection curve. The Japanese method is an absolute method of determining flexural toughness, as opposed to the relative method used in ASTM C 1018. In JSCE-SF4, toughness is defined as the area under the load-deflection curve to a deflection value that is the length of the specimen divided by 150. A toughness factor, or equivalent flexural strength, is also calculated, which is a function of the specimen's geometry and its calculated toughness. This toughness factor was created in response to criticisms that the toughness value calculated using the Japanese method was completely dependent on the geometry of the specimen being tested (Chen, Mindness, and Morgan 1995). JSCE-SF4 interpretations are shown in Figure 2.13.

While there are many who feel that absolute testing procedures are inferior to relative testing methods, Chen, Mindness, and Morgan. (1995) present research that has been conducted by

Gopalaratnam (1991) and Chen, Mindness, and Morgan (1993) to show that JSCE-SF4 is actually better at differentiating the toughness of various fiber dosages and types as compared to ASTM C 1018. This is because the Japanese method for determining toughness is not dependent on the determination of the first-crack deflection, which has already been shown to be the key weakness of ASTM C 1018. The other advantage of using the Japanese method over ASTM C 1018 is its insensitivity to instabilities that may occur after cracking first occurs because the deflection value used (L/150) is significantly outside of the region where instability may occur. While steps should always be taken to minimize instabilities in the load-deflection curve, they cannot always be avoided for mixes that have little or no fiber reinforcement. JSCE-SF4 is a good option for implementation in construction specifications because it is simpler than ASTM C 1018 and enables more flexibility in the testing apparatus (Chen, Mindness, and Morgan 1995).

Chen, Mindness, and Morgan (1995) also present disadvantages that can be attributed to the test method's toughness values being independent of first crack formation. Since the Japanese method relies only on determining a standard deflection limit based on the geometry of the specimen, there can be huge variations in the shape of the load deflection plot in regards to its pre-peak and post-peak behavior, and it would still produce the same measured value of toughness. Another disadvantage of JSCE-SF4 is that the deflection value of L/150 is much greater than those often used in FRC applications, thus putting the calculated value of toughness out of the range of typical design loads (Chen, Mindness, and Morgan 1995). While JSCE-SF4 may not be used quite as frequently as ASTM C 1018, it is a good method for determining relative differences in flexural toughness performance.

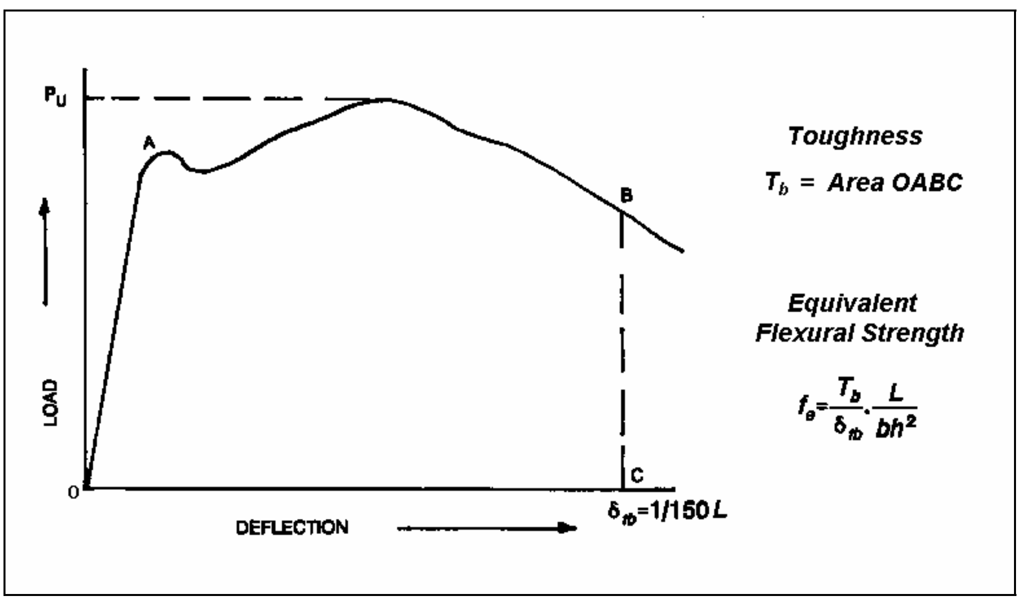


Figure 2.13: JSCE-SF4 Load-Deflection Interpretation (Tatnall and Kuitenbrouwer 1994)

2.12.3 New Developments—Template Approach

Chen, Mindness, and Morgan (1995) have proposed a new methodology for determining toughness that takes the strengths of ASTM C 1018 and JSCE-SF4 and combines them with

Norway's test method (NBP No. 7) for classifying fiber-reinforced shotcrete. The Norwegian specification uses different toughness classes to evaluate the performance of different FRC mix designs. These toughness classes are based on the residual flexural stress at predefined net mid-span deflections. This methodology of defining a minimum criterion for a given FRC mix design is known as a template approach, and its main advantage is that it allows for an easy determination of performance. As shown in Figure 2.14, these different toughness classes would be plotted on the load-deflection graph for a given test and used as a threshold for defining levels of concrete toughness.

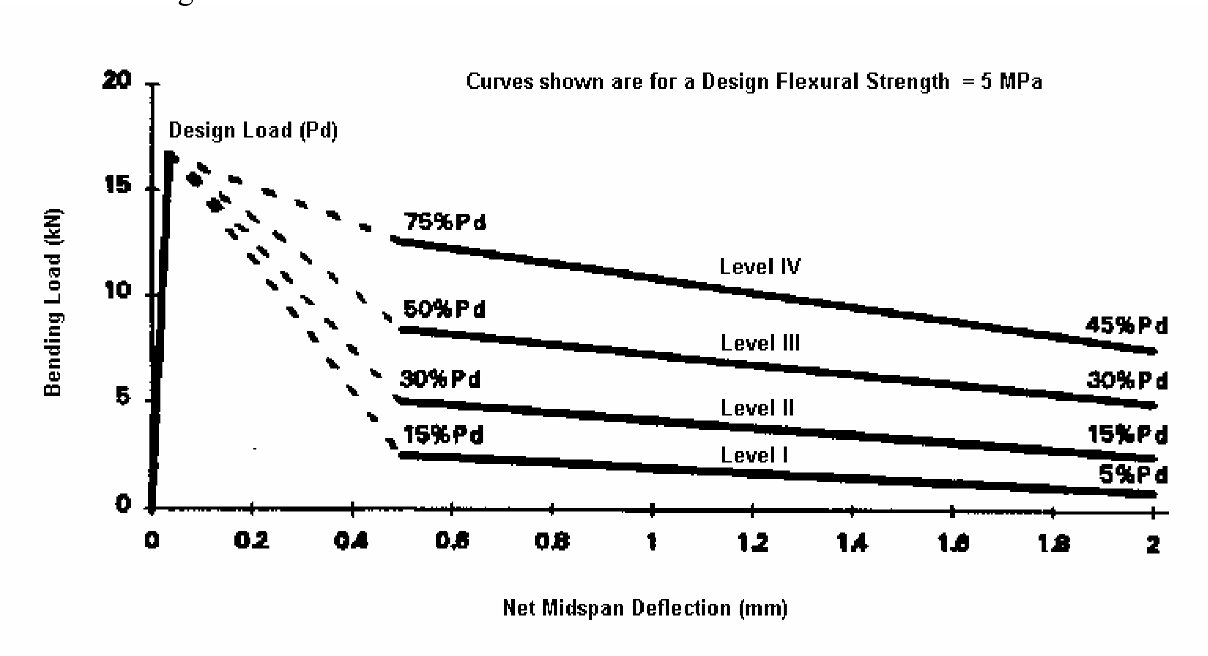


Figure 2.14: Template Approach (Chen et al. 1995)

The proposed methodology would follow the same specifications to create the load-deflection curve used in ASTM C 1018 and JSCE-SF4. The load-deflection curve would then be compared to predetermined toughness performance levels, which would be a percentage of the flexural strength of the specimen at net mid-span deflections of 0.5 mm (.020 in.) and 2.0 mm (.08 in.) (L/600 and L/150, respectively). Finally, a flexural strength and a toughness performance level would be assigned to the design based on where the load-deflection curve fell on template. Not only is this methodology simple, but it also can be applied to any previously available load-deflection data.

2.13 Applications of Fiber-reinforced Concrete

2.13.1 Industrial Floor Slabs

Fiber-reinforced concrete has been used for more than 20 years to replace traditional reinforcing bars and welded wire fabric in industrial floor slabs. The practice of using fiber-reinforced concrete for slabs tends to be used predominantly in Europe, where, by the early 1990s, there were already nearly 320 million ft^2 of FRC slabs in service (Tatnall and Kuitenbrouwer 1994).

While it has taken longer for North America to accept the use of FRC for industrial floors, its use is becoming more widespread every year.

Fiber-reinforced concrete is suitable for industrial floor slabs, because these elements are loaded in a very dynamic manner, making fatigue and impact performance key factors when they are designed. Shrinkage is also a concern due to the massive footprints these floors tend to have. Deterioration is another important factor. Since these floors are used for various manufacturing and storage purposes, deterioration can result in huge losses in productivity and could cost the owner substantial amounts of money in repairs.

Research has been conducted in Europe and North America to better understand the potential benefits of using FRC in industrial floor slabs. While synthetic fibers can be used in these floors, most of the research and implementation of FRC in industrial floor slabs has focused on steel fiber reinforcement. It has been found that steel FRC at relatively low dosages (34 to 50 lb/cy) can provide significant gains in load-bearing capacity for slabs on grade (Tatnall and Kuitenbrouwer 1994). The ability of FRC to dissipate energy, especially in dynamic loading, makes this phenomenon possible. It has also been shown through field performance that steel FRC is capable of producing the same load capacities as slabs using traditional reinforcement (Schrader 1987). This is often the case because of the improper placement of welded-wire mesh or reinforcing bars, which leads to a decrease in performance. Other enhancements that steel FRC provides to industrial floor slabs include increased spacing or elimination of contraction joints, reduced crack widths, and the possibility of slab thickness reduction when higher dosages of fibers are implemented (Robinson et al. 1994, Schrader 1987). Synthetic fibers have also been implemented in floor slabs for the sole purpose of controlling shrinkage cracking and are not used to provide enhanced load-bearing performance (ACI Committee 544 1997).

2.13.2 Pavements

The loading of pavements is essentially the same as that of floor slabs, and therefore the same parameters that influence slabs will affect pavements as well. A well-performing pavement will have a long fatigue life and small crack widths to eliminate typical pavement distresses (spalling, faulting, and punchouts). Pavement research has been conducted on both highway and airport pavements to quantify the performance benefits fiber reinforcement can add to these elements. Most research conducted in this area has been to improve the performance on nonreinforced, jointed plain concrete pavements (JPCP) through the use of steel fiber reinforcement.

Various field implementations of steel fiber reinforcement in pavements have shown promising results. Ramakrishnan (1995) documented the placement of two steel fiber-reinforced jointed concrete pavement sections in Rapid City, South Dakota. These sections used 0.5 percent by volume of either hooked-end fibers or corrugated fibers, and they were monitored over a period of five years. Results showed that there was minimal impact of the FRC on typical construction practices, and the performance of the FRC sections over the 5-year period was superior to that of nonreinforced concrete (fewer cracks, smaller crack widths). Research conducted by the U.S. Army Construction Engineering Research Laboratory has also shown significant improvements in jointed runway pavements subjected to airplane wheel loading and even allowed for pavement thickness reduction in comparison to plain concrete (Parker 1974).

Newer monofilament polypropylene fibers could be implemented in pavements because they have been developed to behave more like steel fibers, but more research needs to be conducted in this area. Synthetic fibers today are primarily used at a low dosage rate (0.1 percent by volume) to control plastic shrinkage cracking. This is particularly important for pavements, because it is a mass concrete element that is usually constructed in open environments that may lead to plastic shrinkage.

2.13.3 Thin Bonded Overlays

There has been substantial use of thin bonded overlays (TBO) containing fiber reinforcement in the United States. TBOs are primarily used as a repair method for pavements and have a high cost associated with them. Steel fibers are desirable in this application because they are much easier to place properly than traditional reinforcing steel used in thin bonded overlays. Also, the small cover associated with the use of traditional reinforcement in TBOs may lead to pavement distress. As with other flatwork applications discussed in the previous sections, FRC offers improved fatigue response and crack width control that is desirable for good performance of TBOs. This topic will be covered in more detail in Chapter 4 of this report, where the field performance of FRC thin bonded overlays is assessed.

2.13.4 Shotcrete

Shotcrete is sprayed concrete that is typically used for tunnel linings, slope stabilization, and concrete repair. Since shotcrete is brittle in nature, reinforcement is required to ensure good ductility and crack control. Initially, welded wire fabric was used with shotcrete to provide the necessary ductility. The main disadvantage to this method of reinforcement is that the fabric is often difficult and even dangerous to install because shotcrete is used with instable environments (tunnels, soil, etc.). This need for reinforcement installation results in increased costs because of labor and the potential for injury or death if the given tunnel or soil embankment were to collapse.

These concerns have led to the implementation of fiber reinforcement in shotcrete applications. Fiber reinforcement greatly simplifies the installation of shotcrete and also provides workers with a safer environment. Steel fiber reinforcement was initially implemented in shotcrete beginning in the 1970s, with synthetic fiber finally being implemented about a decade later (Morgan and Heere 2000). Typical dosages used for steel FRC shotcrete are 0.5 to 2.0 percent by volume, in which the dosage rate is dependent on fiber geometry (fibers with higher aspect ratios require smaller dosages) (Ramakrishnan 1995). Both fibrillated and monofilament polypropylene fibers have been used for shotcrete applications, with dosage rates averaging 0.5 percent and 1.0 percent by volume, respectively (Morgan and Heere 2000). Both steel and synthetic fibers at these prescribed dosages have been found to be successfully applicable to shotcrete, providing the required improvement in ductility and impact resistance.

The main concern with using fiber reinforcement is the high level of rebound that is experienced during installation. While rebound is expected of shotcrete, the concern when fiber reinforcement is used is that the fibers tend to have a higher percentage of rebound as compared to the rest of the concrete material. This can lead to performance issues and a lower in-place percentage of fiber reinforcement than that for which the mix was designed (Armelin and Banthia 1998). In fact, fiber rebound can be as high as 80 percent to 90 percent for both steel and synthetic fibers, while the overall shotcrete rebound is only 20 to 40 percent (Armelin et al. 1997, Beaupre and Lamontagne 1995).

It has been shown that the addition of silica fume in the mix design will create a more cohesive mix and help control this differential rebound between the fiber reinforcement and the rest of the matrix (Wolsiefer and Morgan 1993). Also, proper installation helps to minimize rebound effects.

2.13.5 Structural Members

The use of fiber reinforcement for structural elements has been rather limited in actual field applications because of the difficulty in properly understanding how to translate the ductility and toughness fibers provide into usable design standards. Despite this current lack of implementation, there is much research being conducted to better understand how fibers improve the ductility of structural members. It should be noted that almost all structural FRC research being conducted is focusing on steel fiber reinforcement (Balaguru 1995). One of the most promising uses of FRC is in earthquake-resistant design, where ductility is the key to creating a successful design.

The main aspect of earthquake resistance being investigated using FRC is improved shear strength, especially at beam-column connections. Shear failures in buildings are undesirable due to the very brittle nature of this failure. The goal of introducing FRC at beam-column connections is to improve the shear response such that ductile, flexural hinges will be able to form. Another benefit to using fibers at connections is that the large amounts of shear reinforcement required at joints by seismic design codes often make it difficult to construct. If FRC can be used to reduce or eliminate traditional shear reinforcement at the connections, large savings in labor costs can be obtained. Research conducted by Sood and Gupta (1987) showed significant improvement in the shear strength of beam-column connections using only up to 1 percent steel fiber reinforcement. The improvements were so great that the transverse reinforcing bars could actually be eliminated from the joint regions. More recent research conducted at Ecole Polytechnique in Montreal has also found that steel fiber reinforcement could be used as a potential replacement for conventional steel reinforcement (Filiatrault 1998).

As stated previously, the success of implementing FRC into structural engineering practice will be the ability to create design provisions that can be applied to any fiber type and dosage. To attain this goal, much more research needs to be conducted to have a database large enough to properly quantify the benefits of FRC in structures.

2.14 Potential Applications of Fibers in CRCP and TBO in Texas

Continuously reinforced concrete pavement (CRCP) has been widely used in Texas for a number of years, generally providing a service life that exceeds its intended design life. However, several pavement sections have experienced significant pavement distresses that resulted in premature failure. It is generally understood that the cause of these failures is related to design, construction, and materials. One way to provide better-performing pavements is to avoid materials that cause distress. However, because of the abundance of many of these materials, significant effort and research has been applied to utilizing these materials while still achieving an acceptable level of performance. The addition of fibers may help to offset some of the damage caused by these deleterious materials by providing a reduction in crack width, an increase in toughness, and a reduction in plastic shrinkage. There are also some challenges that fibers present in the incorporation of CRCP, such as constructability, rideability, and an increase in cost. It is difficult to quantify the effects of fibers, and thus difficult to implement in design code. There have been several studies that have shown steel fibers used in thin bonded concrete overlays to be an excellent supplement to concrete pavements.

2.15 Field Performance of Bonded Concrete Overlays

Continuously reinforced concrete pavement (CRCP) has been in existence for over 80 years. Because of the traffic loads these pavements experience, repair and rehabilitation is inevitable. CRCP provides a suitable foundation for repair and rehabilitation. The absence of joints in CRCP generally reduces the amount of reflective cracking associated with overlays, thus increasing the effectiveness of the pavement repair. Overlays provide an economical repair solution and have been used to resurface pavements for more than 75 years. There are two types of overlays that are used for rehabilitation: flexible and rigid overlays. Flexible overlays are constructed of paving materials made from bituminous materials, and rigid overlays are primarily constructed from portland cement concrete. The two main types of rigid overlays are bonded and unbonded concrete overlays. More than 1,000 miles of continuously reinforced concrete (CRC) overlays have been constructed in the United States since 1959. Bonded concrete overlays (BCO) provide an economical solution that requires minimal maintenance when designed and constructed properly (Sriraman and Zollinger 1999). It is very important to provide adequate surface preparation to ensure that sufficient bond is achieved between the existing pavement surface and the overlay.

Bonded concrete overlays are used for several different rehabilitation purposes. These include pavements that did not meet required specifications during construction, those that have suffered surface damage in the early part of its service life, and those wanting improved rideability. It is important that the BCO maintain sufficient bond so that the tensile stresses can be transferred to the existing pavement. Two of the main factors that affect bond are the cleanliness and strength of the existing pavement surface (Sriraman and Zollinger, 1999).

Bonded concrete overlays are often impaired by various distresses, resulting in premature failure. One of the main distresses associated with BCO are delaminations. A poor bond between the overlay and the existing pavement surface causes delaminations. Another distress associated with BCO is transverse cracking. Transverse cracking is often manifested through reflection cracking from the existing pavement. It is also important to match joints between the two surfaces to prevent additional reflection cracking (Sriraman and Zollinger, 1999).

There have been several studies conducted on I-610 in Houston that evaluated the different types of bonded concrete overlays. These studies assessed the overlay performance as it related to different types of reinforcement, coarse aggregates, bonding agents, and existing pavement conditions. The issue of delamination was also evaluated in one study. The project study verified that BCO significantly reduced the pavement deflection, giving an improvement in the riding quality. The overlays also helped to reestablish the load transfer across the cracks in the existing CRCP surface, improving the fatigue life of the pavement. The sections containing limestone had significantly fewer transverse cracks (Koesno and McCullough 1987). It was also determined that delaminations occur within the first few weeks of the construction.

Major problems with BCO in previous applications were associated with poor construction methods. However, many of these issues have been addressed, resulting in better-performing overlays that significantly lengthen the life of existing pavements. Current overlay designs also generally incorporate steel fibers or reinforcing steel bars, as opposed to welded wire mesh.

Bonded concrete overlays (BCO) provide an economical solution that requires minimal maintenance when designed and constructed properly. BCO construction requires adequate surface preparation to ensure a sufficient bond between the existing pavement surface and the overlay. Delaminations, transverse cracking, and reflection cracking often impair BCO. Several field studies have been performed to assess the performance of bonded concrete overlays. Overall, BCOs have greatly enhanced existing pavements by reducing pavement deflection, improving riding quality, and reestablishing load transfer. Many of the problems that BCO have experienced are associated with inadequate design or poor construction methods (Sriraman and Zollinger, 1999). The majority of these problems have now been resolved.

There are still areas within BCO that can be improved. The addition of modern fibers may help to enhance the use of river gravel in BCO. Since there are several BCO already constructed with steel fibers, the first step should be to continue monitoring previous sections to obtain long-term data. Additional studies could then be made based on this monitoring. Synthetic fibers should also be considered a possible solution. The final step should be to perform additional field studies with modern fiber types and dosages. These field studies should be performed similarly to the ones completed for the CRCP study.

2.16 Existing Problems with CRCP in Texas

Most CRCP constructed in Texas have experienced much larger traffic loads than predicted and still have provided service lives exceeding their intended design lives. However, there are many cases where the pavement has been under-designed or constructed with poor materials, resulting in significant pavement distress and premature failure. A simple solution would be to avoid the materials that generally cause distress. However, significant effort and research has been conducted to utilize poor materials and still achieve an acceptable level of performance.

One design problem in Texas CRCP is related to inadequate splicing of the longitudinal reinforcing steel. It is important for reinforcing steel to develop its full tensile strength to prevent a pull out failure. Using a single splice location for the longitudinal steel often augments this type of failure. This creates a weakened plane that prevents the steel from developing its full tensile strength. This flaw has been amended by requiring a staggered splice location that reduces the areas of weakness (Buch et al 1999).

The type of material used in CRCP construction also has a great effect on the pavement performance, particularly the aggregate type. Because of the large volume that it occupies within the concrete, the aggregate type dominates the concrete strength, coefficient of thermal expansion (CTE), modulus of elasticity, and amount of shrinkage. The two main aggregate sources for Texas, as described in Section 2.2.3, are crushed limestone and siliceous river gravel.

Field performance has shown that pavements constructed with crushed limestone generally perform much better than those constructed with siliceous river gravel. TxDOT has spent a considerable amount of effort to use river gravel in concrete pavements but with little success. It is generally believed that the problems with river gravel are associated with CTE, modulus of elasticity, and/or bond strength. River gravels also have higher values for modulus of elasticity than limestone, thus reducing the amount of creep within the pavement. This is also a disadvantage of river gravel because creep helps to reduce the stiffness of the concrete and results in lower stresses.

There are several construction issues that have been known to affect CRCP performance. These include placement season, placement time, and ambient temperature. TxDOT has recently recognized the effects of ambient temperature and has introduced a maximum temperature of 95º F in an effort to minimize the effects of plastic shrinkage. The season and time of day that CRCP is constructed also has a significant effect on the crack pattern development. It is primarily affected by thermal gradients in the slab and uniform temperature changes. Concrete placed in the morning generally sets at a higher temperature and develops higher stresses and more early-aged cracking than concrete placed in the afternoon, giving shorter crack spacing. Night placement has been shown to give some improvement, but the long-term results indicate only slightly better performance than daytime placement (Dossey and McCullough 1999).

The season of construction plays a major role in concrete performance. Field studies have shown that pavement sections placed during the winter do not experience as much cracking over time as those placed during the summer. Concrete placed under hot weather conditions is affected by an increase in drying shrinkage (Buch et al. 1999). However, recent studies have shown that pavements containing river gravel that are placed in the winter tend to develop more spalling because of the way in which the crack propagates. Field studies are currently underway at The University of Texas at Austin to determine the effect that placement season has on spalling.

2.17 Possible Benefits of Fiber Reinforced Concrete in CRCP

The implementation of fiber-reinforced concrete (FRC) has the potential to greatly improve the performance of continuously reinforced concrete pavements because of the increased post-cracking ductility FRC contributes to plain portland cement concrete. The following sections will discuss each parameter that fibers can improve in CRCP performance.

2.17.1 Reduced Plastic and Drying Shrinkage Cracking

Since CRCP is typically quite thick, the internal heat produced during the hydration of the cement can increase the tendency of shrinkage. This is even more evident in Texas, where placement temperatures often reach levels that will amplify the heat generated by hydration.

It is well documented that low dosages of fiber reinforcement (0.1 percent by volume) will mitigate shrinkage cracking by controlling crack propagation at the microstructural level and limiting the capability of the microcracks to develop into more significant macrocracks. The ability of fiber reinforcement to redistribute stresses is key to FRC's ability to control crack widths. It has also been shown that fibrillated polypropylene fibers, which are typically used for plastic shrinkage control, reduce the bleeding of the concrete. This can help ensure that there is sufficient moisture inside the matrix to control drying shrinkage at later ages. The reduced bleeding may cause concern because bleed water acts as a protective barrier for concrete in its plastic state. While this is normally the case, the ductility imparted on the matrix by the presence of fiber reinforcement reduces the importance of bleed water in controlling plastic shrinkage.

2.17.2 Crack Width Reduction

Minimizing transverse crack widths is crucial when evaluating the performance of CRCP. Unlike other types of concrete pavements, the restraint imparted on the pavement by the longitudinal reinforcement establishes transverse cracks spacings from 3.5 ft. to 8 ft. Small crack widths are also desired in CRCP because as the crack opening increases, water and non-deformable particles (i.e., sand) can become lodged into the crack opening and create increased local distresses around the crack. These distresses will eventually lead to local failures in fatigue and spalling of the surface.

Fiber reinforcement is an excellent means to ensure that small crack widths are maintained in CRCP. As the matrix cracks, the fiber reinforcement increases the toughness of the system by allowing a distributed transfer of load across cracks that normally cannot transfer load. This continued post-cracking load transfer allows the matrix on either side of the crack to act as a continuous material, and therefore the expansion and contraction typical across transverse cracks is significantly less than there would be without fibers providing local load transfer. The improved load transfer reduces stress concentrations normally associated with cracked materials and leads to not only a decrease in crack propagation but also an increase in fatigue performance. This in turn, reduces the potential for spalling along crack edges.

2.17.3 Enhanced Performance of Longitudinal Steel

While fiber reinforcement would provide load transfer across cracks if implemented in CRCP, the primary load transfer system is still the traditional longitudinal reinforcing steel. The longitudinal steel must not only transfer the load across cracks but also interact with the uncracked concrete surrounding it to ensure full composite action and transfer of loads between the steel and the concrete. Debonding of the longitudinal steel could reduce concrete performance and initiate premature failure after multiple load cycles.

Fiber reinforcement is well documented to enhance bonding between traditional reinforcing bars and the surrounding concrete matrix by controlling any microcracks that may form around the reinforcement and lead to eventual bond loss in fatigue. This improved bonding performance provided by FRC will help enable the longitudinal steel to properly transfer loads across cracks and into the rest of the uncracked concrete matrix. This improved load transfer will result in improved fatigue performance and increased service life.

2.17.4 Improved Ride Quality

The ride quality of CRCP is dependent on controlling the typical pavement distresses that lead to the loss of the concrete cross-section. These distresses include spalling, punchouts, blowups, faulting, etc. Every one of the distresses listed is caused by the same general mechanism. Cracks initially form and then begin to widen and propagate through fatigue loading. Eventually a section of concrete loses its bond to the rest of the pavement and creates various *potholes* or *bumps* in the road.

Fiber reinforcement can help reduce or eliminate all of these distresses by controlling crack widths and propagation throughout the life of the pavement. The increased toughness imparted on the CRCP by fiber reinforcement helps maximize bonding of the concrete matrix across every crack. These enhancements keep the pavement in tact, ensure adequate traction between vehicles and the riding surface, and help maintain safer highways.

2.17.5 Improved Spalling Resistance

While the actual mechanism that causes spalling is not completely understood, it is known that controlling crack widths will help improve spalling resistance. This fact is what makes fiber reinforcement a viable means to mitigate spalling in CRCPs. The ability of fiber reinforcement to control plastic and drying shrinkage cracking has been well documented in the laboratory, and these same results should be expected in the horizontal and vertical cracking that results in spalling concrete pavements. Fiber reinforcement improves the bond between traditional reinforcement and concrete by controlling cracking in the interface region. This improved performance should also reduce mid-depth horizontal cracking, which has been linked to excessive spalling in CRCP.

2.17.6 Extended Service Life

In pavement design, service life is based primarily on fatigue. In other words, the life of a given pavement will be determined by its ability to withstand a certain stress limit for a prescribed number of cycles, which is dependent on the traffic type and density. Fiber reinforcement will no doubt be capable of extending the service life of any pavement due to its improved fatigue performance. The degree to which the service life is extended will be a function of the fiber types and dosages, but improvements will be observed nonetheless.

2.18 Challenges of Implementing FRC in CRCP

While there is no doubt that using fiber-reinforced concrete in conjunction with continually reinforced concrete pavements leads to significant improvements in pavement performance, there are a few important issues that may impede the joining of these two technologies in standard practice.

2.18.1 Difficulty in Correlating FRC Performance to Pavement Design

As has been the case with all FRC applications, there has been great difficulty in relating flexural toughness, which is the predominant parameter used to evaluate FRC performance, to traditional parameters used for design. Without an effective means of quantitatively relating fiber performance to pavement performance, FRC will most likely never be used in standard practice. Currently, the only way to truly understand the effect fiber reinforcement has on CRCP or any other type of pavement has been through field performance monitoring. While this method does shed some light on the influence of fiber reinforcement on pavement performance, it only determines performance for the particular fiber type and dosage that was implemented in the field. This is a huge disadvantage because not only can it take decades to see long-term performance effects, but also the fiber studied will almost certainly be obsolete by the time sufficient data has been obtained.

While there is a large database of lab data quantifying FRC performance, continued research is necessary to build a larger, more diverse database of long-term field performance of CRCP with various fiber types and dosages. Only after this has happened can reliable mathematical relationships be derived to properly correlate lab data to the performance field of FRC.

2.18.2 Constructability and Finishability

Since CRCP mixes typically have little slump (1.5 to 3 in.) and are less workable than many other types of concrete mixes, there is some concern in regards to the impact fiber reinforcement will have on standard CRCP construction practices. It is well known that as the dosage of fiber reinforcement for a given mix increases, there can be significant losses in slump and workability. CRCP is generally placed with slipform paving operations. It is critical that FRC not have any significant impact on the operation of this machinery. Also, the finishing and tining of the surface must not be affected by the addition of fibers to ensure easy placement and adequate ride quality. Fibers, particularly steel, protruding from the pavement surface could lead to increased tire wear and possibly tire blowouts. For these challenges to be overcome, more studies should be conducted to better understand how fiber type and dosage would impact CRCP construction and finishing practices.

2.18.3 Increase in Cost

The largest hurdle fiber-reinforced concrete will need to overcome is not performance based but economically based. Since pavements consist of such a large quantity of raw materials, slight increases in the material cost per cubic yard will make or break a project. This is even more critical in Texas, where inexpensive, non-unionized labor makes the material cost a larger percentage of the overall project cost. This puts fiber reinforcement at a huge disadvantage because currently the price per pound of fibers is orders of magnitude higher than aggregate and reinforcing steel costs. Only if fiber prices decrease or FRC is written into project specifications will FRC likely be used in standard, lowest-bid construction.

2.18.4 Conclusions and Recommendations for Future Research

Continuously reinforced concrete pavement (CRCP) has generally performed very well in Texas. However, there are still performance issues that plague CRCP. Most of these issues have been handled by modifying design procedures and construction guidelines. However, some of the material-related issues have yet to be resolved. The simplest solution to this problem is to avoid using these poor materials, but there is great incentive to continue utilizing these resources despite their history of poor performance. The biggest issue related to materials is the issue of spalling for CRCP containing siliceous river gravel. A considerable amount of research has been conducted to determine its distress mechanisms, but little success has been found due to the complexity of its nature. Many of the problems associated with siliceous river gravel may be resolved with the addition of fibers, although this could be a costly solution.

There are a variety of different fibers available on the market, mainly steel and synthetic. At the practical dosage rates, potential benefits for the concrete are an increased resistance to plastic shrinkage, an increase in toughness, an increased resistance to impact and fatigue loads, and a small increase in flexural strength. In order to quantify these effects, laboratory and field tests should be implemented. Each of these tests should evaluate siliceous river gravel and crushed limestone with varying dosages of steel and synthetic fibers to determine the most efficient fiber type and dosage. The fibers should be selected based on previous research, economic feasibility, and manufacturer recommendations. Laboratory tests should be used to provide a foundation for the field tests. Laboratory tests should include standard tests such as compression, flexural, and splitting tensile strength. In addition, early-age properties, mainly tensile strength, should be measured using state-of-the-art test methods. Plastic shrinkage, toughness, and fatigue resistance should also be evaluated.

It is difficult to simulate CRCP distress mechanisms in a laboratory setting, particularly spalling, which is considered to be the most detrimental distress in Texas; it is a complex phenomenon that results from the interaction of several variables such as materials, environmental conditions, and traffic loading. Accelerated pavement testing is often used to test pavements, but this type of testing is not always economically feasible. As a result, field tests are generally considered the most viable alternative to evaluate CRCP performance. Field tests are essential as they can realistically depict crack spacing, crack width, and pavement distress development through qualitative monitoring. Another benefit of field tests for fiber-reinforced concrete pavement is that field tests verify the impact that fiber dosages have on the workability of paving operations. Fibers affect workability in several areas that include batching, mixing, transporting, and paving. Finishing, tining, and curing are also important issues that can be investigated in a field test. There are several other considerations that should also be evaluated prior to the selection of a field study. It is important that the field test be located in an area that experiences relatively high traffic loads and extreme environmental conditions. The pavement should also be constructed with siliceous river gravel, because this is the type of pavement in which spalling generally develops. There should be several test sections that contain fiber types and dosages similar to the ones used in the laboratory testing. The concrete mix should be optimized to account for the addition of fibers. The test sections should also contain instrumentation that provides temperature data.

Chapter 3. Laboratory Evaluation

3.1 Overview

In order to implement fiber reinforcement in continuously reinforced concrete pavements (CRCP) design, it is important to understand how fibers will improve the inherent properties of typical CRCP mixtures. The goal of this study was to quantify these effects by performing a comprehensive evaluation of the various fresh and hardened concrete properties relevant to fiber reinforcement and pavement construction, behavior, and design. A variety of fiber materials, geometries, and dosages were used to determine the most effective means of improving the various properties being investigated. Because pavement construction is driven by material costs and the relative cost of fiber reinforcement is high, only low-dosages of fibers were tested. Both crushed limestone and siliceous river gravel were used in this study to provide insight into why limestone typically provides better long-term spalling performance than siliceous river gravel.

The data obtained in the laboratory will also be used to provide correlations between fiber reinforced concrete properties and the actual performance of CRCP with fibers as determined by the field studies. It also provides quality control data to ensure that the various concrete properties evaluated in the field study are accurate and repeatable.

The sections of this chapter will cover the following aspects of the laboratory investigation: materials, mixture designs, testing procedures, test results, and conclusions.

3.2 Materials

The materials used in the laboratory evaluation were chosen to complement the materials used in the field evaluations. The goal of using similar materials is to improve the correlation between the performance parameters evaluated in the laboratory study and the actual performance of the field pavements. All the materials that will be discussed in the following sections are typically used in CRCP throughout Texas, which will allow for implementation of the knowledge gained by the laboratory findings.

3.2.1 Cementitious Material

The cementitious materials chosen are those that are typically used in new concrete pavement construction in Texas. In particular, the materials used for this laboratory study were obtained from the same sources that were used for the field evaluations to reduce variability between the two studies. The cement is classified as Type I/II, as specified by ASTM C 150. The fly ash is classified as a Class C fly ash and conforms to ASTM C 618.

3.2.2 Aggregates

The choice of coarse aggregate is critical because it has been attributed to the greatest variation in field performance of CRCP. It is also one of the primary variables being investigated in the lab evaluation. Crushed limestone and siliceous river gravels are the primary coarse aggregates used in pavements in Texas. While the actual mechanism is not completely understood, the use of siliceous river gravel has been found to be the primary cause of premature spalling of CRCP in

Texas. The aggregate properties that have been attributed to this variation in CRCP performance are coefficient of thermal expansion, elastic modulus, angularity, and bonding.

A limestone known to provide good field performance and siliceous river gravel known to provide poor field performance were chosen for the lab evaluation to provide insight into the differences between the two materials and how fiber reinforcement influences their typical fresh and hardened properties. The siliceous river gravel used in this study is also from the same source as the field investigations to provide correlation between the laboratory and field studies. The siliceous river gravel is a TxDOT Grade 2 coarse aggregate that also conforms to ASTM C 33 (Size 467). The fine aggregate that was chosen is a TxDOT-designated Grade 1 natural river sand that also conforms to ASTM C 33. The sand is also from a source that is typically used for new pavement construction in Texas. Table 3.1 lists the properties of the coarse and fine aggregates used in the laboratory evaluation as well as the respective aggregate specifications found in TxDOT Item 421.2.

Table 3.1: Aggregate Properties

Property	Sieve Size	Coarse Aggregate (%)			Fine Aggregate (%)	
		TxDOT Spec. For Grade 2	**Crushed Limestone (LS)**	**Siliceous River Gravel (SRG)**	**TxDOT Spec. For Fine Agg.**	**Natural River Sand**
Cumulative Percent Passing for each Sieve:	2"	100	100.0	100.0	-	-
	1 1/2"	95-100	98.9	100.0	-	-
	1"	-	73.3	92.0	-	-
	3/4"	35-70	51.5	84.6	-	-
	1/2"	-	22.5	62.5	-	-
	3/8"	10-30	12.8	26.7	100	100
	No. 4	0-5	2.6	3.3	95-100	98.8
	#8	-	-	-	100-80	89.3
	#16	-	-	-	50-85	68.8
	#30	-	-	-	25-65	44.6
	#50	-	-	-	10-35	18.8
	#100	-	-	-	0-10	4.5
	Pan	-	-	-	0-3	0.5
Bulk Specific Gravity		-	2.59	2.58	-	2.65
Fineness Modulus		-	NA	NA	-	2.75
Absorption Capacity (%)		-	1.65	0.73	-	0.66

3.2.3 Admixtures

The chemical admixture used in the laboratory study was a water-reducing/retarding admixture. It is the same admixture that was used in the field evaluation to provide better correlations between the two studies. The low range water-reducing/retarding admixture conforms to ASTM C 494 (Type A and D). It is an aqueous solution that is blended from refined lignosulfate salt and other water-reducing and plasticizing materials. No calcium chloride is added throughout the manufacturing process. This type of admixture is generally used for applications that require extended set time. Typically, an air-entraining agent is used in conjunction with a low range water-reducing/retarding admixture in Texas pavements, but it was omitted from this study. This exclusion was recommended by TxDOT due to the lack of freeze-thaw problems in CRCP in Texas.

3.2.4 Fibers

The various fiber reinforcements chosen for the lab evaluation are the typical types currently used in practice and are the most readily available. The fibers used vary in material, geometry, and cost to provide an unbiased, comprehensive evaluation of fiber reinforcement currently being manufactured. The goal is to determine which fiber will offer the greatest improvement in performance at the most reasonable price. The fiber types chosen are also the same as the ones implemented in the field evaluations to provide consistency between the two studies. Fiber dosage is also a major contributor to improved performance, and this will be discussed in greater detail in the mixture proportions section of this chapter. Table 3.2 lists the fiber types used in this study as well as the material properties and geometries. This table also shows the fiber designations that will be referenced throughout this chapter. Figures 3.1 to 3.4 provide images of each of these fiber types.

Table 3.2: Fiber Designation

Fiber Designation	Description	Length (in)	Aspect Ratio
SF1	Steel—Collated Hooked-End	2.36	65
SF2	Steel—Corrugated	1.97	44
SnF1	Synthetic—Monofilament	1.57	90
SnF2	Synthetic—Collated-Fibrillated	< 1.18*	NA*

*The SnF2 fiber is graded and does not conform to a specific length or aspect ratio.

Figure 3.1: SF1 Fiber Type

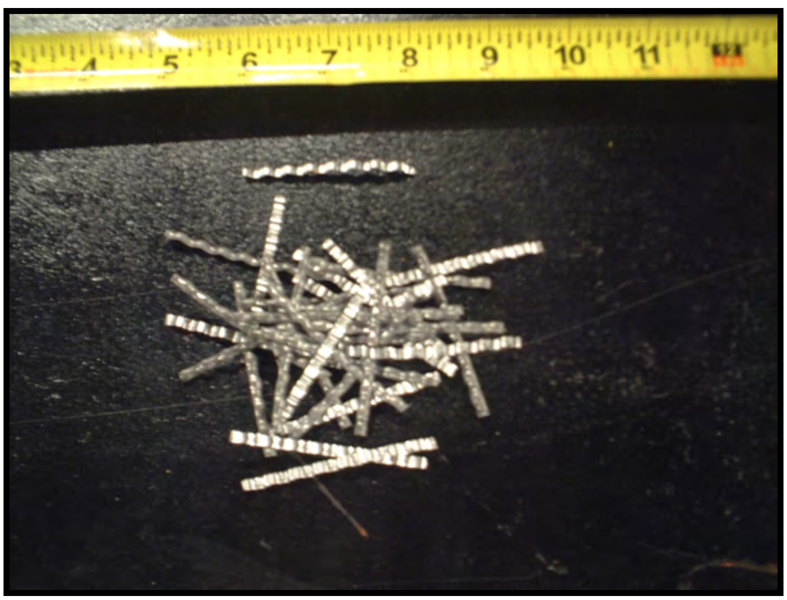


Figure 3.2: SF2 Fiber Type

Figure 3.3: SnF1 Fiber Type

Figure 3.4: SnF2 Fiber Type

3.3 Mixture Proportions

Because the goal of this study is to provide correlation between material properties in the laboratory and actual pavement performance in the field, the mixture proportions used in the laboratory were designed to be very similar to those that were used in field evaluations. All mixtures used in this study were designed to meet the TxDOT specifications for a Class P concrete, which is used for all CRCP, including the field study. These requirements are located in Item 421.9 of the TxDOT Standard Specifications Manual and are outlined in Table 3.3.

Table 3.3: TxDOT Specifications for Class P Concrete

Parameter	Requirement
Cement Per Cubic Yard, Minimum	5.0 Sack (470 lb)
Minimum Flexural Strength @ 7 Days	555 psi
Maximum Water/Cementitious Ratio	6.25 Gallons/Sack (0.55 w/cm)
TxDOT Coarse Aggregate Grade Number	2,3
Desired Slump	1.5 inches
Maximum Slump	3 inches

The general mixture proportions used in the laboratory and field study are presented in Table 3.4. There are a couple key differences between these two studies. As stated in Section 3.2.3, air entrainment was not used in the laboratory study and therefore only entrapped air would be present in the laboratory mixtures. This was accounted for by reducing the air factor from 5 percent to 2 percent. The other main modification was the reduction of water used in the laboratory mixtures in comparison to the mixture proportions used in the field. This was done because the water specified in the field mixture design is a maximum value allowed to ensure the necessary strength gain, and typically the in-place concrete will have less water present than the actual mixture design states. Also, the water content was reduced so that all mixtures would meet slump requirements even if no water reducer was present. This provided a constant water/cementitious (w/cm) ratio between mixtures and allowed water reducer to be added as needed to ensure good workability independent of the amount of fiber added to the mixture.

Table 3.4: General Mixture Proportions for Laboratory and Field Studies

Parameter	Field Study	Laboratory Study
Cement Factor	6 Sacks/CY (564 pcy)	6 Sacks/CY (564 pcy)
Coarse Agg. Factor	0.72	0.72
Water Factor	4.8 Gal/Sack (0.43 w/cm)	4.5 Gal/Sack (0.40 w/cm)
Air Factor	5%	2%
Fly Ash	20%	25%

The fiber dosages chosen for the lab study reflect the need to find an economically feasible way to use fibers in CRCP. Since the cost of pavement construction in Texas is so dependent on material costs, only low-dosage fiber reinforcement was included in this study. The goal of using low dosages is to find which fiber provides the largest improvement in spalling performance at the lowest cost. The actual dosage values were chosen to be compatible with current construction practices. This was accomplished by choosing a dosage of fibers that would be capable of being added to a typical 10 cubic yard concrete mixture in full-bag increments. For example, SF1 comes in 50 lb bags, therefore the number of bags necessary to achieve 25 lbs per cu yd of fibers in a 10 cu yd mixture would be 5 bags, or 250 lbs.

A summary of the complete mixture proportions used in this laboratory evaluation, including the mixture designations that will be used throughout this chapter, is provided in Table 3.5.

Table 3.5: Final Mixture Proportions for Lab Study

Mixture	Mixture Designation*	Water (pcy)	Type A/D Reducer/ Retarder (oz/cy)	Type I/II Cement (pcy)	Class C Fly Ash (pcy)	Fine Aggregate (pcy)	Coarse Aggregate**		Fiber Reinforcement	
							Type	Amount (pcy)	Type	Amount (pcy)
1	SRG-Control	225	6.6	429	120	1,341	SRG	1,905	-	-
2	SRG-SF1-25	225	10.0	429	120	1,341	SRG	1,905	SF1	25
3	SRG-SF1-40	225	13.3	429	120	1,341	SRG	1,905	SF1	40
4	SRG-SF2-27.5	225	19.9	429	120	1,341	SRG	1,905	SF2	27.5
5	SRG-SnF1-4	225	18.3	429	120	1,341	SRG	1,905	SnF1	4
6	SRG-SnF1-6	225	24.9	429	120	1,341	SRG	1,905	SnF1	6
7	SRG-SnF2-1.5	225	10.0	429	120	1,341	SRG	1,905	SnF2	1.5
8	LS-Control	225	13.3	429	120	1,341	LS	1,913	-	-
9	LS-SF1-25	225	24.9	429	120	1,341	LS	1,913	SF1	25
10	LS-SF1-40	225	33.2	429	120	1,341	LS	1,913	SF1	40
11	LS-SF2-27.5	225	19.9	429	120	1,341	LS	1,913	SF2	27.5
12	LS-SnF1-4	225	24.9	429	120	1,341	LS	1,913	SnF1	4
13	LS-SnF1-6	225	41.5	429	120	1,341	LS	1,913	SnF1	6
14	LS-SnF2-1.5	225	24.9	429	120	1,341	LS	1,913	SnF2	1.5

* Mixture Designation reported as follows: Aggregate type – Fiber Type – Fiber Dosage (pcy)

** SRG = Siliceous River Gravel, LS = Crushed Limestone

3.4 Testing Procedures

A comprehensive lab evaluation was conducted to understand the differences between siliceous river gravel and crushed limestone and how various fiber types and dosages change the parameters that are important to the design of CRCP. The following sections discuss the testing procedures that were implemented to evaluate the fresh and hardened properties of these materials.

3.4.1 Fresh Concrete Properties

Typical fresh property tests were conducted on all laboratory mixtures for quality control of the specimens and to ensure the desired workability had been achieved. All tests performed were in accordance with ASTM specifications and are provided in Table 3.6. The tests listed were conducted on each of the 14 mixtures produced and were performed immediately after mixing had been completed.

Table 3.6: Test Methods for Fresh Properties

Test Number	Description
ASTM C 143	Standard Test Method for Slump
ASTM C 138	Standard Test Method for Air Content
ASTM C 138	Standard Test Method for Unit Weight

3.4.2 Hardened Concrete Properties

A comprehensive evaluation of hardened concrete properties is critical to understanding how fiber reinforcement and aggregates impact CRCP performance. All hardened properties were tested in accordance with ASTM specifications and are summarized in Table 3.7. All specimens tested were moist-cured until testing, as specified by ASTM.

Table 3.7: Test Methods for Hardened Properties

Test Number	Description
ASTM C 39	Compressive Strength of Concrete
ATM C 496	Splitting Tensile Strength of Concrete
ASTM C 469	Modulus of Elasticity of Concrete
ASTM C 78	Flexural Strength
ASTM C 1018	Flexural Toughness

3.4.3 Compressive Strength, Elastic Modulus, and Splitting Tensile Strength

For each mixture, fourteen 6-in. x 12-in. cylinders were cast with seven specimens tested at 7 days, and the remaining seven specimens were tested at 28 days. For each specified day of testing, three specimens were used for splitting tensile strength testing and the remaining four specimens were tested for compressive strength and modulus of elasticity.

Of the four specimens used for compression and modulus testing, two specimens were initially failed in compression to determine the average compressive strength (f'c) of the given mixture. The data was used to determine the upper threshold of the linear-elastic range of the stress-strain curve (0.4 f'c), which allowed the modulus of elasticity to be properly determined. The two remaining specimens were then tested for modulus a minimum of two times or until consistent modulus values could be obtained. After the modulus was reliably determined, the specimen was loaded in compression until failure. Therefore, four specimens were tested for compressive strength and two specimens were tested for modulus of elasticity. All specimens tested for compressive strength and/or modulus were sulfur-capped prior to testing to achieve accurate results.

Since fiber reinforcement can enable a specimen to carry load after failure has already taken place, extra care was taken in determining splitting tensile strength. The splitting tensile strength for a given specimen was determined as the point in which a load carrying instability first occurs due to the initiation of cracking. Any load carried by the specimen after this initial cracking was considered extraneous because it is more synonymous with toughness than an actual increase in tensile strength.

3.4.4 Flexural Toughness and Flexural Strength

For each mixture produced, flexural toughness testing was conducted on three 6-in. x 6-in. x 20-in. prisms using a closed-loop, deflection-controlled testing system developed at The University of Texas at Austin. All mixtures were tested at 35 days to ensure that the concrete had achieved its full strength and to provide consistency between mixtures. An image of the test apparatus is presented in Figure 3.5. A Japanese yoke was used to mount LVDTs on both sides of the prism in order to monitor the net mid-span deflections and provide closed-loop control. This method of deflection control ensures extraneous deflections due to seating of the specimen and/or frame deflections are not included in the deflection monitoring. An illustration of this apparatus is provided in Figure 3.6 and an image of the deflection monitoring device is presented in Figure 3.7.

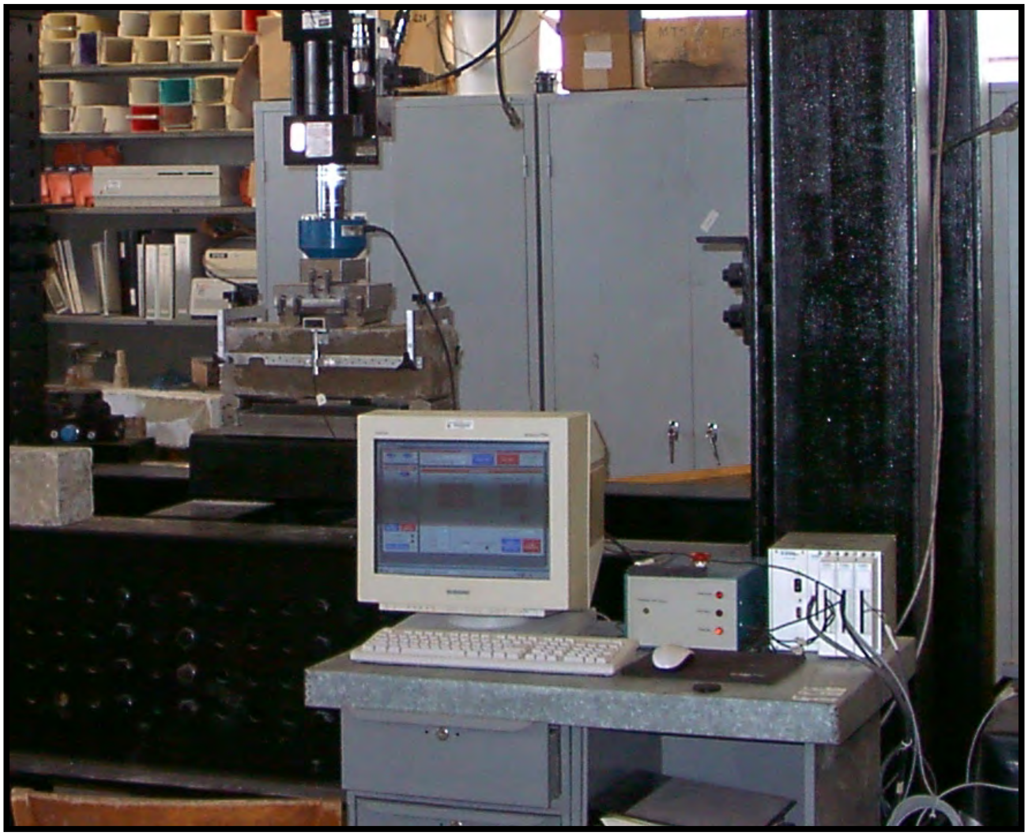


Figure 3.5: Flexural Toughness Test Setup

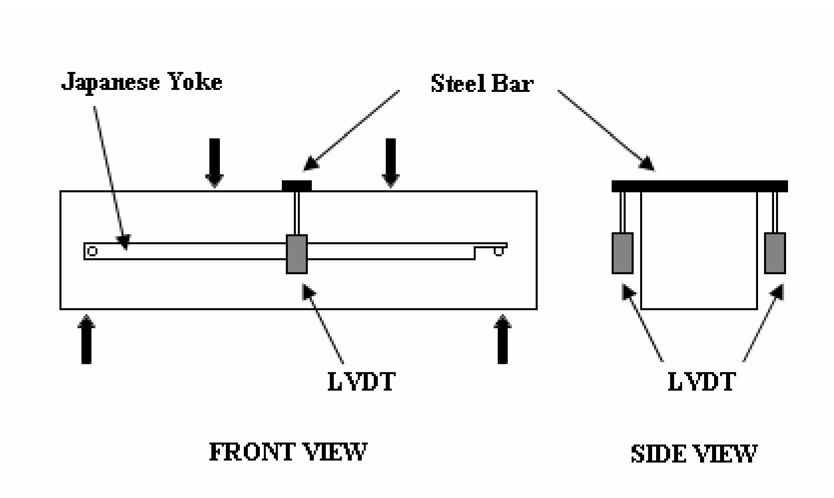


Figure 3.6: Illustration of Deflection Monitoring for Flexural Toughness Test

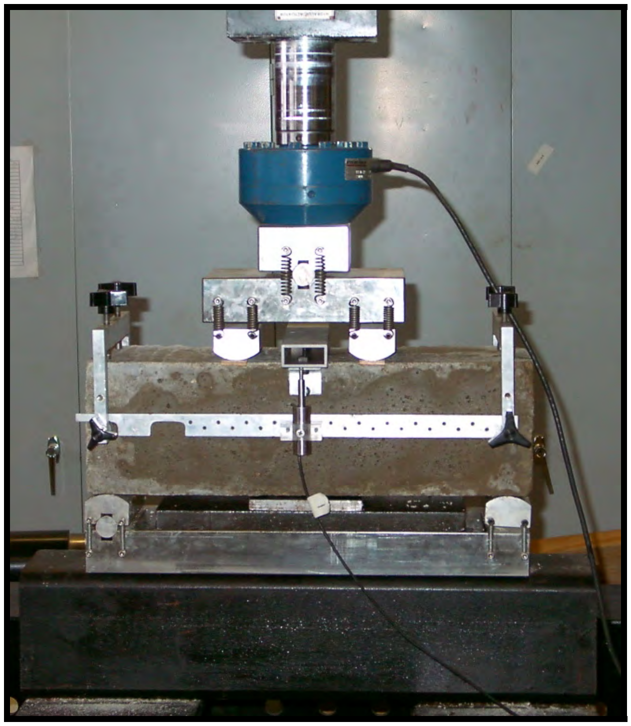


Figure 3.7: Flexural Toughness Test in Progress

Both the data acquisition and the closed-loop control were handled by a single Labview program developed at UT Austin for this project. Raw analog data was acquired by I/O boards (National Instruments Corporation, Austin, TX) and then processed by the Labview software. The software's user-interface was designed to provide control of all aspects of the testing system, as well as to provide the user with the real-time data being acquired. An image of the software's interface can be found in Figure 3.8. The control system monitored the deflection rate of the specimen at a sampling rate of 2500–3500 Hz to provide adequate deflection control when the prism experienced instabilities due to the initiation of flexural cracking. Load and mid-span deflection data was acquired throughout the duration of the test at a sampling rate of approximately two times per second. The acquired data was presented in the software's user interface as a load-deflection plot and was also recorded into a text output file. This output file was later manipulated using a spreadsheet to construct the load-deflection plots required to determine various flexural toughness parameters. These parameters will be discussed in greater detail in the results section of this chapter.

The flexural strength of each mixture was determined using the peak load determined from the flexural toughness testing. Even though ASTM C78 requires a load-controlled test, using data obtained from ASTM C1018 is acceptable. Because concrete is initially linear-elastic when tested in flexure, a deflection-controlled test still provides a constant loading rate and therefore makes both control methods valid for flexural strength testing.

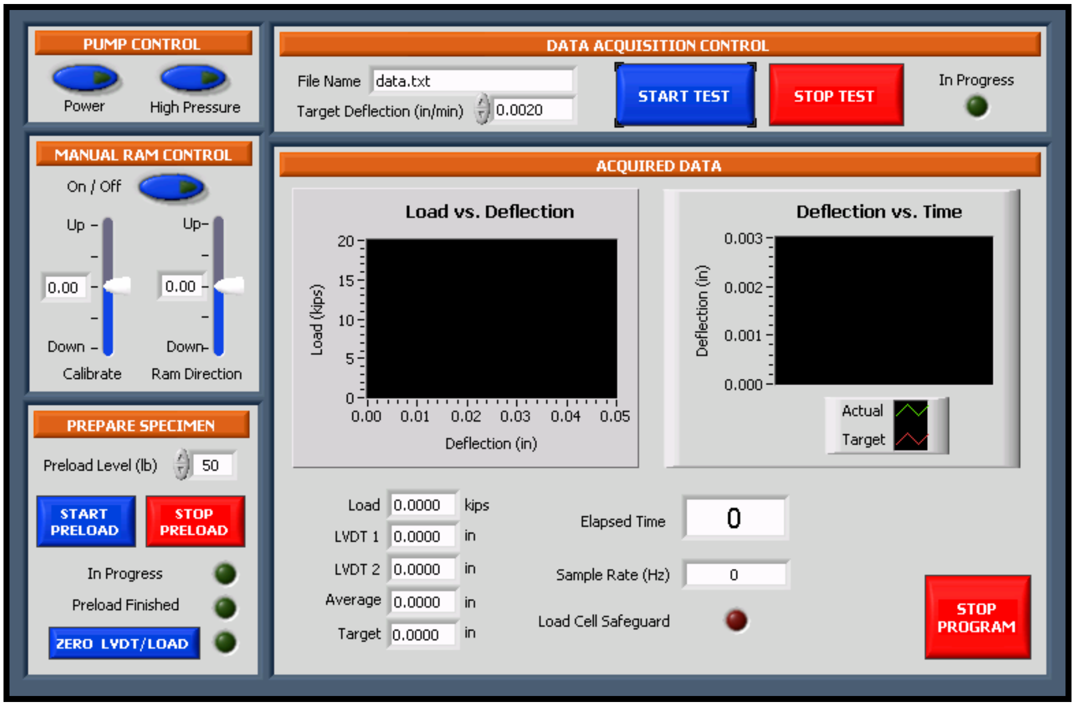


Figure 3.8: Screen Capture of Flexural Toughness Test Control Program

3.5 Testing Results and Discussion

The following sections present the results of the laboratory evaluation of fiber-reinforced concrete (FRC) and provide insight into the impacts fiber reinforcement have on concrete utilizing typical CRCP mixture proportions. Inherent differences between FRC containing crushed limestone and FRC containing siliceous river gravel will also be evaluated.

3.5.1 Fresh Concrete Properties

Understanding the effects of fiber reinforcement on fresh concrete properties is critical to CRCP construction. Since CRCP is typically constructed using a slipform paving machine, the concrete must be stiff enough to maintain the desired shape without traditional formwork but also workable enough to provide good consolidation and finishability. Fiber reinforcement is known to have significant impacts on workability, which is why it is important to quantify the impacts of various fiber types and dosages on CRCP mixture proportions and determine the necessary changes required to make construction of CRCP with fiber reinforcement feasible. Table 3.8 summarizes the fresh properties evaluated for each of the fourteen mixtures produced for this study. The following sections investigate each fresh property in greater detail to provide a better understanding of the impacts of fiber reinforcement on pavement properties.

Table 3.8: Summary of Fresh Concrete Properties

Mixture	Mixture Designation	Type A/D Reducer/ Retarder (oz/cy)	Concrete Temp. (ºF)	Slump (in.)	Air Content (%)	Unit Weight (lb/yd3)
1	SRG-Control	6.6	74	1.25	2.25	148.68
2	SRG-SF1-25	10.0	74	1.75	2.00	148.76
3	SRG-SF1-40	13.3	74	1.25	2.25	148.44
4	SRG-SF2-27.5	19.9	74	2.50	2.50	149.04
5	SRG-SnF1-4	18.3	74	1.25	2.50	144.60
6	SRG-SnF1-6	24.9	74	0.75	2.75	144.88
7	SRG-SnF2-1.5	10.0	74	0.75	2.75	146.68
SRG Average		14.7	74	1.25	2.50	147.30
8	LS-Control	13.3	74	1.75	3.00	147.92
9	LS-SF1-25	24.9	74	1.25	2.75	149.64
10	LS-SF1-40	33.2	75	1.50	2.75	149.52
11	LS-SF2-27.5	19.9	74	1.00	3.00	149.64
12	LS-SnF1-4	24.9	75	0.50	2.50	150.36
13	LS-SnF1-6	41.5	74	1.00	3.00	148.48
14	LS-SnF2-1.5	24.9	75	2.50	2.75	149.08
LS Average		26.1	74	1.25	2.75	149.23

3.5.2 Slump

The target slump for the lab mixtures was 1.5 to 3 in. as prescribed by TxDOT specifications for CRCP construction. Since the w/cm ratio was held constant for all mixtures in order to ensure consistency, the amount of water reducer was varied in order to obtain the desired slump. The slump values observed were typically lower than desired, despite the use of water reducer. This phenomenon can be attributed to a variety of variables. The addition of fibers and relatively high percentage of sand increased the surface area and therefore increased the water demand of the mixture. Water demand was also increased when crushed limestone was used as the coarse aggregate due its higher absorption capacity compared to siliceous river gravel. By increasing the water demand and utilizing a relatively low w/cm ratio (0.40), achieving the desired slump using these proportions becomes difficult. This problem can be alleviated in the future by using a slightly higher w/cm ratio and/or the use of a midrange water reducer instead of the low-range water reducer used in the study.

It is also important to recognize that fiber reinforcement will typically reduce the apparent slump of concrete even though the actual workability will remain the same. This phenomenon was most apparent containing SnF1 because the fiber count was much higher than the mixtures containing steel fibers.

It should be noted that care was taken to ensure proper consolidation of all specimens made despite the low slump values observed. Any mixture having a slump of less than 1 in. was mechanically vibrated to ensure reliable results when the specimens were tested.

3.5.3 Air Content

Because there were no air-entraining admixtures used in the mixtures evaluated, the expected entrapped air content was 2 to 3 percent. The air content values documented were found to support this by yielding 2 to 3 percent air for all of the mixtures tested. The average air content was 2.75 percent and there was no observed impact due to fiber reinforcement or coarse aggregate type.

3.5.4 Unit Weight

Unit weights were measured using a typical 0.25 cu ft unit weight bucket. Typical concrete has a unit weight between 145-150 lbs per cu ft, and the mixtures prepared for this study were calculated to be approximately 149 lbs per cu ft. As shown in Table 3.9, the unit weights documented were similar to the values expected, and there were no measured effects due to the addition of fiber reinforcement.

3.5.5 Hardened Concrete Properties

Evaluating the effects of fiber reinforcement on hardened concrete properties is crucial to understanding the potential of fiber reinforcement for improving long-term pavement performance. As discussed in Chapter 2, the effects of fibers only become apparent once cracking is initiated, and therefore the effect of fibers on most hardened properties is expected to be minimal. Flexural toughness is the property of greatest significance because it quantifies the load carrying potential of concrete after the initiation of cracking. While toughness is not a typical design parameter for CRCP, its inclusion does provide great insight since CRCP performance is directly related to minimizing crack widths and maintaining load transfer capability across cracks.

3.5.6 Compressive Strength

It has been well documented that fiber reinforcement typically has an insignificant impact on the compressive strength of concrete because fiber additions only become relevant after cracking has been initiated. This concept was verified by the compressive strength testing conducted for this study. As shown in Table 3.9, no significant correlation can be made between the addition of fibers and compressive strength.

One observation that was not expected in this study was the generally high compressive strength recorded for all of the mixtures. Concrete used for CRCP construction typically yields 28-day strengths of approximately 5000 psi, and the strengths recorded in this study were significantly higher. This phenomenon can be attributed to a water-to-cement ratio that was slightly lower than typical CRCP mixtures and the increase in water demand due to the addition of fine aggregate to account for the reduction in air content. In addition, this study used a continuous moist-curing regime, which is not typical of actual pavements.

The type of coarse aggregate used has typically been shown to have an impact on concrete strength, which was verified by this study. All mixtures containing crushed limestone were found to have higher compressive strengths than siliceous river gravel, independent of fiber type and dosage. The failure mode of mixtures containing the various coarse aggregate differed as well. Mixtures containing siliceous river gravel failed only in the mortar, while mixtures containing crushed limestone had more abrupt failures by shearing through the aggregate. Shearing of aggregate would most likely not occur in typical pavements because compressive strengths of concrete in pavements are normally less than what was observed in this study.

Table 3.9: Summary of Compressive Strength Test Results

Mixture	Mixture Designation	Average Compressive Strength (psi)	
		7-day	28-day
1	SRG-Control	5,420	7,080
2	SRG-SF1-25	5,200	6,780
3	SRG-SF1-40	5,320	7,080
4	SRG-SF2-27.5	5,450	7,080
5	SRG-SnF1-4	5,090	6,900
6	SRG-SnF1-6	5,380	7,150
7	SRG-SnF2-1.5	4,670	6,430
SRG Average		5,220	6,930
8	LS-Control	6,160	7,520
9	LS-SF1-25	6,330	7,820
10	LS-SF1-40	6,470	7,920
11	LS-SF2-27.5	6,230	7,690
12	LS-SnF1-4	5,880	8,180
13	LS-SnF1-6	6,290	8,150
14	LS-SnF2-1.5	6,140	7,550
LS Average		6,210	7,830

3.5.7 Elastic Modulus

While the actual cause of poor performance of siliceous river gravel in CRCP has not been completely isolated, many believe that the elastic modulus of concrete could be one of the key variables. Siliceous river gravel is typically a much stiffer aggregate than crushed limestone. This additional stiffness can lead to expedited degradation of cracks and an increase in the occurrence and severity of spalling in CRCP. A higher elastic modulus from the aggregate will also translate into higher stresses due to thermal and/or drying shrinkage.

Table 3.10 summarizes the elastic modulus testing conducted during this study. As expected, mixtures containing siliceous river gravel had a consistently higher modulus than mixtures containing crushed limestone. However, the relative difference in elastic modulus between the two aggregate types was not as significant as expected. This can be attributed to the unexpected higher compressive strength of the mixtures produced for the study. A more significant deviation in elastic modulus would be present in the typical lower-strength concrete used for pavements.

The impact of fiber reinforcement on elastic modulus was much less significant than the coarse aggregate used in the mixture. Variations in elastic modulus between mixtures were proportional to the variations observed in the compressive strength of each mixture. Therefore there is no real effect on elastic modulus due to the addition of fiber reinforcement.

Table 3.10: Summary of Elastic Modulus Test Results

Mixture	Mixture Designation	Average Elastic Modulus x 10^3 (psi)	
		7-day	28-day
1	SRG-Control	5,700	6,450
2	SRG-SF1-25	5,650	6,500
3	SRG-SF1-40	5,450	6,350
4	SRG-SF2-27.5	5,350	6,000
5	SRG-SnF1-4	5,350	6,150
6	SRG-SnF1-6	5,550	5,900
7	SRG-SnF2-1.5	5,400	5,800
SRG Average		5,500	6,150
8	LS-Control	5,000	5,450
9	LS-SF1-25	5,250	5,800
10	LS-SF1-40	5,150	5,550
11	LS-SF2-27.5	4,850	5,550
12	LS-SnF1-4	5,200	5,650
13	LS-SnF1-6	4,950	5,550
14	LS-SnF2-1.5	5,100	5,650
LS Average		5,100	5,600

3.5.8 Splitting Tensile Strength

Table 3.11 summarizes the results of the splitting tensile strength tests conducted for this laboratory study. This method of determining tensile strength was not an extremely reliable method because typically it was difficult to determine the load at which the specimen first cracked. This was especially true for specimens containing steel fibers. Often the specimens would reach a load at which cracking initiated, but the fibers would allow for additional load carrying capacity even though the specimen had already reached *failure* in tension. Care was taken to minimize these effects, but often they could not be avoided. Overall, the effect of fibers and/or coarse aggregate type on splitting tensile strength was minimal.

Table 3.11: Summary of Splitting Tensile Strength Test Results

Mixture	Mixture Designation	Average Splitting Tensile Strength (psi)	
		7-day	28-day
1	SRG-Control	555	625
2	SRG-SF1-25	615	640
3	SRG-SF1-40	685	640
4	SRG-SF2-27.5	620	645
5	SRG-SnF1-4	525	600
6	SRG-SnF1-6	545	620
7	SRG-SnF2-1.5	495	595
SRG Average		575	625
8	LS-Control	485	645
9	LS-SF1-25	560	625
10	LS-SF1-40	555	690
11	LS-SF2-27.5	560	660
12	LS-SnF1-4	555	620
13	LS-SnF1-6	625	725
14	LS-SnF2-1.5	540	710
LS Average		555	670

3.5.9 Flexural Strength

Table 3.12 summarizes the results of the flexural strength testing. As described previously, fibers have been found to have a minor impact on concrete prior to cracking, and therefore it was expected that fibers would not change flexural strength values. While the results did show some degree of variation between mixtures, there was no specific correlation between fiber type and/or dosage and flexural strength. Any observed differences were associated with typical statistical variability. Siliceous river gravel mixtures generally had higher flexural strengths than mixtures containing crushed limestone despite the fact limestone provided greater compressive strengths. This effect was most likely due to the more severe shearing failure of the limestone itself as opposed to the more gradual failure of the concrete matrix around the siliceous river gravel.

Table 3.12: Summary of Flexural Strength Test Results

Mixture	Mixture Designation	Average Peak Load (lb)	Average Flexural Strength (psi)
1	SRG-Control	11,624	970
2	SRG-SF1-25	11,670	970
3	SRG-SF1-40	11,904	990
4	SRG-SF2-27.5	12,021	1,000
5	SRG-SnF1-4	10,871	905
6	SRG-SnF1-6	10,734	895
7	SRG-SnF2-1.5	10,315	860
SRG Average		11,305	940
8	LS-Control	10,949	910
9	LS-SF1-25	11,038	920
10	LS-SF1-40	10,671	890
11	LS-SF2-27.5	10,963	915
12	LS-SnF1-4	11,485	955
13	LS-SnF1-6	10,468	870
14	LS-SnF2-1.5	10,896	910
LS Average		10,924	910

3.5.10 Flexural Toughness

Flexural toughness is the key parameter for understanding the impact fiber reinforcement will have on hardened concrete and, more specifically, CRCP performance. Even though flexural toughness is not a design parameter, it will provide useful data because improving load transfer across cracks is critical to ensuring good pavement performance. By maintaining load transfer across cracks, distresses in CRCP can be reduced and a longer service life can be achieved.

The results from the flexural toughness testing are presented both qualitatively and quantitatively in the following figures and tables. Figure 3.9 and Figure 3.10 summarize the load-deflection data used to determine the toughness indices (I) and residual strength factors (R) needed to quantitatively evaluate flexural toughness. These calculated values of toughness and residual strength can be found in Table 3.13.

Various conclusions can be made from the data obtained from this testing. All fiber types and dosages improved toughness when compared to the control specimens, but the degree of improvement varied significantly. SF1-40 produced significantly higher levels of toughness and residual strength than the rest of the fiber types and dosages. For most current applications, SF1 is used in larger quantities than it was in this study; it therefore has the potential to make very significant improvements if costs are not a concern. SnF1-6, which is near the maximum useable dosage for this fiber type, offered performance similar to that of the lower dosages of SF1 and SF2. Surprisingly, SF2-27.5 performed well at low levels of deflection, but once deflections were increased, its ability to carry load dropped significantly. This can be attributed to excessive pull-out of the individual fibers and suggests fiber geometry can have a great impact on toughness. Overall, steel fibers provided greater improvements in toughness and residual strength than synthetics. The improvements in toughness and residual strength due to the various fiber types and dosages tested are graphically presented in Figure 3.11 through Figure 3.14.

The effect of fiber reinforcement was also dependent on the type of coarse aggregate used. Mixtures containing fibers in conjunction with siliceous river gravel had greater improvements in toughness than the same mixture containing crushed limestone. This is not surprising because the failure mode of concrete containing limestone failed the matrix, as well as shearing the rock itself. Mixtures containing river gravel failed around the aggregate, allowing greater stability in the specimen because of aggregate interlock. It can be hypothesized that if strength of the mortar was decreased such that the failure mechanism was similar for both aggregates, the variation in toughness between the two aggregate types would be minimized. This is important to recognize since typical CRCP mixtures are usually lower in strength than what was observed in the laboratory study.

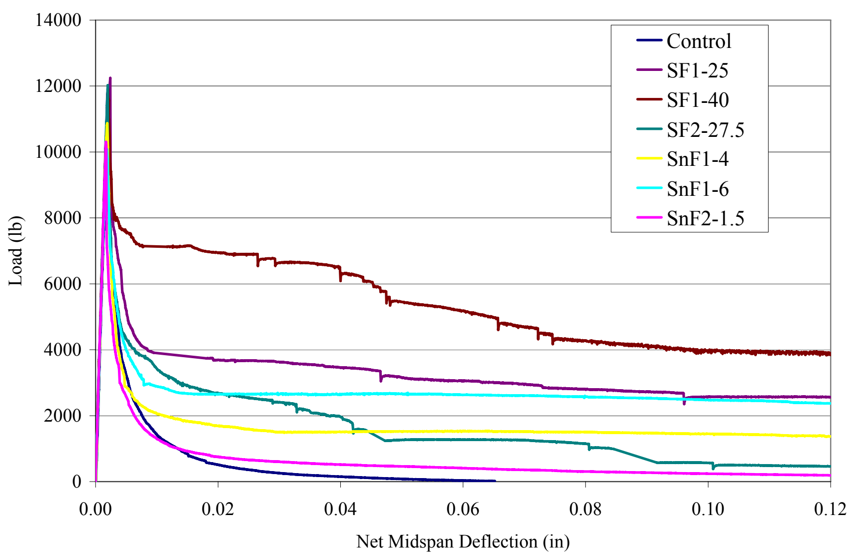


Figure 3.9: Summary of Load-Deflection Data (SRG)

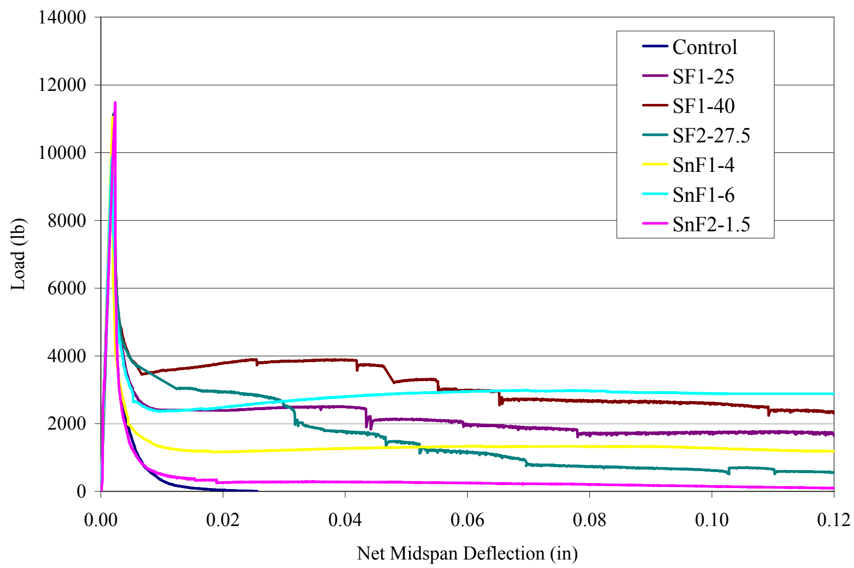


Figure 3.10: Summary of Load-Deflection Data (LS)

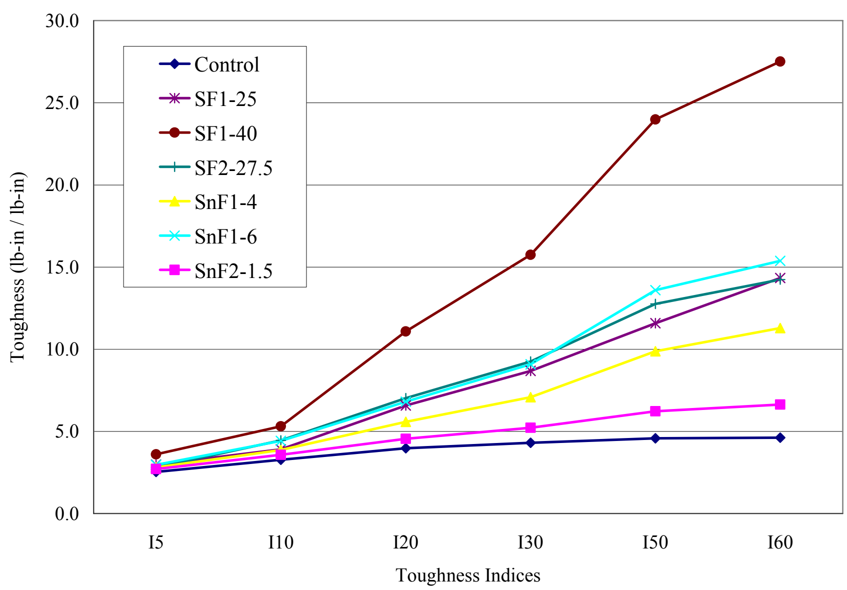


Figure 3.11: Effects of Fibers on Toughness (SRG)

Table 3.13: Summary of Flexural Toughness Test Results

Mixture	Mixture Designation	Toughness Indices						Residual Strength Factors				
		I_5	I_{10}	I_{20}	I_{30}	I_{50}	I_{60}	$R_{5,10}$	$R_{10,20}$	$R_{20,30}$	$R_{30,60}$	$R_{10,50}$
1	SRG-Control	2.5	3.3	4.0	4.3	4.6	4.6	14.7	7.0	3.3	1.1	3.3
2	SRG-SF1-25	3.0	3.9	6.6	8.7	11.6	14.3	18.5	26.7	21.1	18.8	19.2
3	SRG-SF1-40	3.6	5.3	11.1	15.7	24.0	27.5	34.2	57.8	46.6	39.2	46.7
4	SRG-SF2-27.5	2.8	4.5	7.0	9.3	12.8	14.2	32.9	25.7	22.3	16.6	20.8
5	SRG-SnF1-4	2.8	3.9	5.6	7.1	9.9	11.3	21.2	17.1	14.9	14.0	15.0
6	SRG-SnF1-6	3.0	4.4	6.8	9.1	13.6	15.4	29.3	23.9	22.7	21.0	22.9
7	SRG-SnF2-1.5	2.7	3.6	4.6	5.2	6.2	6.6	17.1	9.8	6.7	4.7	6.6
SRG Average		2.9	4.1	6.5	8.5	11.8	13.4	24.0	24.0	19.7	16.5	19.2
8	LS-Control	2.2	2.4	2.5	2.6	2.6	2.6	5.5	1.0	0.2	0.0	0.3
9	LS-SF1-25	2.5	3.4	5.5	7.2	11.1	12.8	18.2	21.5	17.1	18.6	19.4
10	LS-SF1-40	2.9	4.5	8.4	12.1	18.9	21.8	33.5	38.4	37.0	32.3	35.9
11	LS-SF2-27.5	2.5	3.8	6.0	8.1	11.2	12.3	26.0	22.9	20.6	14.0	18.7
12	LS-SnF1-4	2.1	2.7	3.9	5.1	7.7	9.0	13.6	11.8	12.1	12.9	12.3
13	LS-SnF1-6	2.4	3.4	5.5	7.7	12.5	15.0	20.7	20.8	22.3	24.1	22.7
14	LS-SnF2-1.5	1.8	2.1	2.4	2.8	3.3	3.5	5.7	3.4	3.0	2.5	3.0
LS Average		2.3	3.2	4.9	6.5	9.6	11.0	17.6	17.1	16.0	14.9	16.0

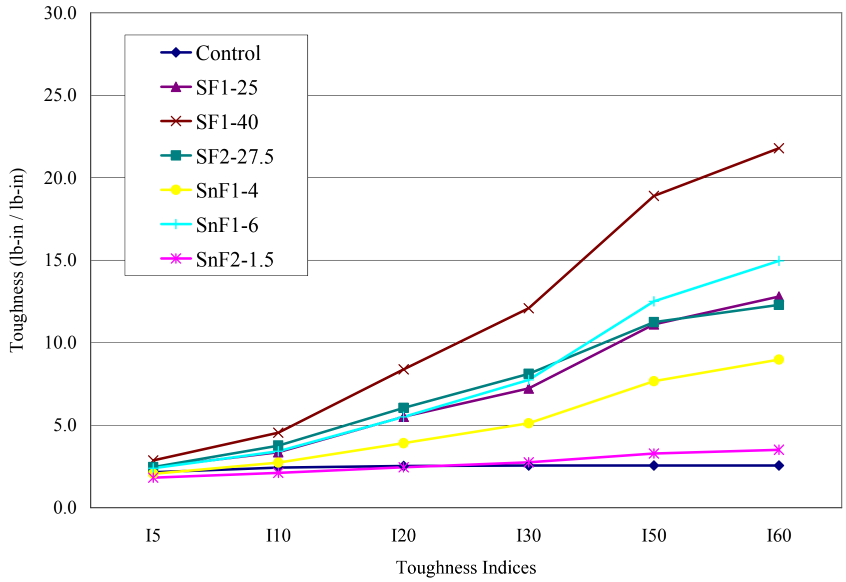


Figure 3.12: Effects of Fibers on Toughness (LS)

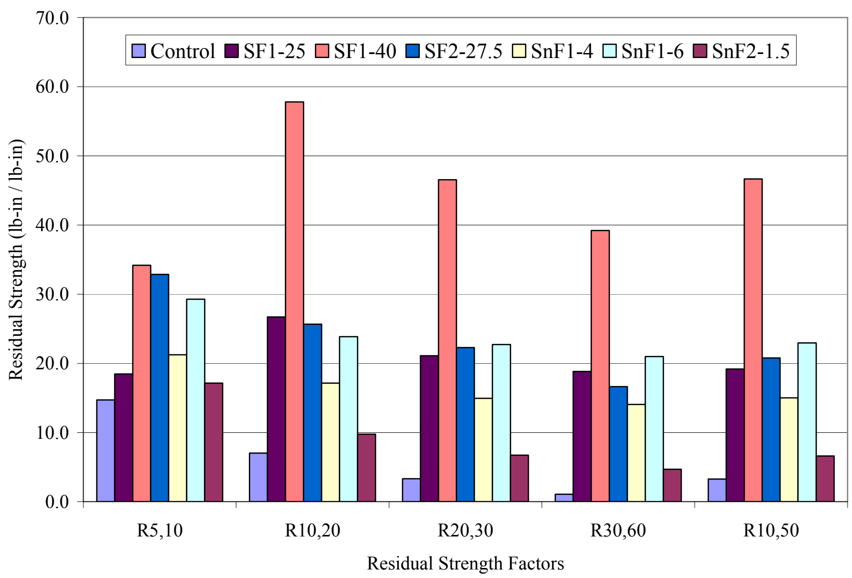


Figure 3.13: Effects of Fibers on Residual Strength (SRG)

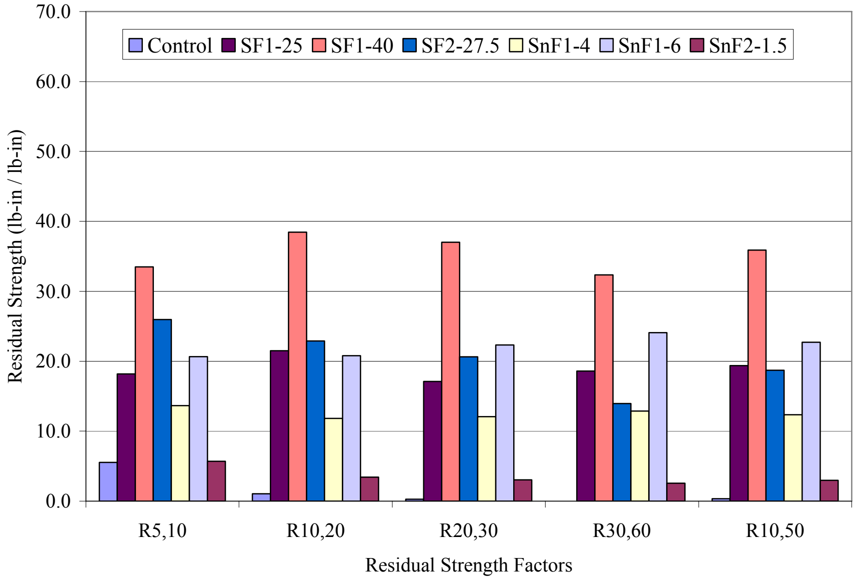


Figure 3.14: Effects of Fibers on Residual Strength (LS)

3.6 Summary

The impact of fiber reinforcement on typical fresh and hardened concrete properties has been found to correlate well with the previous work presented in Chapter 2. Fibers can have a significant effect on workability and flexural toughness, but for most parameters they provide little or no significant difference when compared to concrete containing no fibers. This means no significant design changes are required in order to implement fiber reinforcement for use in CRCP. The main change that will be required is the use of additional admixtures (i.e., water reducers) to ensure good workability of the mixture since fiber reinforcement can impact slump and water demand.

This study has also determined that the choice of fiber type and dosage has a significant effect on the post-cracking behavior of concrete. Steel fibers typically provide greater improvements in toughness and residual strength than synthetic fibers, and both parameters are proportional to dosage rate for any fiber used. Toughness and residual strength should be good indicators of improved spalling performance of CRCP, but field evaluations of CRCP containing fibers will be critical for verifying this hypothesized correlation. Once an adequate database of the long-term performance of fiber-reinforced CRCP has been constructed, trends can be formulated relating toughness and residual strength to various CRCP performance parameters including, crack width, crack spacing, and spalling performance.

Chapter 4. Field Evaluation 1

4.1 Overview

The main purpose of this preliminary field evaluation is to document the effects of fibers in continuously reinforced concrete pavements (CRCP). A frontage road was selected for the preliminary field study so that the fibers could be tested on a small scale rather than subjecting a mainline paving job to potential difficulties. The pavement was constructed by hand using a concrete spreader. Construction began at 6:00 a.m. and ended at 3:30 p.m. on August 2, 2002. It is located on the northbound frontage road on the inside lane of the Kirkwood Drive Exit on Highway 59, just southwest of Houston, TX. The location of the field site is shown in Figure 4.1 and an overall view is portrayed in Figure 4.2. This location was selected due to the relatively high truck traffic and the times of lower traffic flow, which enabled flexibility with monitoring. Houston has a history of poorly performing pavements due to the environmental conditions and aggregate sources, and therefore it provides an excellent basis for evaluating the benefits of fibers.

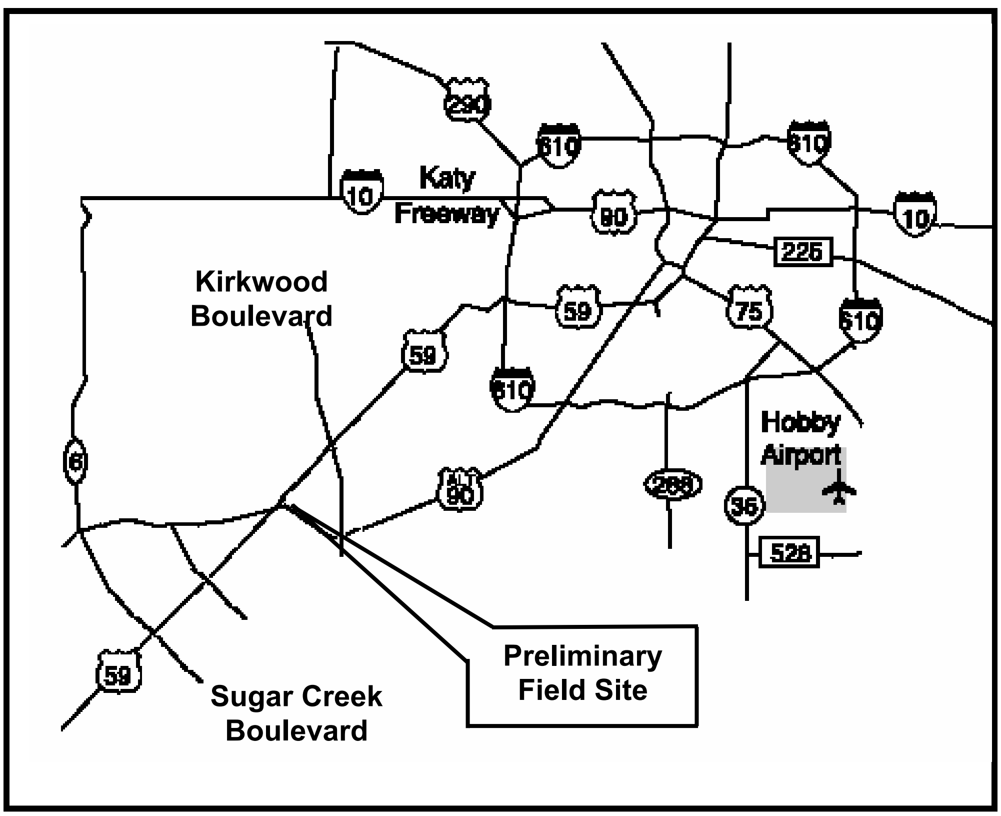


Figure 4.1: Map of Preliminary Field Site (Otero-Jimenez et al., 1992)

Figure 4.2: Overall View of Field Site

The concrete pavement has typical properties for a frontage road. It is 10 in. thick, 10 ft wide, and has a total length of 1270 ft. The final 62 ft are not considered in this evaluation because of special curing testing performed by TxDOT. It should be noted that there has been no pavement constructed at the beginning of the field site, reducing the amount of restraint in the first section. The primary reinforcement is Grade 60 reinforcing steel that was placed at the mid-depth of the pavement. For the longitudinal direction, the reinforcement consists of No. 6 bars spaced at 8.25 inches. For the transverse reinforcement, No. 5 bars are spaced at 36 inches. Since the addition of fibers is not intended to increase the design strength of the pavement but merely to enhance its durability, the existing reinforcement and thickness design for the CRCP test section was not changed. This allowed the contractor to construct the pavements as originally bid, making the only expense for implementation the material cost of fibers.

One of the main objectives of the field evaluation is to verify the impact that fibers have on the workability and constructability of paving operations. This is imperative since CRCP has never incorporated fibers. The main areas of concern with workability include batching, mixing, transporting, and paving. It is important to verify that the addition of fibers has no effect on the transporting and paving methods used during the actual construction. Finishing, tining, and curing are also issues that were investigated.

The field evaluation was also being used to evaluate general CRCP distress mechanisms, particularly spalling. Spalling causes the highest number of repairs in CRCP in Texas. It is difficult to simulate because its mechanism is not fully understood. The material selection for the field study was devoted to producing conditions that encourage spalling. However, the materials and mixture proportions are still representative for a typical paving job. It is well-documented that pavements constructed with siliceous river gravel tend to cause spalling, particularly in the Houston area. Consequently, the field study was constructed with this type of aggregate.
The preliminary field study is also being utilized to evaluate the pavement's crack spacing, crack width, and general behavior to see if fibers can help to prevent spalling. Three different types of

fibers were selected for this study. These include one steel and two synthetic fibers that have varying geometries and anchorage types. These fibers were selected based on documented research, economic feasibility, and manufacturer recommendations. A high and low volume dosage was chosen for each fiber to determine the minimum dosage that could be effective.

4.2 Materials

The materials selected for the field study are typical for CRCP containing siliceous river gravel. A river gravel pavement was chosen since it has been shown to exhibit spalling at an accelerated rate. This type of pavement is also typical for the Houston area and will provide a realistic comparison for each fiber type and dosage. This section discusses the different types of materials that are utilized in this study.

4.2.1 Cementitious Material

The characteristics for both the cement and fly ash are shown in Table 4.1. The cement is classified as Type I/II, as specified by ASTM C 150. The fly ash is considered a Class C fly ash that conforms to ASTM C 618. These are the conventional materials used by the concrete supplier for CRCP containing siliceous river gravel.

Table 4.1: Cement and Fly Ash Chemistry

Chemical Analysis	Type I/II Cement (%)	Class C Fly Ash (%)
SiO	20.1	-
SiO_2	-	32.44
Al_2O_3	5.0	19.06
Fe_2O_3	3.7	6.83
CaO	64.0	28.26
MgO	0.6	4.15
SO_3	-	2.14
Na_2O	NA	1.46
Loss on Ignition	1.3	0.10
Free Lime	0.9	-
Moisture Content	0.14	0.11
Physical Analysis		
Fineness	95.6	17.0
Specific Gravity	-	2.70
Autoclave Expansion	0.00	0.04
Potential Compounds		
C_3S	59.0	-
C_3A	7.1	-

4.2.2 Aggregates

The type of coarse aggregate has been shown to be the most influential variable in CRCP performance. The two main aggregate sources for Texas are crushed limestone and siliceous river gravel (see Sections 2.2.3 and 2.19). Field performance has shown that pavements constructed with crushed limestone generally perform much better than those constructed with siliceous river gravel. This has mainly been attributed to the river gravel's high coefficient of thermal expansion, elastic modulus, and lack of bond. The field study was constructed with siliceous river gravel because this is the type of pavement that could benefit most from the addition of fibers. The river gravel is a Grade 2 aggregate that conforms to ASTM C 33 (Size 467). The sand is natural river sand that conforms to ASTM C 33 and Item 421 in the TxDOT Specifications. Table 4.2 lists the properties for each aggregate used in the field study.

Table 4.2: Aggregate Properties

Property	Sieve Size	Coarse Aggregate (%)	Fine Aggregate (%)
Cumulative Percent Passing for each Sieve:	2"	100.0	-
	1 1/2"	100.0	-
	1"	92.0	-
	3/4"	84.6	-
	1/2"	62.5	-
	3/8"	26.7	100
	No. 4	3.3	99
	#8	-	92
	#16	-	69
	#30	-	38
	#50	-	11
	#100	-	2
Bulk Specific Gravity		2.58	2.62
Fineness Modulus		NA	2.89
Absorption Capacity (%)		0.7	0.6

4.2.3 Admixtures

The chemical admixtures used in the preliminary field study include a water-reducing/retarding admixture and an air entraining admixture. The low range water-reducing/retarding admixture conforms to ASTM C 494 (Type A or D). It is an aqueous solution that is blended from refined lignosulfate salt and other water-reducing and plasticizing materials. No calcium chloride is added throughout the manufacturing process. This type of admixture is generally used for applications that require extended set time. The air entraining admixture met the requirements of ASTM C 260. It was formulated with a stabilized, modified resin surfactant that has a chloride content less than 0.05 percent.

4.2.4 Fibers

Two types of fibers were selected for this field evaluation—steel and synthetic. One of the main objectives of this field test is to evaluate CRCP with varying dosages of steel and synthetic fibers to determine the most efficient fiber type and dosage. There are several different properties of fibers that dictate their performance and application. An extensive literature review was conducted to study the most widely used fibers and determine the effect they may have on CRCP. A major issue with fiber reinforcement is related to economic feasibility. The cost of fibers can be substantial when considering a typical paving mix. Consequently, several fiber manufacturers were consulted to determine the most viable fibers for this application. Laboratory tests were also performed with various fibers to determine their effect on workability. Table 4.3

lists the types of fibers that were selected and gives a general description for each fiber. Each of the fibers used in the first field evaluation are illustrated in Figure 4.3 through Figure 4.5.

Table 4.3: Fiber Designation for Preliminary Field Study

Fiber Designation	Description	Length (mm)	Aspect Ratio
SF1	Steel—Collated Hooked-End	65	60
SnF1	Synthetic—Monofilament	90	40
SnF2	Synthetic—Collated-Fibrillated	< 30*	NA*

*The SnF2 fiber is graded and does not conform to a specific length or aspect ratio.

Figure 4.3: SF1 Fiber Type

Figure 4.4: SnF1 Fiber Type

Figure 4.5: SnF2 Fiber Type

4.3 Mixture Proportions

Preliminary concrete mixes were prepared in the laboratory to evaluate the effects that the selected fibers have on the given mix design. The objective was to determine the optimum amount of water reducer/retarder and air entraining agent to achieve a desired slump and air content. Based on the results of the laboratory mixes, several modifications were recommended to the concrete supplier to accommodate the addition of fibers into the concrete mixture. The main modification includes replacing 100 lb/yd^3 of coarse aggregate with 100 lb/yd^3 of fine aggregate for each section containing fibers. The additional fine aggregate helps to maintain the concrete's workability, finishability, and packing of the concrete matrix. The amount of water and air entraining agent was also slightly increased for sections containing fibers to accommodate the additional stiffness. The synthetic fibers have the most influence on workability due to the large quantity of fibers required to reach functional dosages. It should be noted that it is difficult to reproduce field conditions within a laboratory setting. Consequently, minor modifications were made in the field as the field study progressed. Table 4.4 shows the average concrete mixture proportions that were used for each test section.

Table 4.4: Typical Mixture Proportions for Field Evaluation

Section	Mixture Designation*	Cement	Fly Ash	Coarse Aggregate	Fine Aggregate	Water	Water Reducer	Air Dosage
1	Control	451 pcy	96 pcy	1936 pcy	1132 pcy	217 pcy	24 oz/yd^3	2 oz/yd^3
2	SF1-25	451 pcy	96 pcy	1842 pcy	1265 pcy	211 pcy	24 oz/yd^3	2 oz/yd^3
3	SnF1-4	451 pcy	96 pcy	1842 pcy	1265 pcy	221 pcy	24 oz/yd^3	2 oz/yd^3
4	SF1-40	451 pcy	96 pcy	1842 pcy	1265 pcy	227 pcy	24 oz/yd^3	2 oz/yd^3
5	SnF1-6	451 pcy	96 pcy	1842 pcy	1265 pcy	234 pcy	24 oz/yd^3	2 oz/yd^3
6	SnF2-1.5	451 pcy	96 pcy	1842 pcy	1265 pcy	227 pcy	28 oz/yd^3	2 oz/yd^3
7	Control	451 pcy	96 pcy	1936 pcy	1167 pcy	229 pcy	24 oz/yd^3	2 oz/yd^3

*Indicates Fiber Type and Dosage (i.e., SF1-25 contains 25 pcy of steel collated, hooked-end fibers)

4.4 Mixing and Paving Procedure

The concrete used for the preliminary field evaluation was produced in a central mix plant located about four miles south of the site at the intersection of Highway 59 and University Drive. The addition of fibers did not greatly affect the mixing process. The main consideration was to uniformly distribute the fibers throughout the concrete mixture in a manner that would not disrupt the operation. There are two different methods that were used for adding the fibers to the concrete mix. Each of these methods is dependent on the type of bag that contains the fibers. The SnF fibers were contained in water soluble bags that dissolve in the concrete mixer, eliminating the need to open the bag during the operation. This is a simple and clean method that reduces the chance of fibers missing the mixer. As shown in Figure 4.6, a worker stands on the platform next to the mixer and adds the appropriate bags of fibers at the beginning of the mixing process. The SF fibers were contained in durable bags that are not water soluble. As a result, the bags must be cut and manually loaded into the mixer, generally on a conveyor belt as shown in Figure 4.7. It should be noted that the addition of fibers did not cause any delays throughout the mixing operation.

Figure 4.6: Manual Loading of Synthetic Fiber Bags into Concrete Mixer

Figure 4.7: Conveyor Belt Loading of Steel Fibers into Concrete Mixer

The concrete was transported to the field site using 10 yd^3 dump trucks. Depending on the time of day, it took between 10 and 30 minutes to transport the concrete. The paving method used for the preliminary field evaluation was a concrete spreader as described in Chapter 2. Once the dump trucks reached the site, the concrete was loaded onto a conveyor system that uniformly spread the concrete across the section, as shown in Figure 4.8. This ensured proper distribution and improved its ability to be worked by hand. The concrete was then shoveled and manually vibrated to ensure proper depth for consolidation and finishing. A roller screed was then used to strike off excess concrete, as shown in Figure 4.9, so that floats could apply a smooth finish to the surface. Surface texturing was next applied by dragging carpet along the length of the pavement. To complete the surface texturing of the pavement, tining was manually applied with a rake. Finally, a curing compound was sprayed onto the surface to minimize water evaporation.

Figure 4.8: Placement of Concrete Using Spreader

Figure 4.9: Roller Screed Operation

4.5 Testing Program

Concrete specimens were obtained from at least one truck in each pavement section. Specimens include twenty 6-in. x 12-in. cylinders, two 4-in. x 8-in. cylinders, and three 6-in. x 6-in. x 20-in. flexural beams, which were tested for compression, splitting tension, modulus, flexural toughness, and permeability at a combination of 7, 28, and 91 days. The results of these tests were used to evaluate the effects that fiber type and dosage have on a typical pavement mix. On the day following casting, each specimen was transported to the Construction Materials Research

Group Laboratory in Austin to cure until it could be tested. Fresh concrete properties and ambient conditions were also measured while the specimens were being cast. Fresh properties include slump, air content, unit weight, and concrete temperature. Ambient conditions include relative humidity, wind speed, and ambient temperature.

4.5.1 Fresh Concrete Properties

Tests were conducted to monitor the properties of the concrete in its plastic state to ensure proper workability and quality control. Fibers may influence the fresh properties of concrete, depending on the fiber type and dosage. Readings were taken from each section to document slump, concrete temperature, air content, and unit weight. Each of the fresh concrete properties was obtained in accordance with ASTM standards, as shown in Table 4.5. This section discusses the results of these findings, which are summarized in Table 4.6. It should be noted that measurements were not taken from Section 6—SnF2-1.5 because of the short section length and rapidity of construction.

Table 4.5: Test Methods

Test Number	Description
ASTM C 143	Standard Test Method for Slump
ATM C 138	Standard Test Method for Air Content
ASTM C 138	Standard Test Method for Unit Weight

Table 4.6: Summary of Fresh Concrete Properties

Section	Mixture Designation	Time Placed	Concrete Temperature (°F)	Slump (in.)	Air Content (%)	Unit Weight (lb/yd3)
1	Control	7:30 am	90	4.5	4.6	140.0
1	Control	8:12 am	92	3.5	4.6	140.0
2	SF1-25	9:00 am	96	2.5	5.5	143.6
2	SF1-25	9:35 am	95	2.75	4.5	144.0
3	SnF1-4	10:41 am	96	2.75	5.0	140.0
3	SnF1-4	10:54 am	98	2.75	NA*	140.0
4	SF1-40	11:35 am	105	4.0	NA*	140.0
4	SF1-40	11:50 am	98	2.25	NA*	144.0
5	SnF1-6	1:15 pm	101	3.25	NA*	139.2
7	Control	2:35 pm	99	4.25	NA*	143.2

*Air content readings were not taken because the seal broke on the pressure meter.

4.5.1.1 Slump

A standard slump cone was used to measure the slump for each section. The desired slump for this concrete mix was approximately 3.5 in., because it was being constructed as a hand pour. Because of the fluctuation in transportation time and high ambient temperature, the concrete has a tendency to set before it reaches the field site, making it difficult to achieve a specific slump. The fibers also influence the slump obtained by a standard slump cone. The slump recorded during the field study ranged from 2.25 to 4.5 in., depending on the time of day as well as the fiber type and dosage.

4.5.1.2 Air Content

A pressure meter was used to determine the air content of the concrete. The desired air content was 3.5 percent. Air content is not considered to be a major issue in this region of the state since freeze-thaw damage does not generally occur. The air content recorded during the field study ranged from 4.5 to 5.5 percent. Unfortunately, the seal broke during the third test section and was unable to yield results for the last four sections.

4.5.1.3 Unit Weight

Unit weight was measured using a standard 0.25 ft^3 cylinder. Fibers do not greatly affect the unit weight of the concrete. The recorded unit weight ranged from 139.2 to 144.0 lb/ ft^3, depending on the section. The average value for unit weight was determined to be 141.4 lb/ ft^3.

4.5.1.4 Concrete Temperature

TxDOT recognizes the effects of concrete temperature and has introduced a maximum temperature of 95º F in an effort to minimize the effects of plastic shrinkage. Because of the extreme ambient conditions, the concrete temperature actually exceeded this limit on the day of the field evaluation. The concrete temperature ranged from 90 to 105º F. As a result, the concrete should experience significant shrinkage effects that encourage pavement distress.

4.5.2 Ambient Conditions

Ambient conditions influence the amount of shrinkage that occurs. Concrete temperature often exceeds ambient air temperature, making the concrete vulnerable to shrinkage. Therefore, it is important to document the ambient conditions present when a pavement is constructed. Readings were taken from each section to document relative humidity, wind speed, and ambient temperature. Temperature and relative humidity readings were taken with a thermo hygrometer. The wind speed was measured using an anemometer vane probe. This section presents the data collected from the day of construction. The results are summarized in Table 4.7. It should be noted that measurements were not taken from Section 6—SnF2-1.5 because of the short section length and rapidity of construction.

Table 4.7: Summary of Ambient Conditions

Section	Mixture Designation	Time Placed	Ambient Temperature (ºF)	Relative Humidity (%)	Wind Speed (ft/min)
1	Control	7:30 am	80.5	87.9	NA
1	Control	8:12 am	83.7	90.3	210
2	SF1-25	9:00 am	NA	83.2	70
2	SF1-25	9:35 am	90.3	78.1	NA
3	SnF1-4	10:41 am	91.2	63.4	190
3	SnF1-4	10:54 am	94.2	61.6	210
4	SF1-40	11:35 am	95.8	47.8	290
4	SF1-40	11:50 am	94.6	56.6	210
5	SnF1-6	1:15 pm	96.6	48.5	160
7	Control	2:35 pm	95.8	47.0	7

4.5.3 Constructability

One of the main benefits of the field evaluation was to measure the constructability of CRCP containing fiber reinforcement. There has been very little experience with fibers in full-depth pavements, making it difficult to predict how they will affect a paving operation. One of the main concerns with placing fibers in CRCP is that pavement mixes generally have a low slump.

This issue is less of a concern for this study because concrete placed by hand typically has a slump of 4.5 inches. Another issue with fiber reinforcement is that it causes inconstancies in the surface during finishing and tining. This was also evaluated in the field study for each fiber type and dosage. This section discusses the observations made throughout the construction of the test pavement.

4.5.3.1 Control Sections

The concrete in the field evaluation was placed by hand and manual vibration. Neither Section 1 nor Section 7 experienced problems related to constructability. The concrete was easily placed throughout the operation. In addition, the laborers did not seem to have any trouble finishing the surface. The final result was a smooth surface that allowed for easy tining. A typical surface profile for a control section is shown in Figure 4.10.

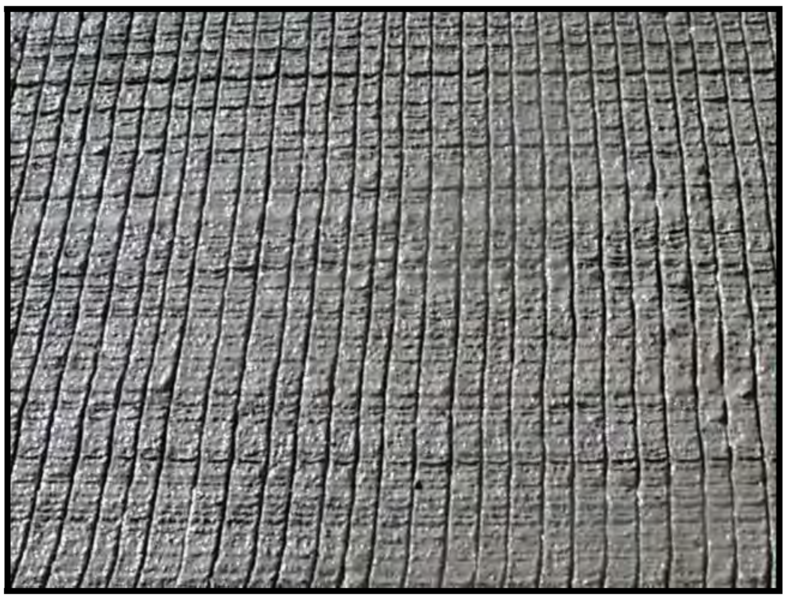


Figure 4.10: Surface Profile of Control Section

4.5.3.2 SF1 Sections

Similar to the control sections, the sections containing hooked-end steel fibers (SF1-25 and SF1-40) were relatively easy to construct. The concrete was initially stiff but was easily maneuvered when agitated by the vibrator. The fibers tended to blend well with the concrete after the roller screed leveled the surface. Very few fibers protruded from the pavement surface before the finishing stage. However, during finishing, the straight edge had a tendency to expose the fibers at the surface. This trend was exacerbated by the tining operation. It should be noted that the majority of exposed fibers were oriented parallel to the pavement surface. However, a small amount of them extended vertically from the pavement, presenting a potential hazard to vehicles tires. Although it is not likely that exposed fibers would puncture a tire, they do increase the rate of degradation in a tire’s tread. A typical surface profile for an SF1 section is shown in Figure 4.11. It should be noted that the SF1 fibers did not cause any damage to the paver during construction.

Figure 4.11: Surface Profile of SF1 Section

4.5.3.3 SnF1 Sections

The sections containing SnF1 fibers encountered the most difficulty during construction. The construction was mainly impeded by the inability to finish the concrete, predominantly in the section containing 6 lb/yd^3 of fibers. The workability of the concrete was also impaired. This is attributed to the low specific gravity of the SnF1 fibers. Initially, the concrete was very stiff but was improved by boosting the amount of water and water reducer to the maximum allowable dosages.

The finishing process produced a hairy surface throughout the section, which is clearly depicted in Figure 4.12. The roller screed was unable to embed the fibers into the concrete, but the carpet drag seemed to help somewhat. The problem was exacerbated by the straight edge and tining operation. It should be noted that the exposed fibers do not pose a hazard to motorists but do diminish the pavement's appearance. The surface profile of completed SnF1-6 section appears in Figure 4.13.

Figure 4.12: Surface Profile of Unfinished SnF1-6 Section

Figure 4.13: Surface Profile of Completed SnF1-6 Section

4.5.3.4 SnF2 Section

The section containing SnF2 fibers did not experience much difficulty during construction. This type of fiber is mainly used to reduce plastic shrinkage and can therefore be used in small dosages. The majority of the fibers seemed to settle into the concrete, providing a suitable surface for finishing. The stiffness of the concrete also did not require additional water or water reducer to be worked. Overall, the SnF2 fiber performed well with regards to constructability.

4.5.4 Hardened Concrete Properties

It is generally understood that fibers have little influence on hardened concrete properties with the exception of flexural toughness. This is especially the case for the fiber dosages selected for this field study. This section discusses the test results of the hardened concrete properties—compressive strength, splitting tensile strength, modulus of elasticity, and flexural toughness. Each test was performed in accordance with the associated ASTM Specification as listed below in Table 4.8. The test results help to evaluate the different fiber types and dosages as well as validate the quality control for each test section. It should be noted that measurements were not taken from Section 6—SnF2-1.5 because of the short section length and rapidity of construction.

Table 4.8: Test Methods

Test Number	Description
ASTM C 39	Compressive Strength of Concrete
ATM C 496	Splitting Tensile Strength of Concrete
ASTM C 469	Modulus of Elasticity of Concrete
ASTM C 1202	Rapid Chloride Permeability
ASTM C 78	Flexural Strength
ASTM C 1018	Flexural Toughness

4.5.4.1 Compressive Strength

Fiber reinforcement generally does not influence the compressive strength of a concrete sample. Incremental strength gains may be obtained in certain instances but are not considered reliable. The results for the compressive strength tests are summarized in Table 4.9. The test results confirm that the addition of fibers at the given dosage rates has little influence over the compressive strength of the concrete. In fact, the majority of sections containing fibers obtained a lower compressive strength. This is attributed to the additional water that was added for required workability. The test results also illustrate a nice consistency between the different sections for each day, indicating proper quality control.

Table 4.9: Summary of Compressive Test Results

Section	Mixture Designation	Average Compressive Strength (psi)		
		7-day	28-day	91-day
1	Control	3230	3760	4740
2	SF1-25	3430	3840	4170
3	SnF1-4	3200	3960	4310
4	SF1-40	3280	3970	4610
5	SnF1-6	2920	3400	4180
7	Control	3070	3960	4380

4.5.4.2 Elastic Modulus

Fiber reinforcement also has little effect on the elastic modulus of a concrete sample. The results for the modulus tests are summarized in Table 4.10. The test results confirm that the addition of fibers at the given dosage rates does not influence the elastic modulus of the concrete. It does appear that the elastic modulus is affected by the time of placement. Each day of testing experienced high results for the first section, which gradually decreased as the day progressed. This may be attributed to the extreme ambient conditions that worsened throughout the day.

Table 4.10: Summary of Elastic Modulus Test Results

Section	Mixture Designation	Average Modulus x 10^3 (psi)		
		7-day	28-day	91-day
1	Control	4850	5050	5700
2	SF1-25	4900	4900	6750
3	SnF1-4	4150	4450	5300
4	SF1-40	4100	4650	5100
5	SnF1-6	3900	4650	5150
7	Control	4200	4650	5200

4.5.4.3 Splitting Tensile Strength

Fiber reinforcement generally does not influence the splitting tensile strength of a concrete sample. The results for the splitting tensile strength tests are summarized in Table 4.11. The test results confirm that the addition of fibers at the given dosage rates does not significantly increase the splitting tensile strength of concrete.

Table 4.11: Summary of Splitting Tensile Test Results

Section	Mixture Designation	Average Splitting Tensile Strength (psi)		
		7-day	28-day	91-day
1	Control	370	480	485
2	SF1-25	365	485	530
3	SnF1-4	355	445	495
4	SF1-40	360	505	545
5	SnF1-6	340	455	495
7	Control	315	415	470

4.5.4.4 Permeability

Permeability tests were conducted at 28 days using the rapid chloride ion penetration test specified in ASTM C 1202. It has been well-documented that fiber reinforcement has little influence on concrete's permeability. The test results are listed below in Table 4.12. The steel fiber reinforced concrete appears to have a much higher permeability than the other test sections. However, this value is deceiving because the steel fibers act as a conductor to the electrical current passing through the cylinder, causing the results to be flawed. The synthetic fiber reinforced samples did not encounter this problem and gave similar results to the control sections. It is believed that the steel fiber specimens would produce similar results for permeability if not for the testing flaw described above.

Table 4.12: Summary of Permeability Test Results

Section	Mixture Designation	Average Permeability (Charge Passed in Coulombs)
1	Control	4050
2	SF1-25	7140*
3	SnF1-4	4420
4	SF1-40	6610*
5	SnF1-6	4760
7	Control	4070

*Values were likely affected by the presence of steel fibers in test specimen.

4.5.4.5 Flexural Toughness

Flexural toughness tests were conducted at 28 days using the test method specified in ASTM C 1018. The specimens used for testing were 6-in. x 6-in. x 20-in. flexural beams that conform to specifications. Flexural toughness is the main performance parameter that is used to quantify the benefits of fibers. A load-deflection plot was analyzed to determine the amount of toughness the concrete contained. Figure 4.14 shows a typical load-deflection plot that was obtained from a fiber reinforced concrete specimen. The main results of the toughness test are toughness indices (I) and residual strength factors (R). The actual amount of toughness gained depends primarily on the fiber type and dosage.

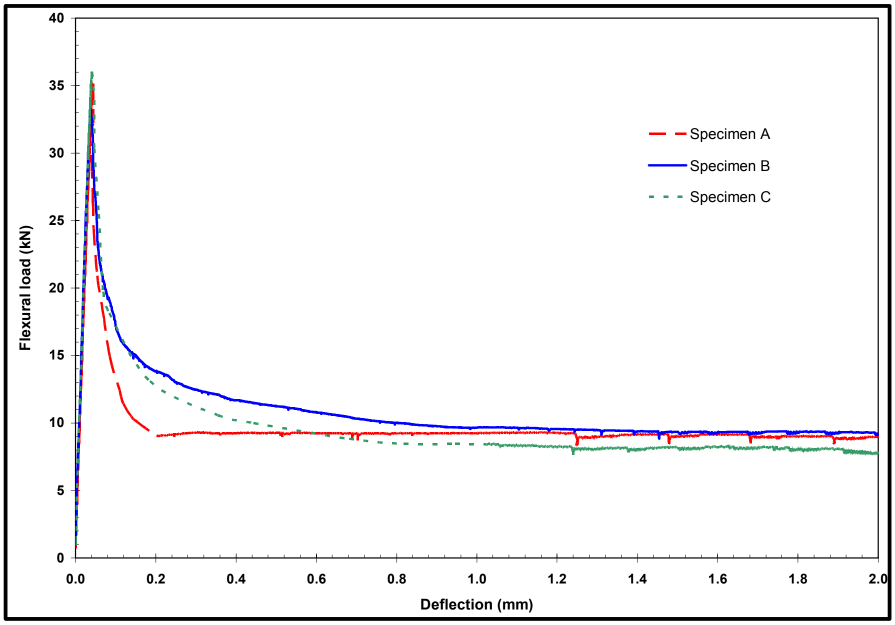


Figure 4.14: Typical Load-Deflection Plot

One of the main performance parameters that governs the amount of toughness gained is the ability of the fiber to bond to the concrete matrix. There is a fine balance between this bond and the fiber's fracture strength that must be obtained to produce desirable results. Maximum values for toughness are obtained when the fiber gradually deforms and eventually separates from the concrete. Tests were conducted to measure this characteristic for each of the fibers used in the study. The SF1 fiber performed exceptionally well in regards to this criteria. However, the SnF1 fiber experienced fracture for approximately one-third of its fibers. This drastically decreased its measured toughness and ability to achieve ductility. Table 4.13 indicates the toughness indices and residual strength factors for each test section. The main criterion used to evaluate toughness is the residual strength factor. It is apparent that the toughness gain is directly proportional to the fiber dosage, as shown in Figure 4.15 and Figure 4.16. The SF1 fiber sections develop much higher values for residual strength factors as opposed to the SnF1 fiber sections. This is attributed to the fact that the SnF1 fibers often fractured before failure.

Table 4.13: Toughness Results

	1–Control	2–SF1-25	3–SnF1-4	4–SF1-40	5–SnF1-6	7–Control
Toughness Indices						
I_5	1.72	2.98	2.71	3.58	2.39	2.17
I_{10}	1.72	4.60	3.40	5.92	3.28	2.62
I_{20}	1.72	6.66	4.24	10.88	4.90	3.08
I_{30}	1.72	9.04	5.18	15.72	6.45	3.36
I_{60}	1.72	15.23	7.92	30.03	10.82	3.82
Residual Strength Factors						
R_5	0.00	32.43	13.83	46.78	17.75	9.07
R_{10}	0.00	20.59	8.39	49.57	16.22	4.57
R_{20}	0.00	23.83	9.39	48.37	15.44	2.79
R_{30}	0.00	20.64	9.12	47.71	14.58	1.55
R_{60}	0.00	21.41	8.97	48.40	15.22	2.71
R_5	0.00	32.43	13.83	46.78	17.75	9.07

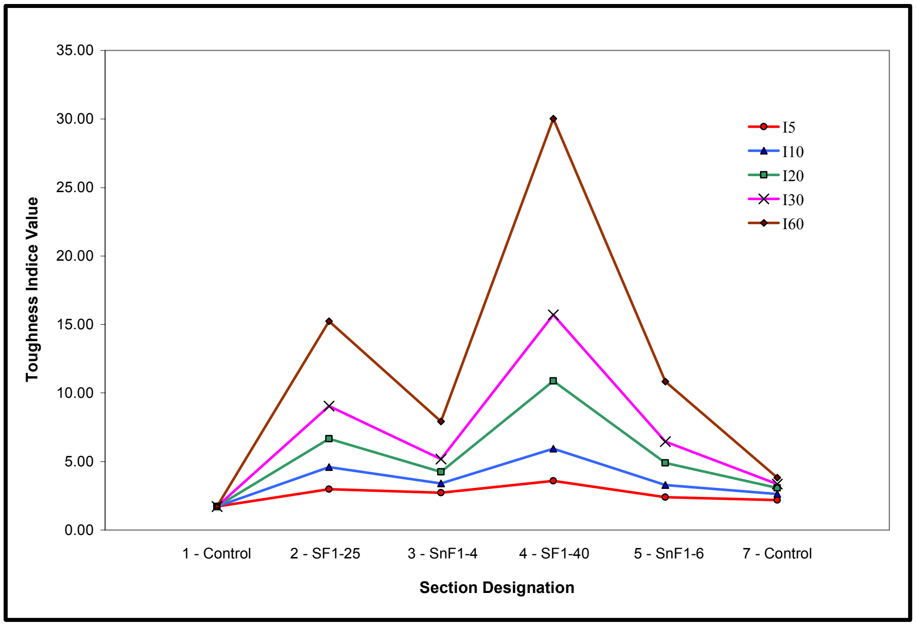


Figure 4.15: Comparison of Toughness Indices

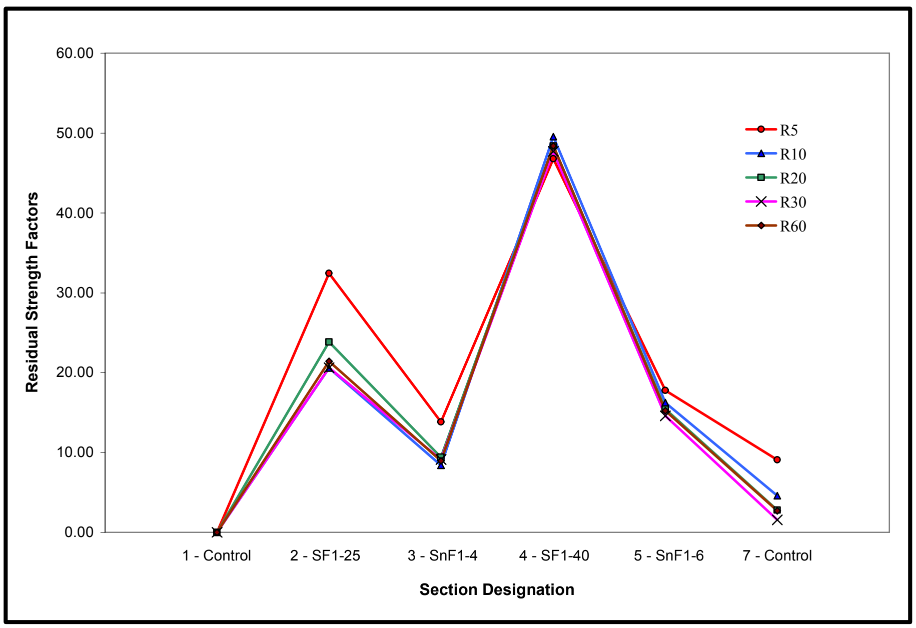


Figure 4.16: Comparison of Residual Strength Factors

4.5.4.6 Flexural Strength

Flexural strength tests were conducted on the same specimens that were used for flexural toughness testing. Research has generally shown that fiber reinforcement increases only the ductility of concrete and not its flexural capacity. This is clearly demonstrated in the flexural strength results that are shown in Table 4.14. In fact, the flexural strength is actually smallest for the sections containing the largest amount of fibers. This is considered to be independent of the fiber reinforcement and is attributed mainly to the additional water required for workability.

Table 4.14: Summary of Flexural Test Results

	1–Control	2–SF1-25	3–SnF1-4	4–SF1-40	5–SnF1-6	7–Control
Flexural Load (lbs.)						
A	8290	7900	7850	6980	7650	7270
B	9250	7490	8670	5950	6960	8610
C	8150	8100	8160	7650	8050	8120
Mean	8560	7830	8230	6860	7550	8000
Flexural Strength (psi)						
A	630	605	630	575	595	560
B	725	590	720	465	560	685
C	665	620	650	640	635	665
Mean	675	605	665	560	595	635

4.6 Monitoring of Test Sections

Monitoring was performed at several intervals to evaluate the condition of the test pavement. The time of monitoring after construction was 10 days, 42 days, 79 days, 130 days, and 235 days. Each day of monitoring was used to document crack spacing, crack width, and spalling. Monitoring was generally conducted between the mid-morning and mid-afternoon to provide the best visibility for crack measurements. Concrete temperature was also monitored by retrieving thermocouple data embedded in the concrete at various locations. In addition to the test pavement, two lanes of pavement were constructed adjacent to the field study with a similar design that offered supplementary monitoring.

4.6.1 Condition of Existing Adjoining Lanes

Two lanes of pavement had been constructed directly adjacent to the field study in January of 2001, 18 months prior to the field study, with a similar concrete mix design. As a result, the existing lanes offer a valid comparison to the field study as well as provide foresight related to the long-term behavior of the field study. The fact that the existing lanes were constructed in the winter season also presented an opportunity to compare the different seasons of placement.

The condition of the existing lanes was monitored and recorded by the research team on August 12, 2002, 10 days after the construction of the field study. The main issues that were noted include spalling, crack width, crack spacing, and continuation cracking. There is clear evidence of spalling, which occurred in the early life of the existing lanes. An example of this spalling is shown in Figure 4.17. Although there are unmistakable cases of spalling that have begun to form, there is not a clear pattern that defines its development. The severity of the spalling also varies with the crack width and spacing, making it more difficult to yield a correlation.

Figure 4.17: Evidence of Spalling in Existing Lane

There is also a significant amount of horizontal reflection cracking that occurred between the two existing lanes of pavement. Horizontal reflection cracking is considered to be cracking that propagates through a pavement joint into an adjacent section. This phenomenon is depicted in Figure 4.18.

Figure 4.18: Horizontal Reflection Cracking in Existing Lane

The average crack spacing for the existing pavement lanes is 9.4 ft. The majority of cracks are spaced more closely, but the average is increased due to a few largely-spaced cracks. The average crack width was 0.020 inches. It should be noted that crack widths could not be obtained across the entire lane due to the presence of traffic.

4.6.2 Thermal Effects

Thermal effects were measured using conventional thermocouple wire and data acquisition equipment. This section discusses the temperature profile within the concrete. It also illustrates the temperature gradient created throughout the pavement.

4.6.2.1 Thermocouple Installation

The thermocouple wire was installed in the pavement the day before the concrete was placed. It was originally intended to take a temperature reading from each test section with three data acquisition boxes. However, the second box malfunctioned and did not record any data. In addition, the test sections were not constructed to the intended lengths. Figure 4.19 and Figure 4.20 illustrate the intended and actual layout for the test sections and data acquisition boxes. The concrete temperature was monitored at three equally spaced layers, as shown in Figure 4.21, to demonstrate the temperature gradient throughout the pavement depth. The wire was placed halfway through each pavement section in the middle of the lane.

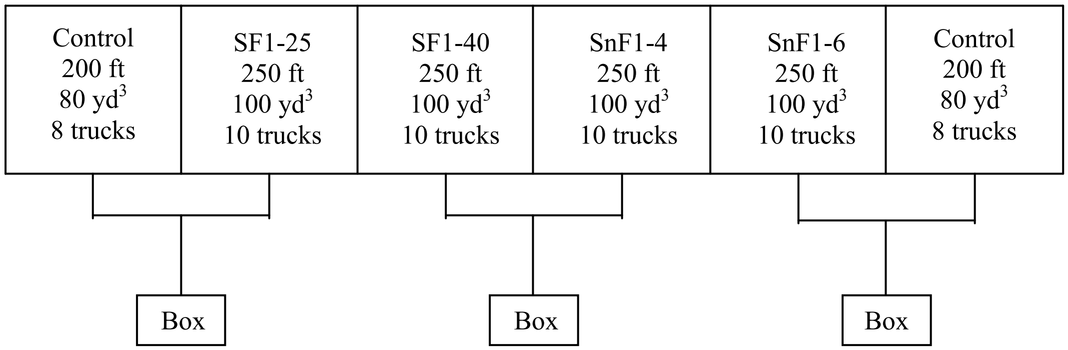


Figure 4.19: Intended Test Section Layout with Instrumentation

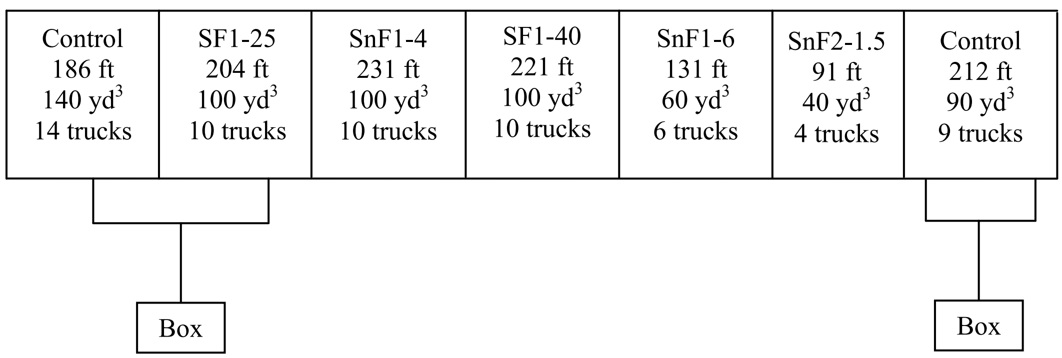


Figure 4.20: Actual Test Section Layout with Instrumentation

Figure 4.21: Close-up of Thermocouple Installation

4.6.2.2 Concrete Temperature Profile

The thermal data offer an accurate representation of the expansion the concrete experiences from environmental conditions. Because the pavement had still not opened to traffic, the majority of cracks were directly related to early volumetric expansion due to heat of hydration followed by rapid surface contractions from drying shrinkage and drops in both ambient and concrete temperatures. It should be noted that the following results are specific for this mixture design placed at this thickness under these temperature humidity, and wind conditions. Controlling these variables result in greater confidence for expected performance of early age concrete slabs. The concrete temperature is especially important during its early stages due to the heat of hydration that it experiences. The main portion of the heat cycle takes place shortly after the concrete is placed. The peak of the heat of hydration is directly related to the ambient temperature. The field study experienced very high ambient temperatures, which can produce a damaging heat of hydration. This may include an increased water demand, high rate of slump loss, quick setting time, and increased plastic shrinkage cracking that gives a decreased strength as well as high shrinkage and creep (Suh et al., 1992). Typical morning and afternoon temperature curves in the field study are illustrated in Figure 4.22 and Figure 4.23. The temperature difference between initial reading and peak heat of hydration reached a value of 40º F for morning placement and 32º F for concrete placed in the afternoon.

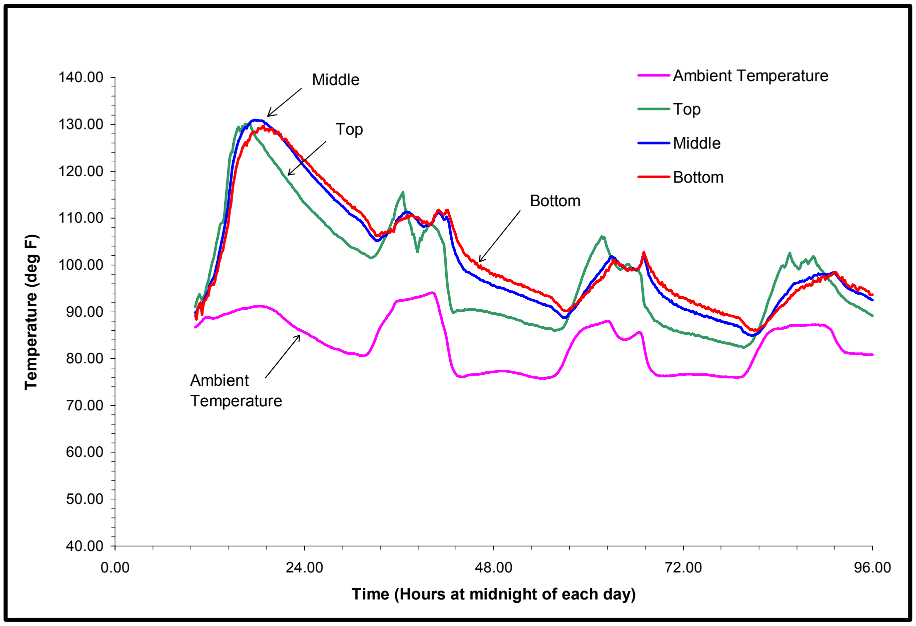


Figure 4.22: Typical Temperature Curve for Concrete Placed in Morning

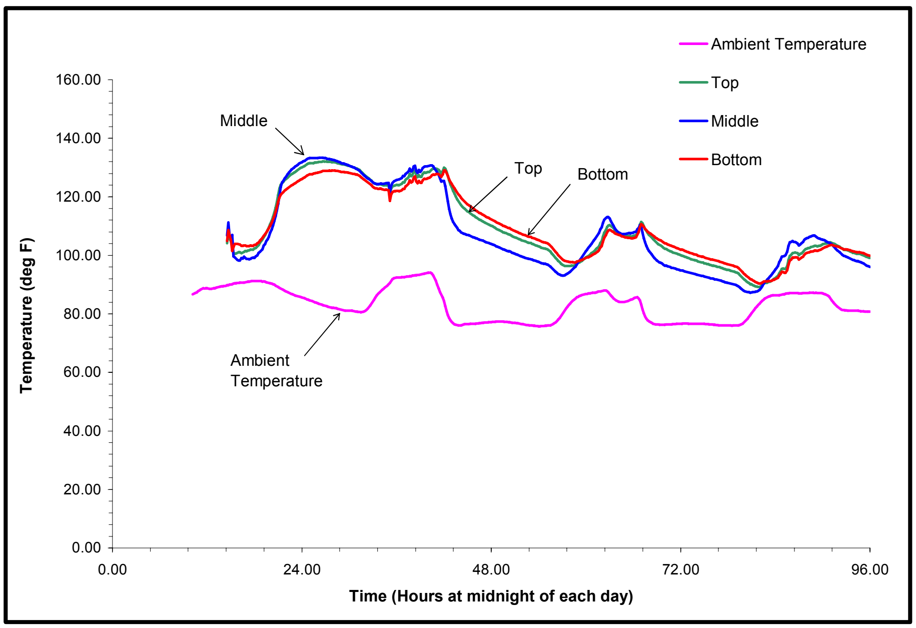


Figure 4.23: Typical Temperature Curve for Concrete Placed in Afternoon

4.6.2.3 Concrete Temperature Gradient

The temperature gradient throughout the slab depth also influences the crack development of the pavement. Figure 4.24 and Figure 4.25 show the rate of temperature change throughout the day for various depths in the slab. These temperature gradients can produce curling and warping stresses, which may lead to undesirable transverse cracks and severe distress. The concrete placed in the afternoon experienced much different ambient conditions than the concrete placed in the morning. For the concrete placed in the morning, the top of the slab had a higher initial temperature, but as the day progressed, the temperature in the bottom of the slab increased at a higher rate and eventually surpassed the top of the slab. The concrete placed in the afternoon had a different effect on temperature gradient. The bottom of the slab initially had a higher temperature until around 6:00 p.m., when the temperature in the top of the slab began to increase at a higher rate. This is a direct result of the ambient conditions and can have detrimental effects on the long-term pavement performance.

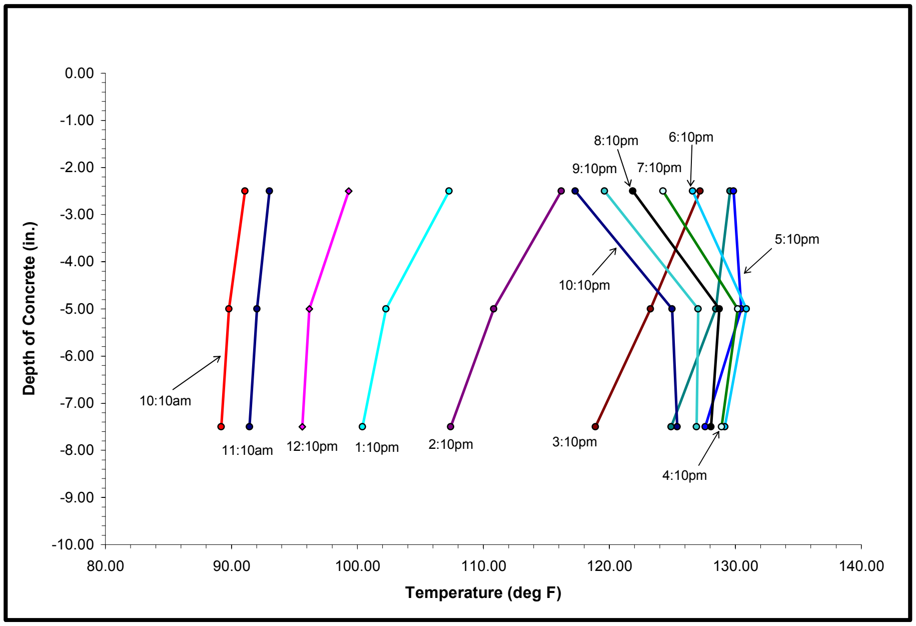


Figure 4.24: Typical Thermal Gradient for Concrete Placed in Morning

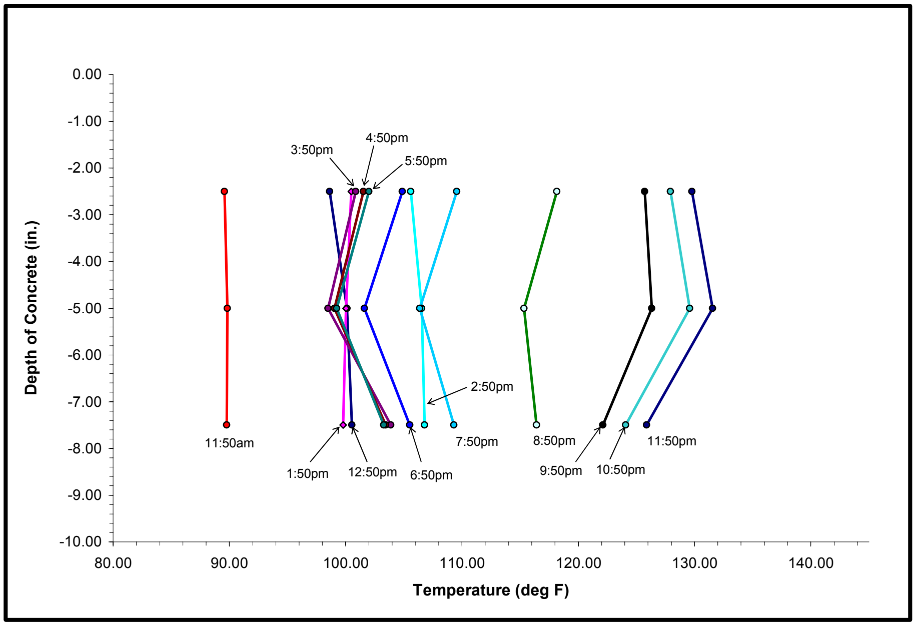


Figure 4.25: Typical Thermal Gradient for Concrete Placed in Afternoon

4.6.3 Crack Spacing

The crack spacing for the field evaluation was determined by surveying the field site at several time intervals. This data was then analyzed to determine the effect that different fiber types and dosages may have on pavement performance. Several different approaches were used to analyze the crack spacing. The percentage of cracks was determined for each individual crack spacing, then compared for each test section. In addition, the average spacing of the closest three cracks was determined and plotted along the length of the pavement to give a crack distribution. Also, the crack spacing was evaluated over time as well as the time of crack formation. This section discusses the findings of the crack spacing analysis and how fibers influence its development.

4.6.3.1 Average Crack Spacing

Fiber reinforcement is not intended to prevent cracking. In fact, the main benefits of fibers come after the concrete has already cracked. It is at this point that fibers begin to bridge cracks and distribute stresses in ways not possible by conventional reinforcement. Consequently, the average crack spacing should not vary significantly between the fiber and controlled sections.

The average crack spacing for each section is shown in Figure 4.26. It should be noted that there is no pavement constructed at the beginning of the field section, reducing the amount of restraint in the first section. In addition, the shoulder has not yet been constructed throughout the field study, allowing further volumetric changes in the concrete. Consequently, the crack spacing is much higher in the first section due to the lack of restraint. To clearly demonstrate this point, note that there are no cracks in the first 41 ft of the first section. The crack spacing also decreases as the restraint increases through the section. The average crack spacing is virtually the same in the remaining test sections, ranging from 3.2 to 4.5 ft. This point is further verified in Figure 4.27, which displays the cumulative percentage of cracks at each spacing after 8 months of construction.

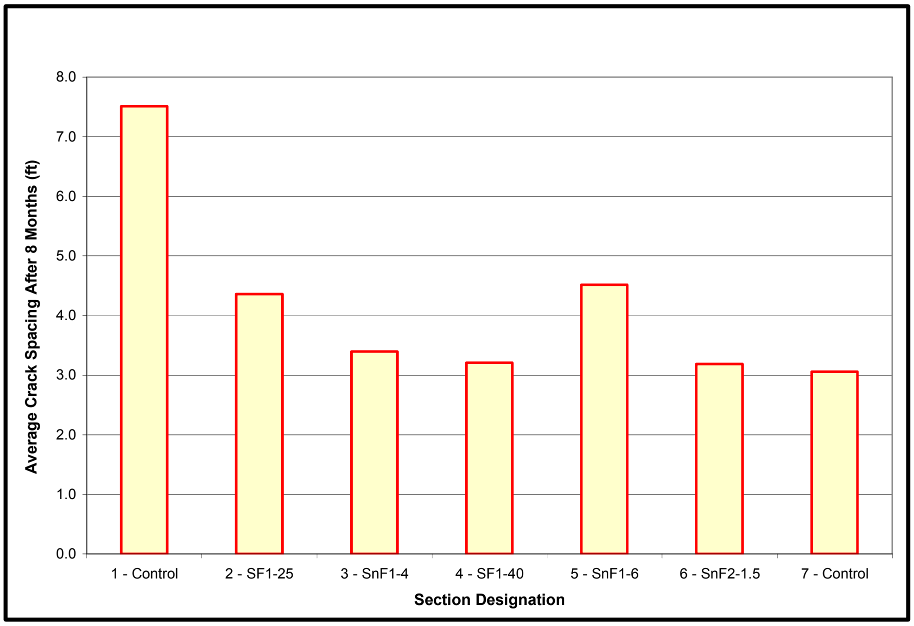


Figure 4.26: Average Crack Spacing for each Section

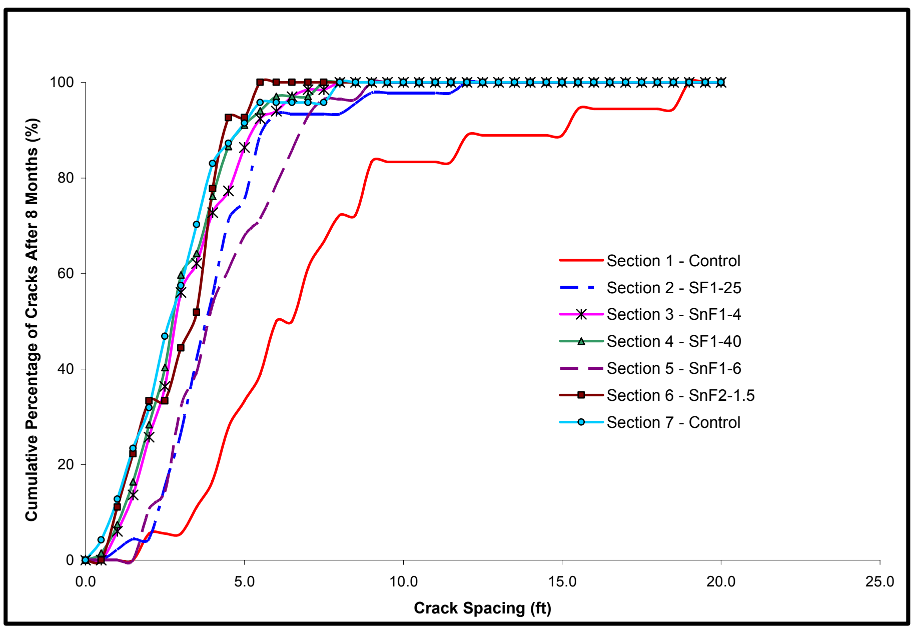


Figure 4.27: Crack Spacing Comparison for each Section

4.6.3.2 Percentage of Cracks at Each Spacing

The percentage of cracks at each crack spacing is also evaluated to determine the crack distribution for each test section. A typical chart that displays this analysis is shown in Figure 4.28. As expected, the percentage of cracks for Section 1—Control has the highest percentage of largely-spaced cracks due to the lack of restraint at the beginning of the site. This is clearly shown in Figure 4.29. The percentage for cracks spaced beyond 8 ft is more than twice that of any other crack spacings. It should be noted that the cracks were last recorded 8 months after construction. Consequently, very few cracks are expected to form in the future, which may reduce this excessive spacing.

Each section containing fibers experienced crack spacing typically between 2 and 5 ft. This is representative for pavement containing siliceous river gravel and may not be associated with fiber reinforcement. Section 7—Control has the bulk of its cracks within this spacing range as well. Consequently, it is difficult to make a correlation with the presence of fibers on crack spacing. It should be noted that AASHTO provisions (1993) are based on crack spacing between 3.5 and 8 ft, to prevent the development of punchouts. However, narrow crack spacing is no longer as critical because of the recent decline in punchouts. This decline is mainly attributed to subgrade improvements and tying of the pavement to the shoulder.

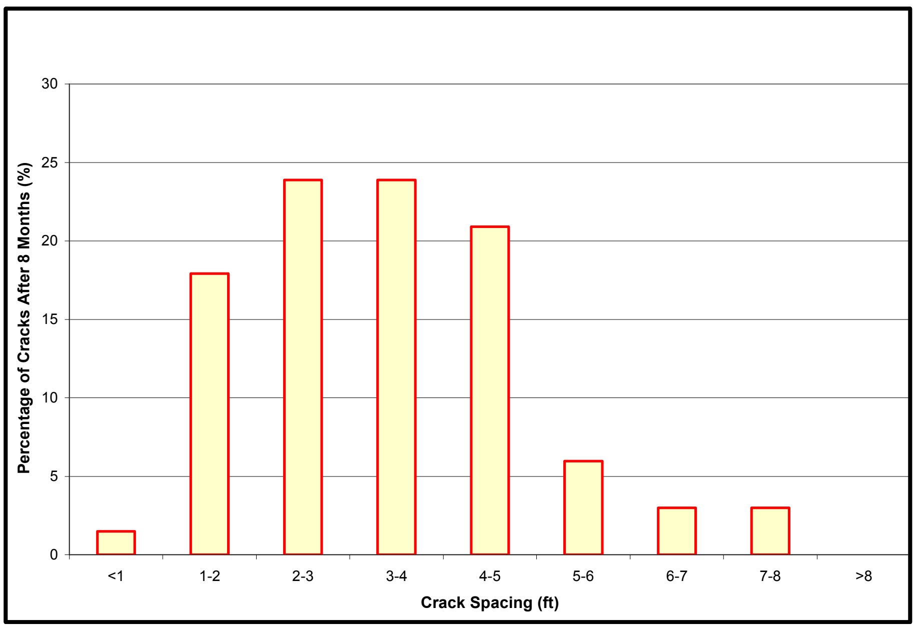


Figure 4.28: Typical Percentage of Cracks for each Spacing

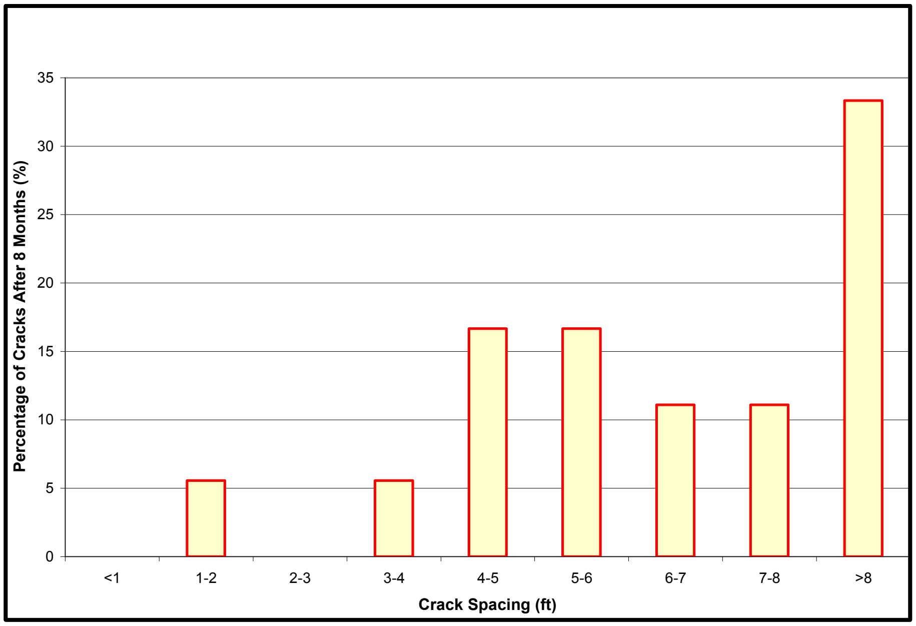


Figure 4.29: Percentage of Cracks for each Spacing in Section 1—Control

4.6.3.3 Average Spacing for Closest Three Cracks

The average spacing for the closest three cracks was analyzed along the field section, as illustrated in Figure 4.30. It clearly shows that the crack spacing is highest at the beginning of the site where fibers are not present. This is also attributed to the lack of restraint in this location. The middle sections containing fibers have a narrow crack spacing that falls below the guidelines set by AASHTO (1993). As previously noted, this violation is of little consequence because of recent improvements in materials and construction methods.

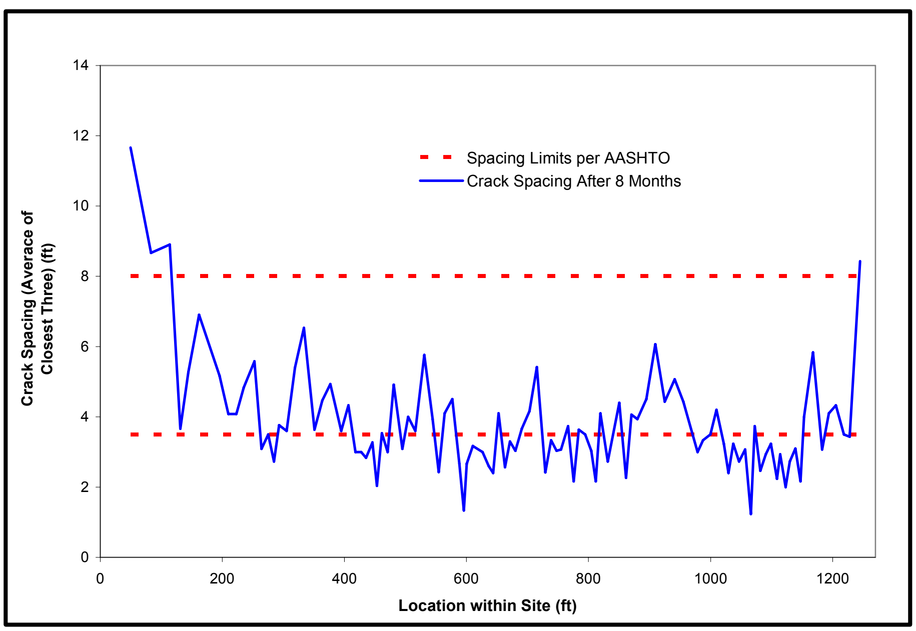


Figure 4.30: Average Spacing of Closest Three Cracks along Pavement

4.6.3.4 Average Spacing versus Time

The average crack spacing is shown in Figure 4.31 as a function of time. It is important to understand the development of cracks as concrete matures. This figure shows that the crack spacing initially fluctuates significantly between sections. However, as the concrete matures, the crack spacing converges to a similar value. It should be noted that the sections containing fibers do not vary noticeably from the control sections.

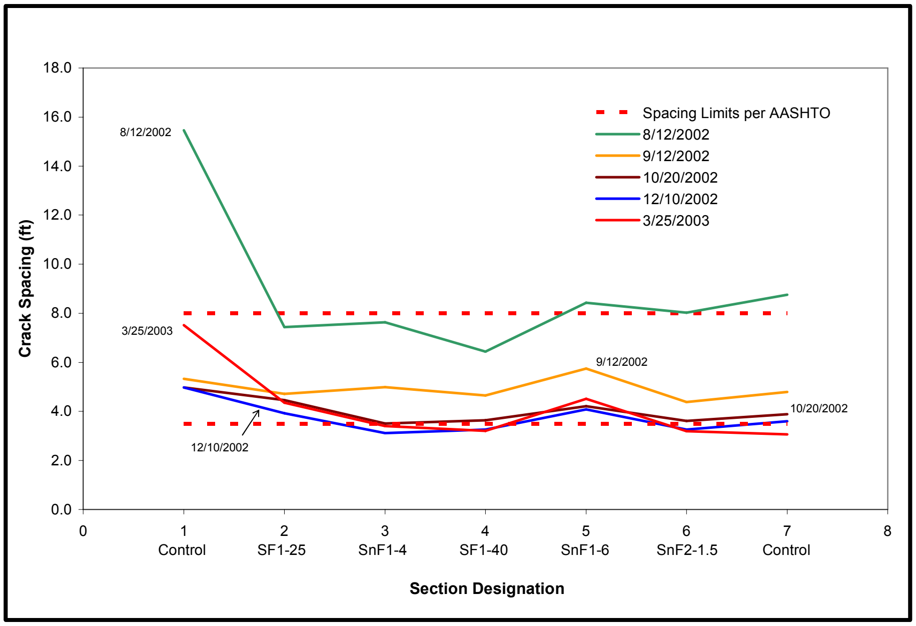


Figure 4.31: Average Crack Spacing versus Time for each Section

4.6.3.5 Time of Crack Formation

Crack development is largely influenced by the concrete temperature in the first few days following construction. It is during this time that concrete experiences the highest temperature resulting from hydration at a point when it has not developed its full strength. This combination tends to produce the majority of cracks that occur in concrete pavement. Concrete placed during the day generally experiences a more rapid crack formation due to the higher temperature at which the concrete sets. This is demonstrated in the figures that compare the time of crack formation for each section. Values for typical crack formation versus time are shown in Figure 4.32. The test sections constructed in the hottest part of the day developed the most cracks in the first few days following construction. McCullough et al. (2000) explains that this is due to the fact that the concrete cannot dissipate the heat generated from hydration as quickly when exposed to high ambient temperature. As a result, the concrete cracks before it can develop sufficient strength. Research has also shown that CRCP generally converges to a stable crack spacing after 100 days. This is clearly demonstrated in the field study because very few cracks were found after the 80-day monitoring. The amount of cracks that form beyond this point are minimal, indicating that the crack development stabilizes.

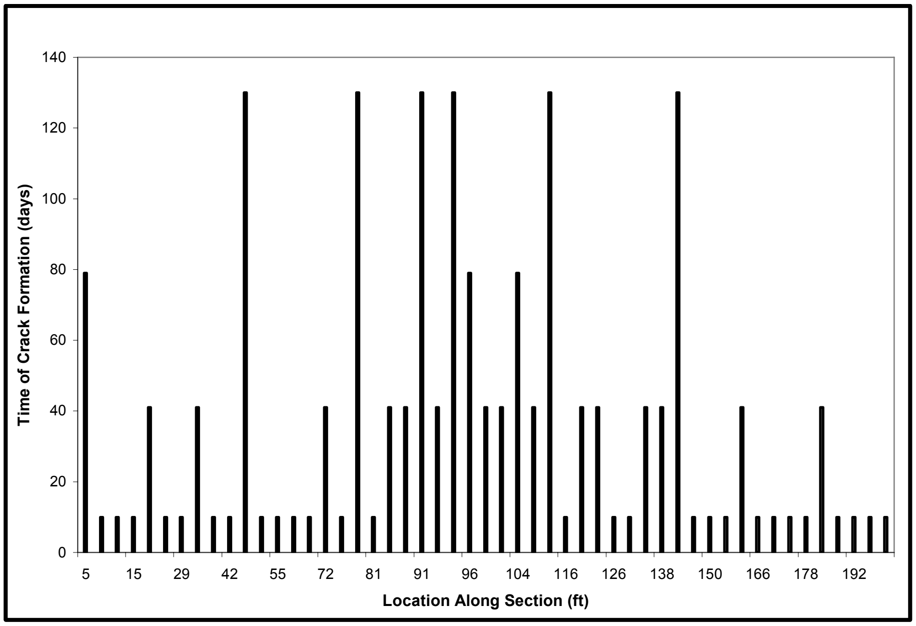


Figure 4.32: Time of Crack Formation along Section

4.6.4 Crack Width

The basic premise of CRCP is the development of cracks that must be held closely together by sufficient longitudinal reinforcement. As a result, crack formation is inevitable for CRCP to function properly. The crack width must be limited to prevent water penetration and maintain load transfer in an effort to mitigate spalling. Tight cracks are also important to prevent corrosion in regions of the country where de-icing salts are used. The main factors affecting the development of crack widths include the placement season, percentage of reinforcing steel, and coarse aggregate type. Crack widths were measured in the field study each time crack spacing was recorded. Each crack width was manually determined using a crack width comparator at three different points along the crack to obtain an average width.

4.6.4.1 Average Crack Width

The coefficient of thermal expansion (CTE) of concrete has a significant influence on crack width. The field study was constructed with siliceous river gravel (SRG). SRG has a much higher CTE value than limestone and results in higher crack widths. It should be noted that concrete constructed with SRG generally also has a larger number of cracks but still maintains higher crack widths due to the additional expansion created. Pavement thickness has also been found to affect crack width. A thicker pavement generally produces a smaller crack width because of the pavement's high volume-to-surface ratio (McCullough et al., 2000). The pavement constructed in the field study has a thickness of 10 in. and thus has a normal volume-to-surface ratio.

Fiber reinforcement provides an excellent measure for maintaining low crack widths by increasing the load transfer across cracks. This helps maintain a continuous material throughout the pavement and reduce stress concentrations. Figure 4.33 compares the average crack width for each section. The main observation is that the first section has the highest crack width. This is mainly attributed to the lack of restraint present at the beginning of the field study, as described earlier. The remaining sections have similar crack widths. At this point, the data do not prove that fibers have much effect on crack width. It should be noted that the crack width decreased for each test section as the day progressed. The main reason for this trend is that the ambient temperature increased throughout the day and eventually peaked as the last section was constructed. The sections constructed during the warmer weather have a tendency to develop a larger number of cracks that need less width to accommodate volumetric changes.

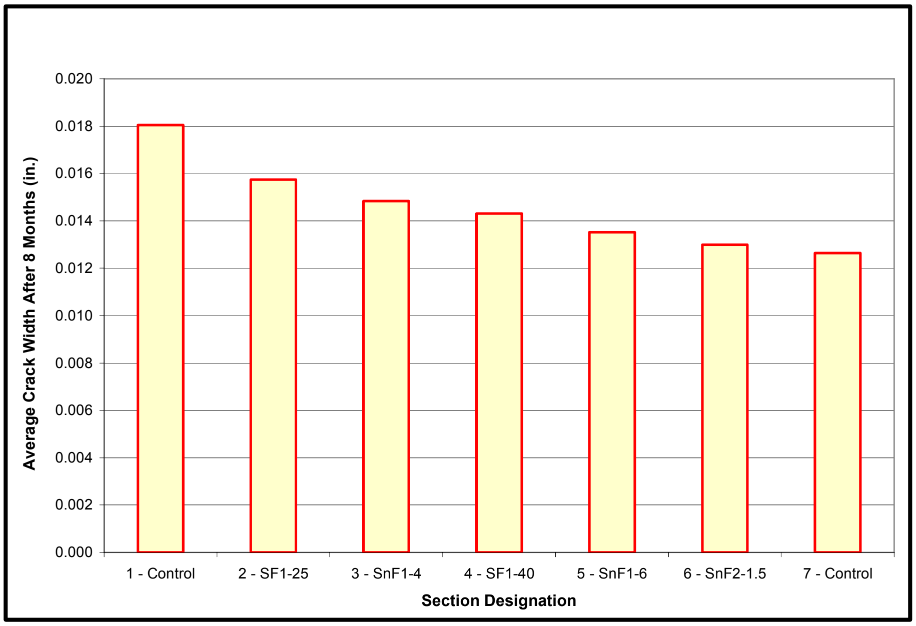


Figure 4.33: Average Crack Width for Each Section

4.6.4.2 Average Width for Closest Three Cracks

The average width for the closest three cracks was analyzed along the field section, as illustrated in Figure 4.34. This graph reinforces the previous point that the crack width decreases as the day progresses. The main change in crack width appears to be related to time of placement and not the fiber reinforcement. It should also be noted that the crack width is much less than the limit of 0.025 in. imposed by TxDOT.

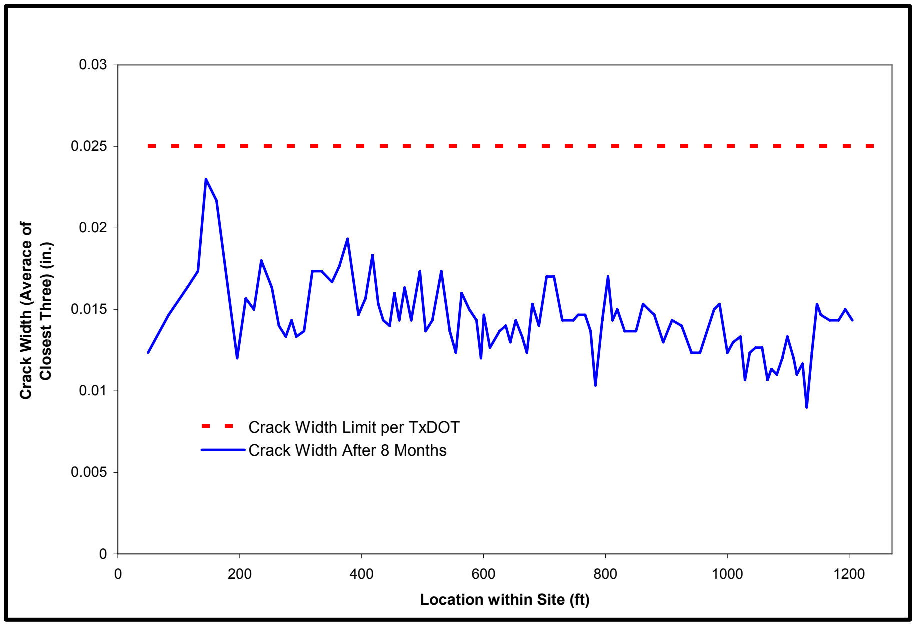


Figure 4.34: Average Width of Closest Three Cracks along Pavement

4.6.4.3 Average Crack Width versus Time

The average crack width is shown in Figure 4.35 as a function of time. This figure shows that the crack width increases with time for each section even though additional cracks form. This is typical of concrete constructed with siliceous river gravel. There was a clear drop in crack width as each test section was constructed. This is again attributed to the increase in ambient temperature during construction.

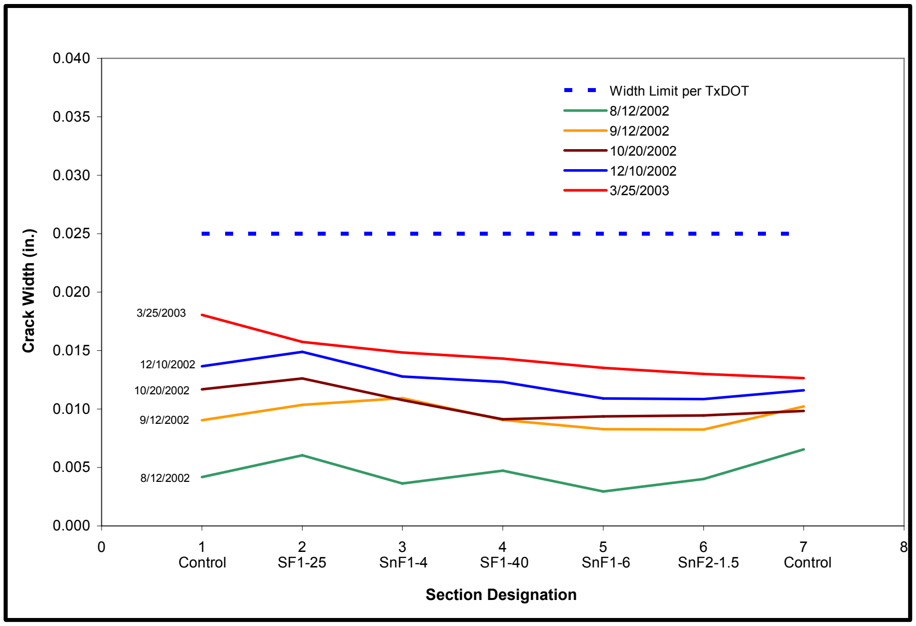


Figure 4.35: Average Crack Width versus Time for each Section

4.6.5 Spalling

The mechanism that causes spalling is not fully understood, although extensive research has been conducted devoted to solving it. It is not clear if it is initiated from repeated heavy truck loading or if it is caused entirely by material behavior. The field evaluation is investigating this subject. This evaluation is particularly useful since the pavement has not yet opened to traffic, offering valuable insight to the initial formation of spalling. Qualitative monitoring has uncovered some evidence that spalling may have begun to develop in each control section of the test pavement, just 235 days after construction. As clearly shown in Figure 4.36, the spalling present in these sections is minor and does not pose a problem at this point. At this time the small bits of surface laitance lost, sometimes called pop-outs, between transverse angle cracks and adjacent tire grooves may never develop into true spalling of well-cured concrete. However, as the pavement is opened to traffic, the repeated heavy truck loading could exacerbate the severity of the spall and require expensive repairs. Although there are some symptoms that spalling may have begun to form, there is not a clear pattern that defines its development, making it difficult to form a correlation. It should be noted that no spalling has been discovered in the sections containing fibers.

Figure 4.36: Evidence of Spalling in Control Sections

4.7 Discussion of Results and Summary

This section serves as a summary for the observations made during construction and the data collected in subsequent monitoring. For each section, the ease of placement and finishability are classified. The levels of classification range from poor to satisfactory to good. In addition, the average crack spacing, average crack width, and presence of spalling are noted. A figure is also displayed which illustrates the crack pattern throughout each test section.

4.7.1 Section 1—Control

Table 4.15 summarizes the findings and observations of the Section 1—Control. It should be noted that this section has begun to develop spalling even though the pavement has not been opened to traffic. The placement and finishing was very smooth, because no fibers were added to the mix. The crack spacing is noticeably higher for this section due to the lack of restraint at the beginning of the section. The layout for the crack locations is listed below in Figure 4.37.

Table 4.15: Summary of Section 1—Control

Ease of Placement	Good
Finishability	Good
Average Crack Spacing	7.5 ft.
Average Crack Width	0.018 in.
Spalling	Yes

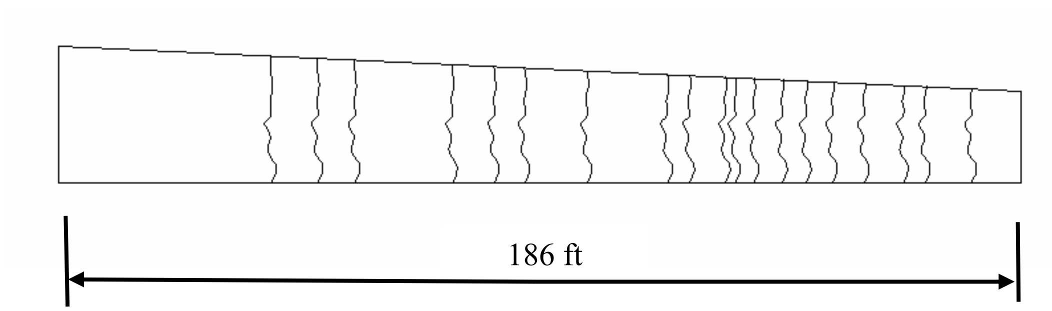


Figure 4.37: Layout of Cracks along Section 1—Control

4.7.2 Section 2—SF1-25

Table 4.16 summarizes the findings and observations of the Section 2—SF1-25. At this point, there is no spalling present in this section. The placement and finishing went reasonably well during construction. There were some problems encountered during the tining operation, which caused many of the fibers to rise out of the surface. The average crack spacing and crack width were typical for the entire pavement length. The layout for the crack locations is listed below in Figure 4.38.

Table 4.16: Summary of Section 2—SF1-25

Ease of Placement	Satisfactory
Finishability	Satisfactory
Average Crack Spacing	4.4 ft.
Average Crack Width	0.016 in.
Spalling	No

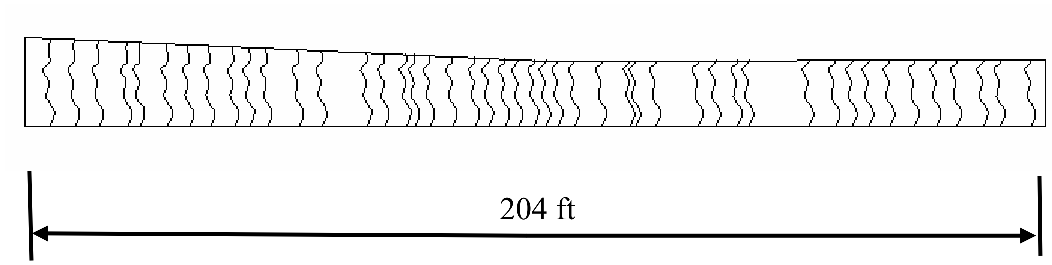


Figure 4.38: Layout of Cracks along Section 2—SF1-25

4.7.3 Section 3—SnF1-4

Table 4.17 summarizes the findings and observations of the Section 3—SnF1-4. At this point, there is no spalling present in this section. The ease of placement went reasonably well during construction. However, there were several problems encountered during the finishing process. The fibers had a tendency to rise out of the pavement surface during each phase of the finishing process. This problem was further exacerbated by the tining operation. The average crack spacing and crack width were typical for the entire pavement length. The layout for the crack locations is listed in Figure 4.39.

Table 4.17: Summary of Section 3—SnF1-4

Ease of Placement	Satisfactory
Finishability	Poor
Average Crack Spacing	3.4 ft.
Average Crack Width	0.015 in.
Spalling	No

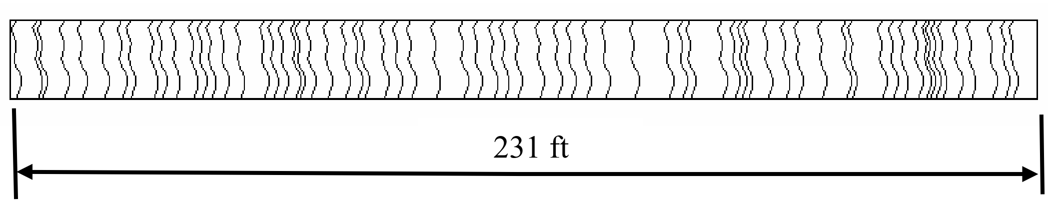


Figure 4.39: Layout of Cracks along Section 3—SnF1-4

4.7.4 Section 4—SF1-40

Table 4.18 summarizes the findings and observations of the Section 4—SF1-40. At this point, there is no spalling present in this section. The placement and finishability was similar to the Section 2—SF1-25. The average crack spacing and crack width were typical for the entire pavement length. The layout for the crack locations is listed below in Figure 4.40.

Table 4.18: Summary of Section 4—SF1-40

Ease of Placement	Satisfactory
Finishability	Satisfactory
Average Crack Spacing	3.2 ft.
Average Crack Width	0.014 in.
Spalling	No

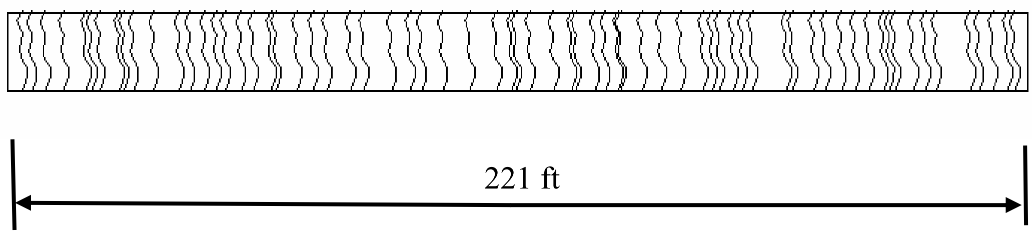


Figure 4.40: Layout of Cracks along Section 4—SF1-40

4.7.5 Section 5—SnF1-6

Table 4.19 summarizes the findings and observations of the Section 5—SnF1-6. At this point, there is no spalling present in this section. Problems were encountered during placement that required the water content to be increased to the maximum allowable dosage. The finishability was similar to the Section 3—SnF1-4. The average crack spacing and crack width were typical for the entire pavement length. The layout for the crack locations is listed below in Figure 4.41.

Table 4.19: Summary of Section 5—SnF1-6

Ease of Placement	Poor
Finishability	Poor
Average Crack Spacing	4.5 ft.
Average Crack Width	0.014 in.
Spalling	No

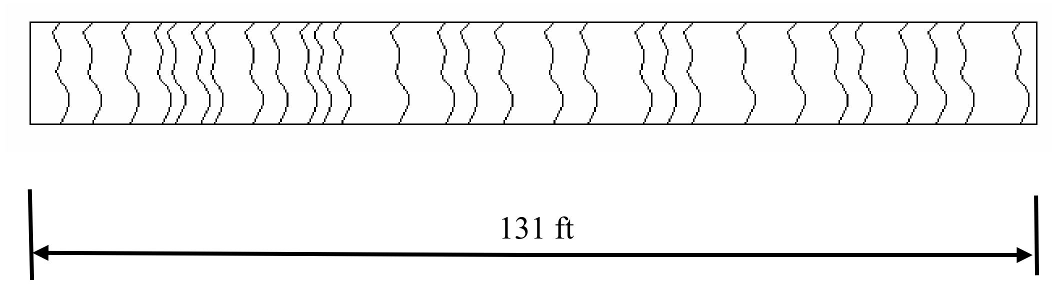


Figure 4.41: Layout of Cracks along Section 5—SnF1-6

4.7.6 Section 6—SnF2-1.5

Table 4.20 summarizes the findings and observations of the Section 6—SnF2-1.5. At this point, there is no spalling present in this section. There were very few problems encountered during placement and finishability. The final pavement surface closely resembled a control section. The average crack spacing and crack width were typical for the entire pavement length. The layout for the crack locations is listed below in Figure 4.42.

Table 4.20: Summary of Section 6—SnF2-1.5

Ease of Placement	Good
Finishability	Good
Average Crack Spacing	3.2 ft.
Average Crack Width	0.013 in.
Spalling	No

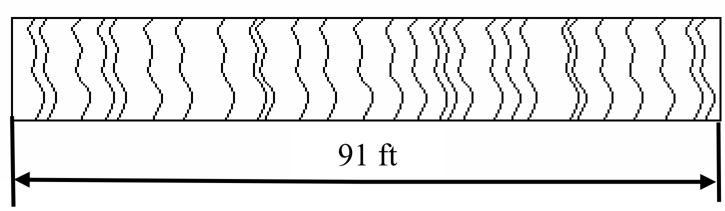


Figure 4.42: Layout of Cracks along Section 6—SnF2-1.5

4.7.7 Section 7—Control

Table 4.21 summarizes the findings and observations of the Section 1—Control. It should be noted that this section has begun to develop spalling even though the pavement has not been opened to traffic. The placement and finishing was very smooth because no fibers were added to the mix. The average crack spacing and width are typical for the entire pavement length. The layout for the crack locations is listed in Figure 4.43.

Table 4.21: Summary of Section 7—Control

Ease of Placement	Good

Finishability	Good
Average Crack Spacing	3.4 ft.
Average Crack Width	0.013 in.
Spalling	Yes

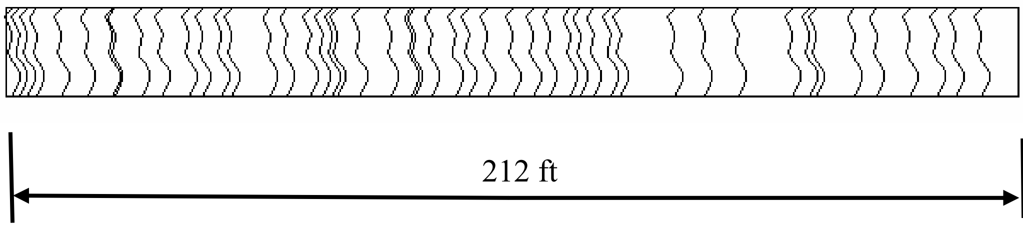


Figure 4.43: Layout of Cracks along Section 7—Control

Chapter 5. Field Evaluation 2

5.1 Overview

The secondary field study was intended to supplement the findings of the preliminary field evaluation. This field pavement was also meant to be constructed under colder conditions than the preliminary field test. Extensive research has shown that pavements constructed in the winter develop much fewer cracks with a better distribution. The concrete placed in the initial field study was constructed in August and experienced severe ambient conditions. Although the second field study was constructed in April, the ambient conditions present during construction closely resemble the conditions that might be experienced in the winter and offer a nice comparison to the previous study. Another key difference between the two field studies is the method of construction. This field study was constructed with a slipform paving machine, making the effect of fibers on workability completely different than the previous study, which was constructed with a concrete spreader. This chapter discusses the influence that each fiber type and dosage imparts on the ability to construct CRCP with a slipform paver.

There were 2 different days required to construct the pavement for this field evaluation. The first day of construction took place on April 11, 2003 from 7:00 a.m. to 3:00 p.m.. It is located on the southbound main lane on Highway 59, just north of Sugar Creek Boulevard. The second day of construction took place on April 19, 2003 from 6:00 a.m. to 11:30 a.m.. It is located on the same section of highway on the northbound main lane of Highway 59. A schematic that illustrates the field study layout on Highway 59 is shown in Figure 5.1. An overall view is portrayed in Figure 5.2, and the location of the field site is shown in Figure 5.3. This location was selected due to the relatively high truck traffic the pavement would encounter. Houston has a history of poorly performing pavements, due to environmental conditions and aggregate sources, which together provides an excellent basis for evaluating the benefits of fibers.

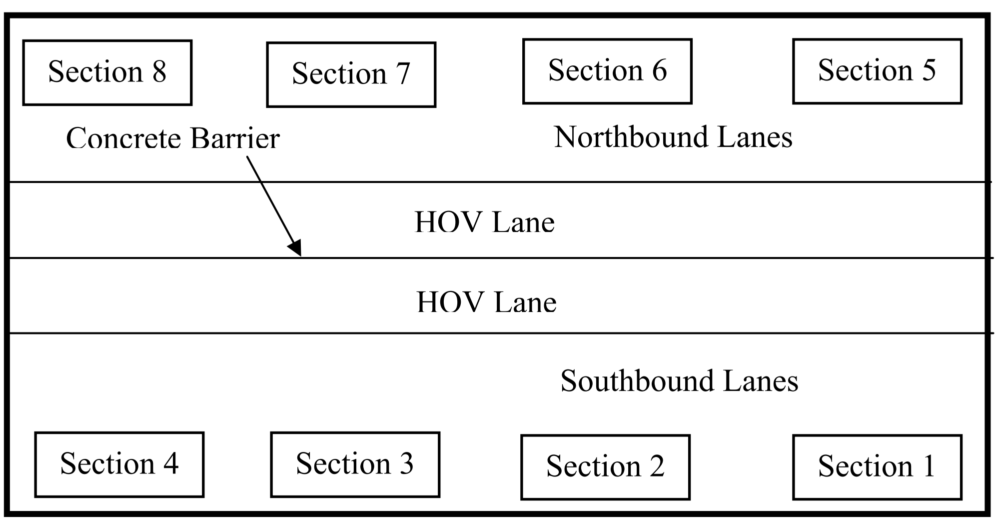


Figure 5.1: Field Study Layout

The concrete pavement has typical properties for a heavily traveled highway. The pavement is 15 in. thick, 24 ft wide, and has a total length of approximately 870 ft. It should be noted that there has been no pavement constructed at the beginning of the field site, reducing the amount of restraint in the first and fifth section. The primary reinforcement is Grade 60 reinforcing steel, which is placed in two equally spaced layers in the pavement. For the longitudinal direction, the reinforcement consists of No. 6 bars spaced at 8.25 inches. For the transverse reinforcement, No. 5 bars are spaced at 36 inches. Since the addition of fibers is not intended to increase the design strength of the pavement but merely to enhance its durability, the existing reinforcement and thickness design for the CRCP test section were not changed.

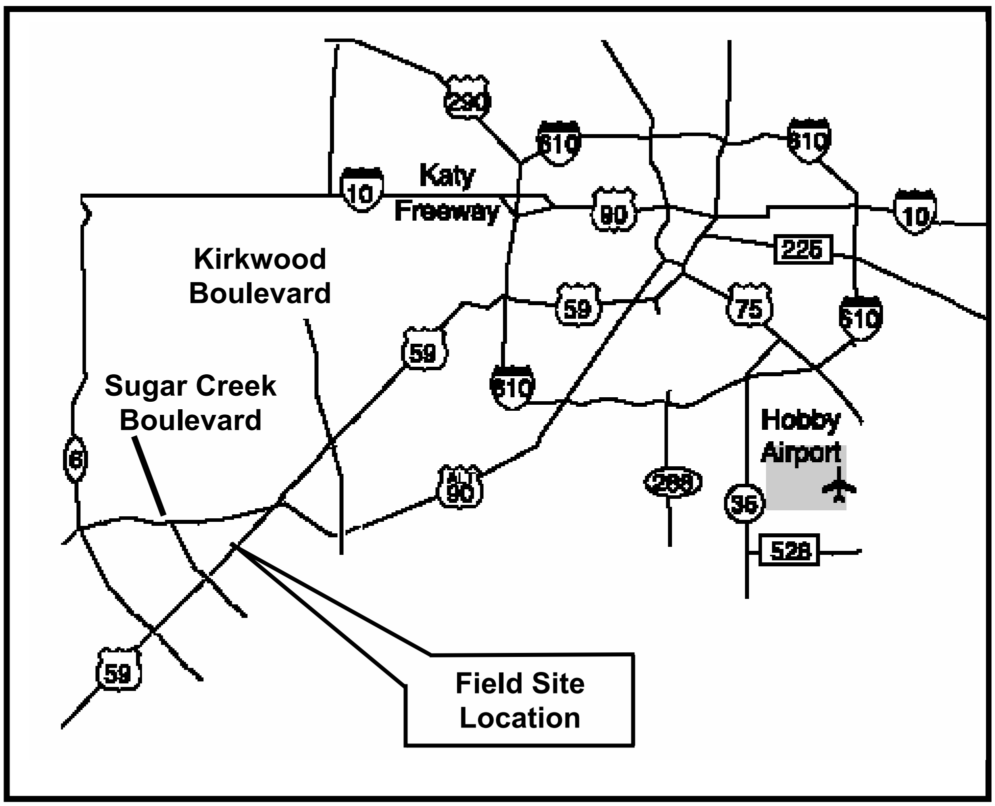


Figure 5.2: Map of Field Site (Otero-Jimenez et al., 1992)

Figure 5.3: Overall View of Field Site

5.2 Materials

The primary materials selected for the field study are identical to those used in the preliminary field evaluation, including cement, fine and coarse aggregate, and chemical admixtures. The materials were kept constant to minimize the number of variables between each study. There was, however, additional length in the second study that enabled a wider variety of fiber types and dosages to be evaluated. The same criteria were used to determine which fibers would be used. There was already valuable information obtained from the preliminary study which provided positive guidance for the material selection in this study. Once again, fiber manufacturers were consulted to determine the most viable fibers for this application. Table 5.1 lists the types of fibers that were selected and gives a general description. The SF1, SnF1, and SnF2 fibers are shown in Section 3.2.5. The additional fibers used in this study are shown in Figure 5.4 and Figure 5.5.

Table 5.1: Fiber Designation for Field Evaluation

Fiber Designation	Description	Length (mm)	Aspect Ratio
SF1	Steel—Collated Hooked-End	65	60
SF2	Steel—Corrugated	50	44
SnF1	Synthetic—Monofilament	90	40
SnF2	Synthetic—Collated-Fibrillated	< 30*	NA*
SnF3	Synthetic—Microfilament	20	5440

*The SnF2 fiber is graded and does not conform to a specific length or aspect ratio.

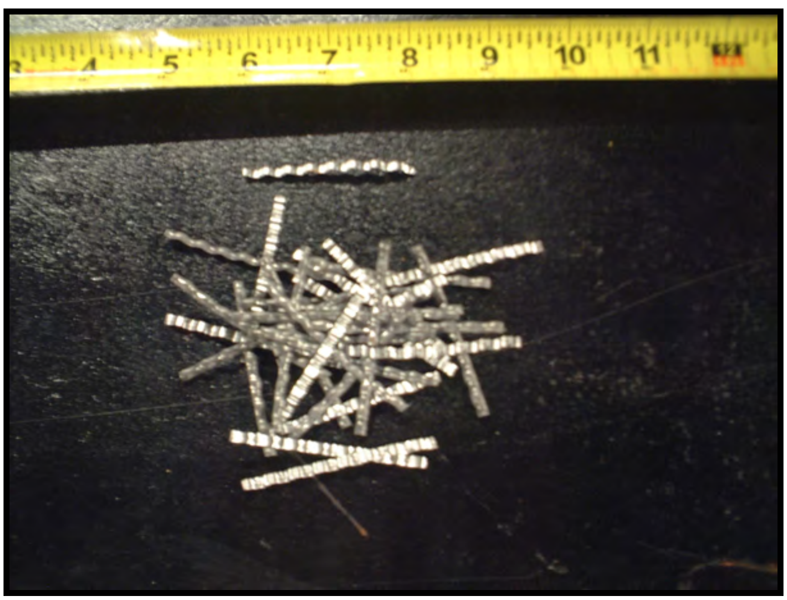


Figure 5.4: SF2 Fiber Type

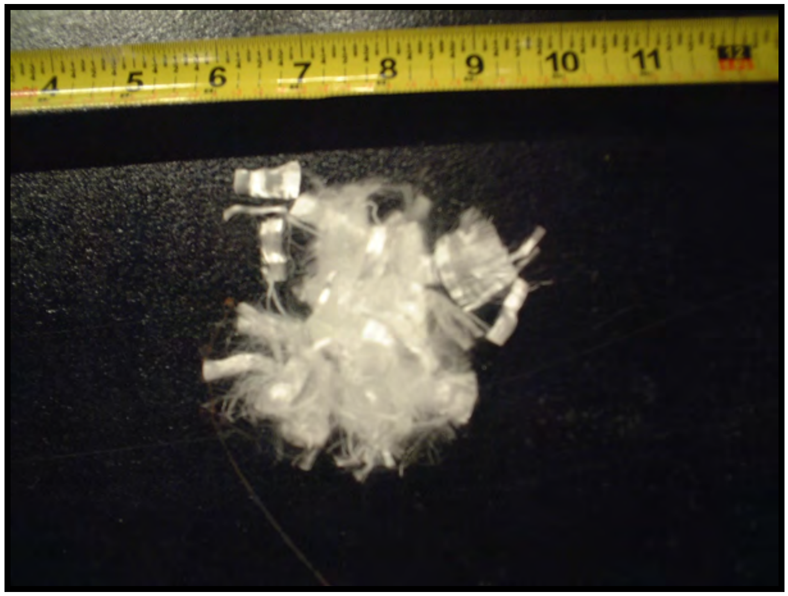


Figure 5.5: SnF3 Fiber Type

5.3 Mixture Proportions

The concrete mix proportions for the second field study have changed slightly because of the method of construction. Since the concrete in this study is being placed with a slipform paver, it must be much stiffer than before. The desired slump in the preliminary study was 3.5 in., whereas the slump in this study ranges from 1.5 to 3.0 inches. Preliminary concrete mixes were prepared in the laboratory to evaluate the effects that the selected fibers will have on the stiffer mix. The majority of the results were similar to the previous field evaluation. Consequently, the same modifications were recommended to the concrete supplier. This includes replacing 100 lb/yd^3 of coarse aggregate with 100 lb/yd^3 of fine aggregate for each section containing fibers to help maintain the concrete's workability, finishability, and packing of the concrete matrix. The amount of water and air entraining agent was also slightly increased for sections containing fibers to accommodate the additional stiffness. Table 5.2 shows the average concrete mixture proportions that were used for each test section.

Table 5.2: Typical Mixture Proportions for Field Evaluation

Section	Mixture Designation*	Cement	Fly Ash	Coarse Aggregate	Fine Aggregate	Water	Water Reducer	Air Dosage
1	SnF1-4	406 pcy	135 pcy	1836 pcy	1277 pcy	198 pcy	24 oz/yd^3	1.7 oz/yd^3
2	SF1-25	406 pcy	135 pcy	1836 pcy	1277 pcy	201 pcy	24 oz/yd^3	1.7 oz/yd^3
3	SF2-27.5	406 pcy	135 pcy	1836 pcy	1277 pcy	197 pcy	24 oz/yd^3	1.7 oz/yd^3
4	Control	406 pcy	135 pcy	1936 pcy	1173 pcy	193 pcy	24 oz/yd^3	1.7 oz/yd^3
5	SF1-40	406 pcy	135 pcy	1836 pcy	1270 pcy	195 pcy	24 oz/yd^3	1.7 oz/yd^3
6	Control	406 pcy	135 pcy	1936 pcy	1167 pcy	186 pcy	24 oz/yd^3	1.7 oz/yd^3
7	SnF2-1.5	406 pcy	135 pcy	1836 pcy	1270 pcy	198 pcy	24 oz/yd^3	1.7 oz/yd^3
8	SnF3-0.5	406 pcy	135 pcy	1836 pcy	1270 pcy	197 pcy	24 oz/yd^3	1.7 oz/yd^3

*Indicates Fiber Type and Dosage (i.e., SF1-25 contains 25 pcy of steel collated, hooked-end fibers)

5.4 Mixing and Paving Procedure

The concrete used for the field evaluation was produced in the same central mix plant described in Section 3.2. Once again, the addition of fibers did not have a significant effect on the mixing process. The fibers were added to the mixer using the same methods as the preliminary field study, depending on the type of bags containing them.

The concrete was transported to the field site using 10 yd^3 dump trucks. The time of transportation varied between 10 and 30 minutes, affecting the slump at the time it reached the site. The paving method used for the mainline field evaluation was a slipform paver as described in Section 2.8.1. Once the dump trucks reached the site, the concrete was loaded onto a conveyor system, releasing the concrete into the middle of the section. The slipform paver is equipped with internal vibrators and an auger which are used to spread the concrete across the section to facilitate consolidation and finishing. A mechanical straight-edge was then used to apply the initial finish to the surface. This enabled the finishers to apply the final finish with a bull float. The surface texturing was next applied using a standard carpet drag. Finally, skid resistance was applied by tining the pavement's surface. A curing compound was then sprayed onto the surface. Each of the main construction sequences is shown in Figure 5.6 through 5.10. The effects that fibers had on the paving operation are discussed later in this chapter.

Figure 5.6: Placement of Concrete Using Slipform Paver

Figure 5.7: Vibrating and Placement of Concrete

Figure 5.8: Finishing Process

Figure 5.9: Mechanical Application of Tining

Figure 5.10: Application of Curing Compound

5.5 Testing Program

Concrete specimens were obtained from one truck in each pavement section. Specimens include twenty 6-in. x 12-in. cylinders, two 4-in. x 8-in. cylinders, and three 6-in. x 6-in. x 20-in. flexural beams that will be tested for compression, splitting tension, modulus, flexural toughness, coefficient of thermal expansion, and permeability at a combination of 7 and 28 days. The results of these tests will be used to evaluate the effects that fiber type and dosage have on a typical pavement mix. On the day following casting, each specimen was transported to the Construction

Materials Research Group Laboratory in Austin to cure until it could be tested. The laboratory tests are in the process of being conducted and will be presented later in a future thesis by David Sutfin. Figure 5.11 and Figure 5.12 illustrate the test section dimensions and indicate the location of iButton instrumentation.

Fresh concrete properties and ambient conditions were also measured while the specimens were being cast. Fresh properties include slump, air content, unit weight, and concrete temperature. Ambient conditions include relative humidity, wind speed, and ambient temperature. The results for each of these are presented and discussed in this section.

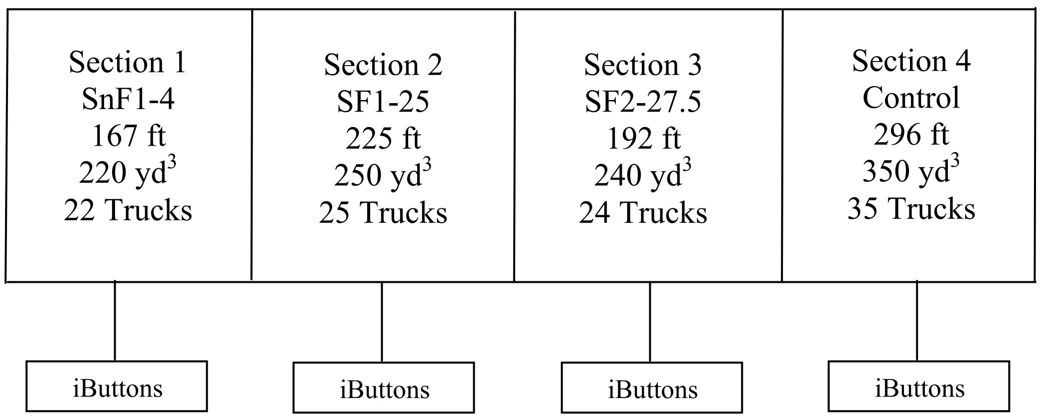


Figure 5.11: Layout of Southbound Test Pavement

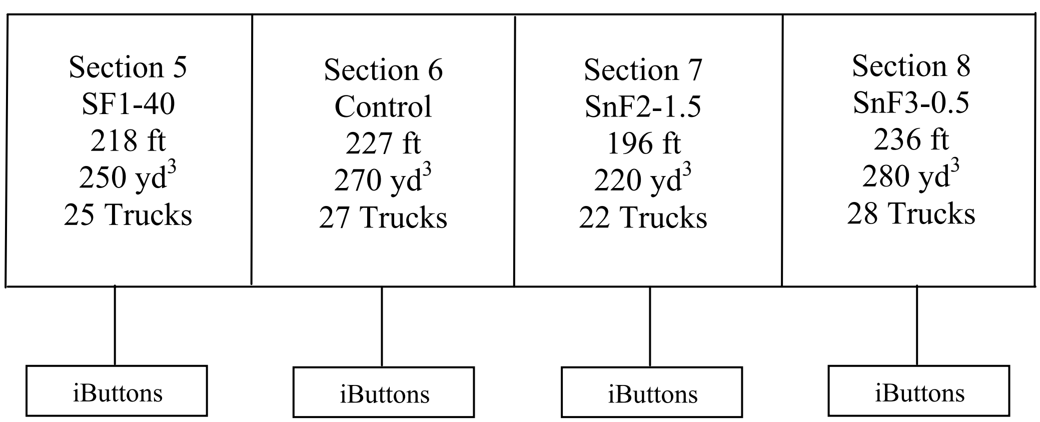


Figure 5.12: Layout of Northbound Test Pavement

5.5.1 Fresh Concrete Properties

Fresh concrete properties were measured as described in Section 3.5.1. Fibers may significantly influence the fresh properties of concrete, depending on the fiber type and dosage. Readings were taken from each section to document slump, concrete temperature, air content, and unit weight. Table 5.3 summarizes the measurements that were taken throughout the day. The results demonstrate a good consistency and quality control in the pavement mix.

Table 5.3: Summary of Fresh Concrete Properties

Section	Mixture Designation	Time Placed	Concrete Temperature (°F)	Slump (in.)	Air Content (%)	Unit Weight (lb/yd3)
1	SnF1-4	8:40 AM	74	1.5	4.5	145.4
2	SF1-25	10:26 AM	74	2.25	NA	148.6
3	SF2-27.5	11:45 AM	73	2.75	4.75	148.6
4	Control	2:05 PM	75	1.5	NA	148.6
5	SF1-40	6:52 AM	81	1.75	3.75	146.6
6	Control	8:10 AM	80	2	4.25	140.6
7	SnF2-1.5	9:21 AM	81	2	4.75	142.6
8	SnF3-0.5	10:37 PM	81	2.25	5	143.6

5.5.1.1 Slump

The slump was obtained using conventional methods, as described in Section 3.5. The desired slump for the second field study has been reduced to the range of 1.5 to 3.0 in., as opposed to 3.5 in. used in the preliminary field study. The main reason for this difference is related to the method of construction. Concrete placed with a slipform paver requires a stiffer mix because of the lack of formwork. The slump recorded during the field study ranged from 1.5 to 2.75 in., depending on the time of day as well as fiber type and dosage.

5.5.1.2 Air Content

A pressure meter and roller meter were used in combination to determine the air content of the concrete. The desired air content for this content was 3.5 percent. Air content is not critical in this region due to the lack of freeze-thaw damage. The air content recorded during the field study ranged from 3.75 to 4.75 percent. It should be noted that air content readings were not taken on two of the test sections due to a malfunction in the pressure meter.

5.5.1.3 Unit Weight

Unit weight was measured using a standard 0.25 ft^3 cylinder. Fibers do not greatly affect the unit weight of the concrete. The recorded unit weight ranged from 140.6 to 148.6 lb/ ft^3, depending on the section. The average value for unit weight was determined to be 145.6 lb/ ft^3.

5.5.1.4 Concrete Temperature

The ambient temperature encountered on each day of construction was relatively mild and constant throughout the day. As a result, the concrete did not experience a wide range of temperatures during construction. The concrete temperature ranged from 73 to 75º F on the first day of construction. The concrete placed on the second day of construction ranged from 80 to 81º F. The variation in concrete temperature between the two different days is attributed to the difference in ambient temperature.

5.5.2 Ambient Conditions

Readings were taken from each section to document relative humidity, wind speed, and ambient temperature. Measurements were obtained with similar instrumentation described in Section 3.5.2 and are listed in Table 5.4. On the first day of construction, the temperature was relatively low, but it increased considerably as the day progressed. The second day did not experience a large fluctuation in temperature. As expected, the relative humidity is indirectly proportional to the ambient temperature.

Table 5.4: Summary of Ambient Conditions

Section	Mixture Designation	Time Placed	Ambient Temperature (ºF)	Relative Humidity (%)	Wind Speed (ft/min)
1	SnF1-4	8:40 AM	62.8	71.2	450
2	SF1-25	10:26 AM	71.1	52.1	430
3	SF2-27.5	11:45 AM	75.1	39.4	60
4	Control	2:05 PM	78.8	29.3	290
5	SF1-40	6:52 AM	72.1	99.9	250
6	Control	8:10 AM	71.1	99.9	100
7	SnF2-1.5	9:21 AM	75	88.0	150
8	SnF3-0.5	10:37 PM	78	78.8	210

5.5.3 Constructability

One of the main benefits of the field evaluation was to evaluate the effect of constructing CRCP containing fibers. The construction method used for this field study is completely different than the preliminary study; consequently, the fibers may have a completely different affect on constructability. The main concern with placing fibers in CRCP is the low slump in the range of 1.5 to 3.0 inches. This section discusses the observations made throughout the construction of the test pavement.

5.5.3.1 Control Sections

The control sections were constructed without any modifications being made to the concrete mix design. Neither control section experienced problems related to constructability or finishability. As the concrete was dumped onto the conveyor belt, the concrete flowed effortlessly across the pavement without segregation in concrete composition. This is shown in Figure 5.13. The concrete was easily finished by the mechanical devices and required little effort by the hand finisher. A typical surface profile for a control section is shown in Figure 5.14.

Figure 5.13: Nice Uniformity during Placement (Section 1—Control)

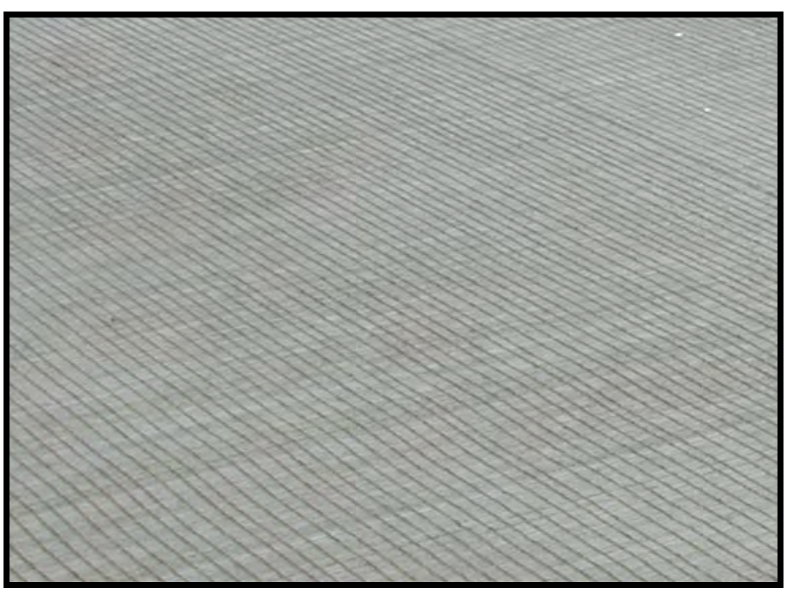


Figure 5.14: Surface Profile (Section 1—Control)

5.5.3.2 SF1 Sections

The sections containing hooked-end steel fibers (SF1-25 and SF1-40) experienced varying levels of difficulty in construction depending on the fiber dosage. The SF1-25 section was relatively easy to construct, while the SF1-40 section was somewhat of a struggle during placement. Initially, the concrete placed in the SF1-40 section had a decent slump but did not flow well across the pavement, as shown in Figure 5.15. In order to facilitate the spreading of the concrete, the water content was increased. This modification was reasonably effective and provided the workers an easier surface to finish. A uniform distribution of fibers was noted in the concrete that maintained a proper concrete composition. This is shown in Figure 5.16.

The mechanical finisher did a fairly good job of laying the fibers down into the concrete to produce a smooth surface. The surface remained relatively smooth throughout the finishing process until the tining operation began. Figure 5.17 illustrates that the fibers were pulled out of the pavement by the tining machine. This phenomenon was also typical in the preliminary field study for this fiber type. The tining process causes fibers to protrude from the surface, as shown in Figure 5.18. The majority of fibers are oriented parallel to the pavement surface and will gradually be removed by environmental conditions and traffic. It should be noted that the SF1 fibers did not cause any damage to the paver during construction.

Figure 5.15: Discontinuity in Concrete during Placement (Section 5—SF1-40)

Figure 5.16: Close-up of Concrete Composition (Section 2—SF1-25)

Figure 5.17: Fibers Disturbed during Tining (Section 2—SF1-25)

Figure 5.18: Surface Profile (Section 2—SF1-25)

5.5.3.3 SF2 Section

The concrete placed in the SF2-27.5 section performed very well with regards to constructability. It maintained a nice slump and flowed very nicely throughout the pavement. A uniform distribution of fibers was noted in the concrete that maintained a suitable concrete composition. This is shown in Figure 5.19.

The mechanical finisher easily embedded the fibers down into the concrete to produce a smooth surface. The surface remained moderately smooth throughout the finishing process, even during the tining operation. Figure 5.20 illustrates that the fibers remained embedded during the tining process. As a result, very few fibers can be seen in the final pavement surface. This smooth surface is shown in Figure 5.21.

Figure 5.19: Close-up of Concrete Composition (Section 3—SF2-27.5)

Figure 5.20: Fibers Undisturbed during Tining (Section 3—SF2-27.5)

Figure 5.21: Surface Profile (Section 3—SF2-27.5)

5.5.3.4 SnF1 Section

The concrete placed in the SnF1-4 section performed inconsistently with regards to constructability. It flowed very easily throughout the pavement and maintained a uniform distribution of fibers in the concrete, as shown in Figure 5.22 through Figure 5.24. The SnF1 fiber did experience a problem that no other section encountered: a considerable amount of the SnF1 fibers was carried over into the subsequent sections. This may cause an inconsistency in the field study, but it should not really affect a typical paving operation, because it is likely that there would only be one fiber type throughout the pavement.

The SnF1 fiber sections also experienced many problems in being finished. The mechanical finisher was unable to hide the fibers in the concrete. This problem was also encountered during the hand finishing with the bull float, as shown in Figure 5.25. The lack of finishing was exacerbated by the tining operation. The teeth from the rake seemed to drag the fibers out of the pavement. This was also experienced in the preliminary field study for this fiber type. The lack of finishability is shown in Figure 5.26 and Figure 5.27.

Figure 5.22: Nice Uniformity during Placement (Section 1—SnF1-4)

Figure 5.23: Smooth Flow during Placement (Section 1—SnF1-4)

Figure 5.24: Close-up of Concrete Composition (Section 1—SnF1-4)

Figure 5.25: Fibers Disturbed by Bull Float (Section 1—SnF1-4)

Figure 5.26: Fibers Disturbed during Tining (Section 1—SnF1-4)

Figure 5.27: Surface Profile (Section 1—SnF1-4)

5.5.3.5 SnF2 Section

The concrete placed in the SnF2 section performed relatively well with regards to constructability. It maintained a sufficient slump that facilitated a smooth flow throughout the pavement. A uniform distribution of fibers and aggregate was also noted in the concrete, as shown in Figure 5.28. The fibers performed beautifully during the mechanical finishing and finishing with the bull float. At this point, it was very difficult to distinguish this section from a control section, as shown in Figure 5.29. However, the tining operation seemed to drag the fibers out of the surface. In some cases, the fibers formed into balls at the surface, as shown in Figure 5.30. It should be noted that this phenomenon did not occur in the preliminary field study for the same fiber type and dosage.

Figure 5.28: Close-up of Concrete Composition (Section 7—SnF2-1.5)

Figure 5.29: Smooth Surface during Initial Finishing (Section 7—SnF2-1.5)

Figure 5.30: Surface Profile (Section 7—SnF2-1.5)

5.5.3.6 SnF3 Section

The concrete placed in the SnF3 section was constructed relatively easily. The concrete maintained a nice slump and flowed smoothly during the pour. It should be noted that, in some instances, the fibers seemed to act as a net and cause segregation between the coarse aggregate and the cement paste. This segregation is shown in Figure 5.31. It is clearly evident that a great deal of coarse aggregate has settled to the bottom during transportation, leaving it at the top of the concrete as it is delivered from the conveyor belt. This could potentially cause damage if the problem is not addressed.

Because of the tiny size of the fibers [20 mm (.79 in)], it was very difficult to notice them during the finishing process. Even the tining operation did not seem to pull the fibers out of the pavement. This made the SnF3 section difficult to distinguish from a control section. The lack of visibility of the fibers at the surface is shown in Figure 5.32.

Figure 5.31: Segregation of Coarse Aggregate (Section 8—SnF3-0.5)

Figure 5.32: Surface Profile (Section 8—SnF3-0.5)

5.6 Monitoring of Test Sections

Monitoring is planned at several intervals, similar to the preliminary field study, to evaluate the condition of the test pavement. This will include the documentation of thermal data, crack spacing, crack width, and spalling. In addition to the test pavement, one lane of pavement was constructed alongside each test lane approximately 1 week earlier and was also monitored. This section will only discuss the general condition of the existing pavement lanes and the initial thermal data readings. The results of the crack spacing, crack width, and spalling in the test pavement lanes will be reported in a future thesis by David Sutfin.

5.6.1 Condition of Existing Adjoining Lanes

There is a lane of pavement that has been constructed directly adjacent to each test pavement lane. Each lane was constructed in the beginning of April, between 2 and 3 weeks before the field study. The concrete was placed by hand with a concrete spreader, similar to the preliminary field study described in Chapter 3. Consequently, the desired slump for this concrete mix is approximately twice that of the field study, which was constructed with a slipform paver. This is the only difference between the concrete mix designs. The exact same materials were used but in different proportions.

The condition of the existing lanes was monitored and recorded on April 23, 2003. The main issues included pop-outs and minor crack-to-tine groove surface spalls with crack development. There were a considerable number of cracks that had already formed throughout the pavement. The spacing varied along the section. The cracks were spaced as closely as 3 ft in some instances. However, other places in the pavement did not experience a crack for over 50 ft. It is too soon at this point to evaluate the crack development. Future monitoring will continue to investigate the crack development of the existing lanes.

There is evidence of minor crack edge popout spalling already beginning to occur in the existing lanes. The majority of spalling has occurred in the northbound lane. It should be noted that this lane has yet to experience traffic load, therefore, popouts and minor crack-edge spalls have been caused entirely by environmental conditions. It is important to note that there is no evidence of horizontal cracking or leamination. An example of a typical spall is shown in Figure 5.33 and Figure 5.34. The spalling is clearly in its early stage of development. At this point, it poses no hazard and does not warrant repair. However, once the pavement is opened to traffic, the spall could accelerate and require restoration.

Figure 5.33: Evidence of Spalling in Existing Northbound Lane

Figure 5.34: Evidence of Spalling in Existing Northbound Lane

5.6.2 Thermal Effects

Thermal effects were measured using iButton technology and data acquisition equipment. This section discusses the temperature profile within the concrete. It also illustrates the temperature gradient created throughout the pavement.

5.6.2.1 iButton Installation

iButtons were installed to monitor the temperature within the concrete pavement. iButtons are small computer chips contained in a 16mm (.63 in) stainless steel can, conventionally used in the food transportation industry to monitor the temperature in a delivery truck. However, they have recently been utilized in concrete applications. There is currently no documentation that specifies the assemblage for concrete purposes. However, extensive testing has been performed by UT Austin's Center for Transportation Research (CTR), studying different assembly methods and verifying the accuracy of the iButtons. The research team worked closely with CTR to ensure proper assemblage and placement in the concrete.

iButtons are designed to transfer data through a receptor. However, concrete pavement requires the iButtons to be imbedded into the concrete and prohibits access to the iButton with a receptor. As a result, the iButton must have a leader wire that can extend beyond the pavement to obtain readings. This wire was soldered to each side of the iButton, as shown in Figure 5.35. The iButton assemblage was then covered in tool dip for protection, as shown in Figure 5.36.

Figure 5.35: Wire Connection to iButton

Figure 5.36: Tool Dip Application

Preliminary tests were performed to evaluate the accuracy of the iButtons. This was accomplished using a water bath that can vary in temperature. Three iButtons were placed in the water bath for evaluation. The water temperature was initially set to room temperature at 23º C (73º F), then lowered to 10º C (50º F), and finally increased to 59º C (138º F). These temperatures were chosen because they exceed the limits that concrete generally experiences. The results of the iButton comparison are illustrated in Figure 5.37. It should be noted that the iButton readings do not significantly deviate from the controlled temperature throughout the test and prove that the iButtons maintain accurate readings for the desired range of temperature.

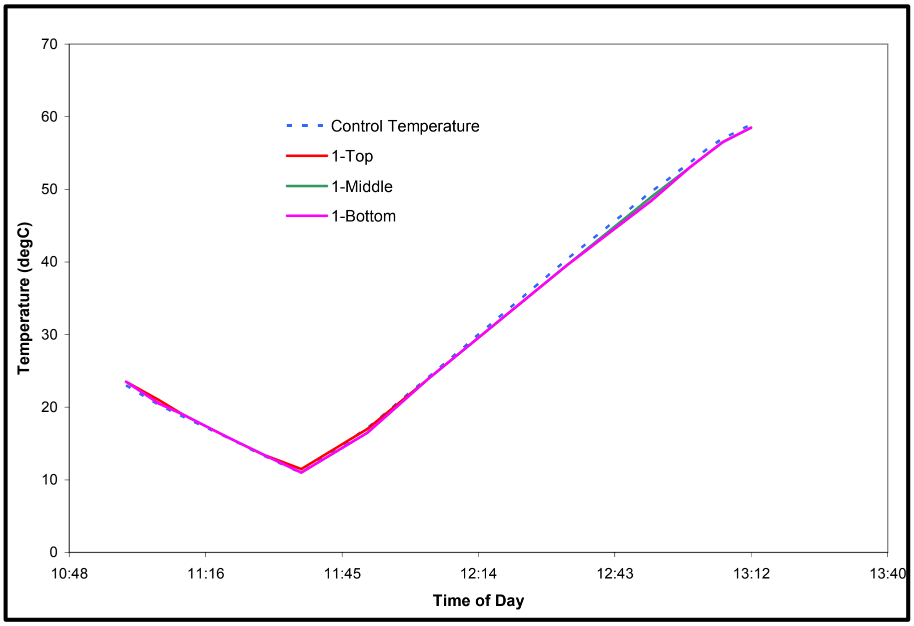


Figure 5.37: Evaluation of iButton Accuracy

The iButtons were installed in the concrete pavement the day before the concrete was placed. The wire extending from the iButton was retrieved from the edge of the pavement after the concrete was finished and then coiled into a PVC pipe, as shown in Figure 5.38, which would allow for future data acquisition. Ambient iButtons were mounted to a concrete barrier located on the jobsite. One iButton was placed on the outside of the barrier, while another was placed underneath the barrier in the lifting area in order to capture the temperature in the shade, as shown in Figure 5.39. The iButtons used to measure the concrete temperature were placed in one row of iButtons for each test section. They were installed in two sets at the midpoint of each pavement section—one in the middle of the cross section and the other 2 ft from the side. Each set of iButtons contains three iButtons placed at three different layers. The top layer is 1.5 in. from the surface, the middle layer is at the mid-depth of the pavement, and the bottom layer is 1.5 in. above the subgrade. This arrangement was chosen to demonstrate the temperature gradient and is shown in Figure 5.40. One of the main concerns with the iButton placement was damage being caused by the slipform paver during construction. A rule of thumb for slipform paving is that anything 1 in. below the pavement surface will not be damaged. This concept was successfully followed, because there was no damage reported to any iButton during the construction, even though heavy pressure from the concrete was exerted against the iButtons, as shown in Figure 5.41.

Figure 5.38: Layout of Ambient iButtons

Figure 5.39: Typical iButton Placement

Figure 5.40: iButton during Construction

Figure 5.41: Retrieval of iButton Wire after Construction

5.6.2.2 Concrete Temperature Profile

The thermal data demonstrate the expansion that the concrete experiences from heat of hydration and ambient conditions. The significance of the heat of hydration is described in Section 4.6.2. The ambient conditions experienced during this field study were mild and did not significantly influence the heat of hydration. Figure 5.42 and Figure 5.43 illustrate typical temperature profiles for the southbound paving portion of the field study. These figures demonstrate that the concrete temperature in the middle and top of the section do not change much between the different transverse locations in the pavement. However, the temperature in the bottom of the pavement is much more affected by the transverse location. This is attributed to the fact that the bottom of the pavement in the middle of the section is isolated from the ambient conditions in the middle of the pavement. However, towards the edge of the pavement, the bottom is more influenced by ambient temperature, as reflected in these graphs. Figure 5.44 and Figure 5.45 demonstrate a similar pattern for the northbound paving portion of the field study.

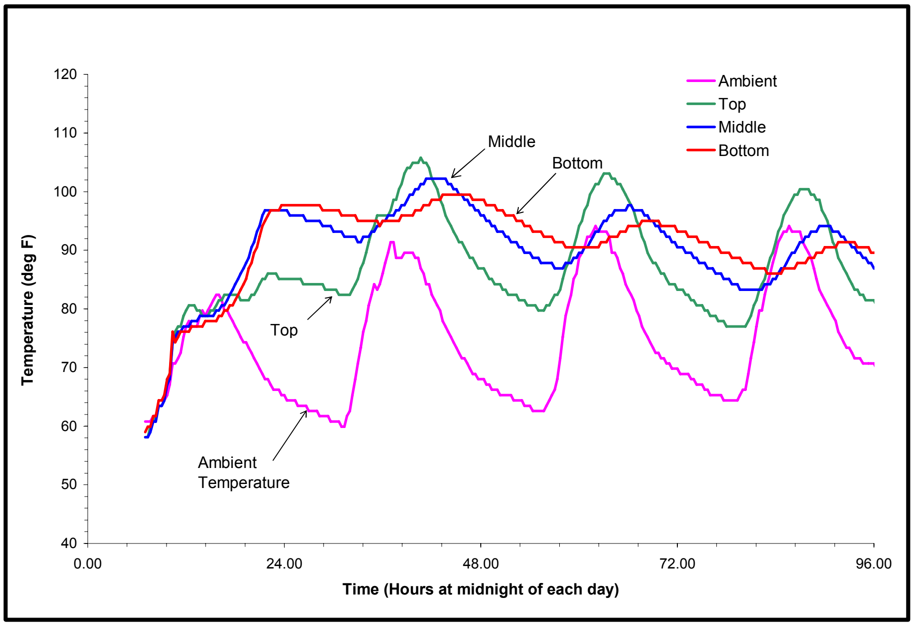


Figure 5.42: Typical Southbound Temperature Profile — Middle of Section

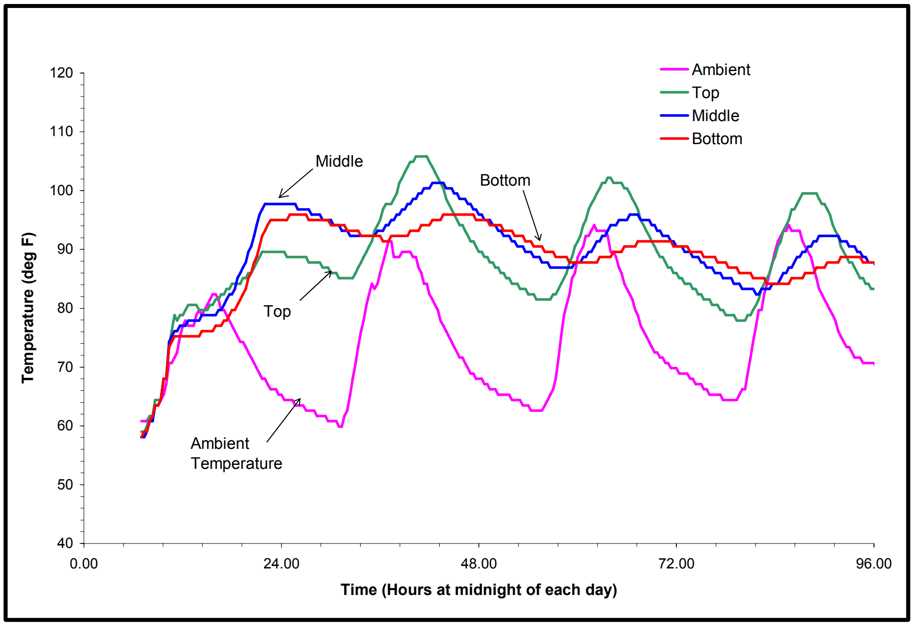


Figure 5.43: Typical Southbound Temperature Profile — 2 Feet from Edge

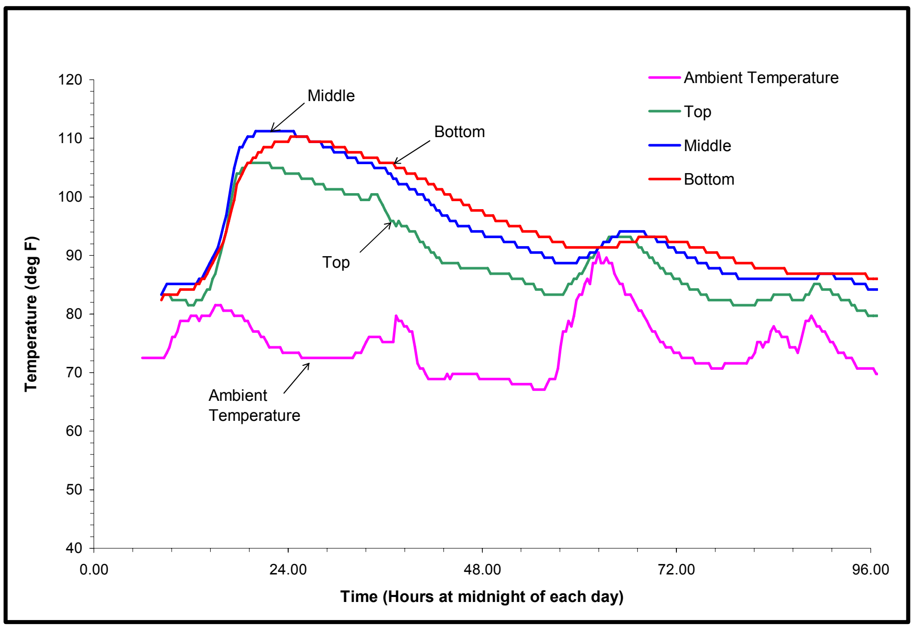


Figure 5.44: Typical Northbound Temperature Profile—Middle of Section

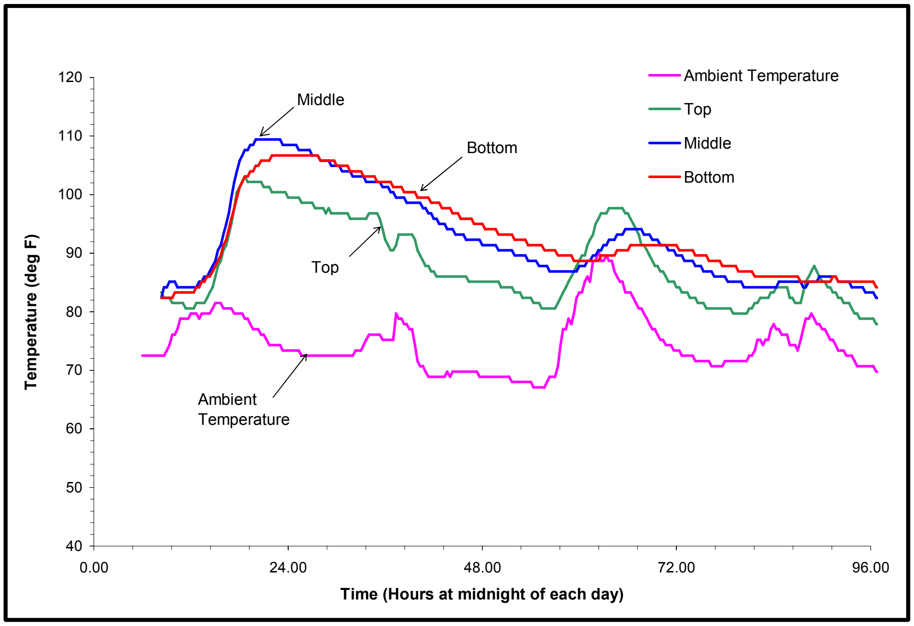


Figure 5.45: Typical Northbound Temperature Profile—2 Feet from Edge

5.6.2.3 Concrete Temperature Gradient

The temperature gradient throughout the slab depth and transverse section significantly influence the pavement's crack development. Figure 5.46 through Figure 5.49 illustrate the temperature gradient for the southbound portion of the field study in the middle of the section and 2 feet from the edge. There is little gradient shown for the concrete placed in the morning. However, the concrete placed in the afternoon experiences a variety of gradients throughout the pavement. This is attributed to the higher ambient conditions encountered in the afternoon, directly influencing the heat of hydration. It is shown that as the day progresses and the ambient temperature decreases, the concrete temperature in the bottom and mid-depth of the pavement continues to steadily increase due to the heat of hydration. However, the temperature in the top of the pavement is dominated by the ambient conditions and increases less rapidly. It should also be noted that the temperature varies along the transverse location of the pavement as well. The rate of temperature change steadily increases in the middle of the pavement lane, which is once again attributed to the heat of hydration. However, the edge of the pavement does not increase at the same magnitude. The reason for this is that the middle portion of the pavement experiences a higher heat of hydration than the edge due to the increased volume of surrounding concrete. The extreme temperature ranges found in the center of the slab segment reinforce the theory that cracks form in the center of the slab and propagate outwards. Similar observations were made for the northbound pavement lanes shown in Figure 5.50 and Figure 5.51.

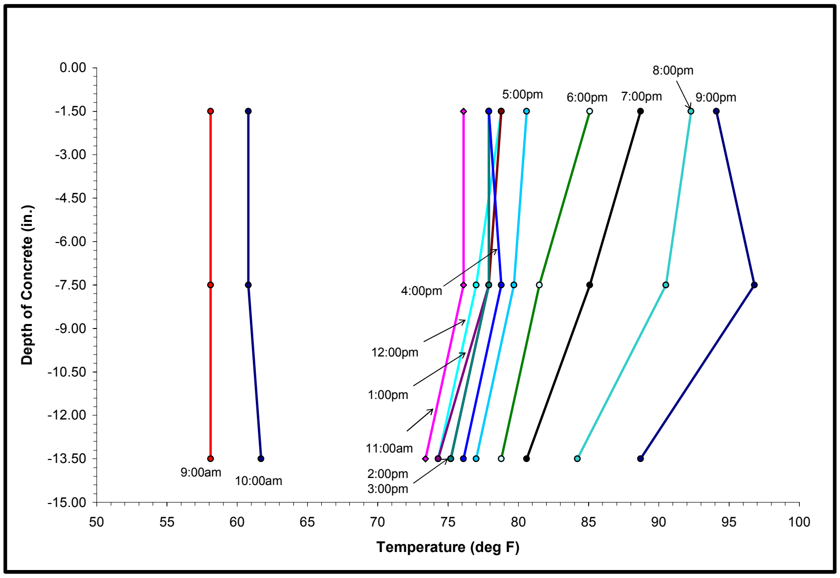


Figure 5.46: Typical Gradient Placed in Morning—Middle of Section

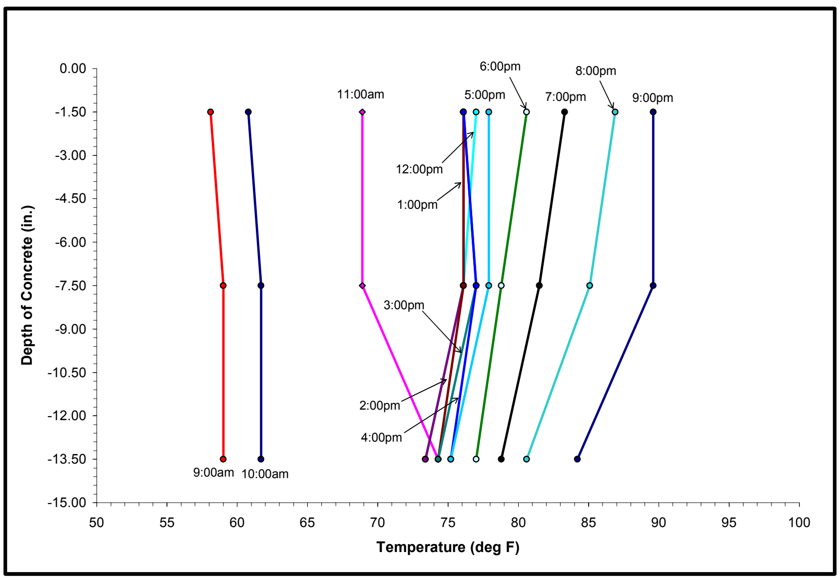


Figure 5.47: Typical Gradient Placed in Morning—2 Feet from Edge

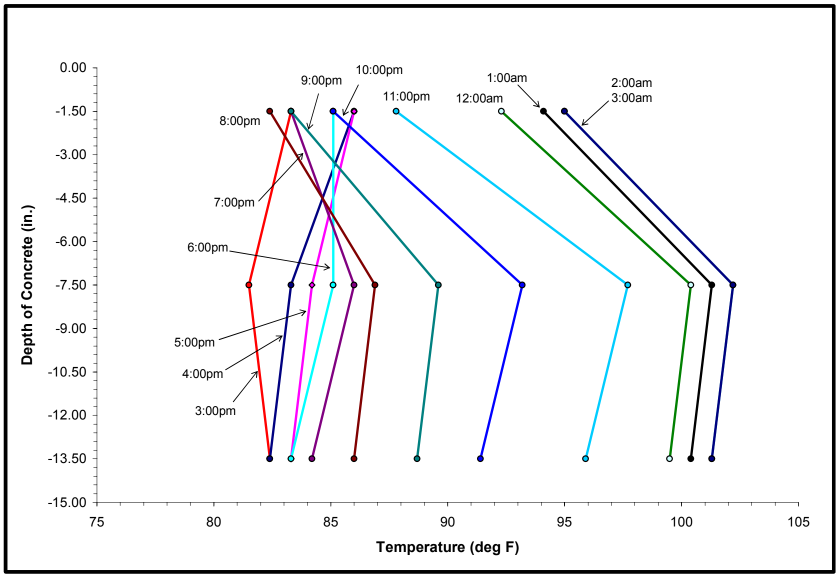


Figure 5.48: Typical Gradient Placed in Afternoon—Middle of Section

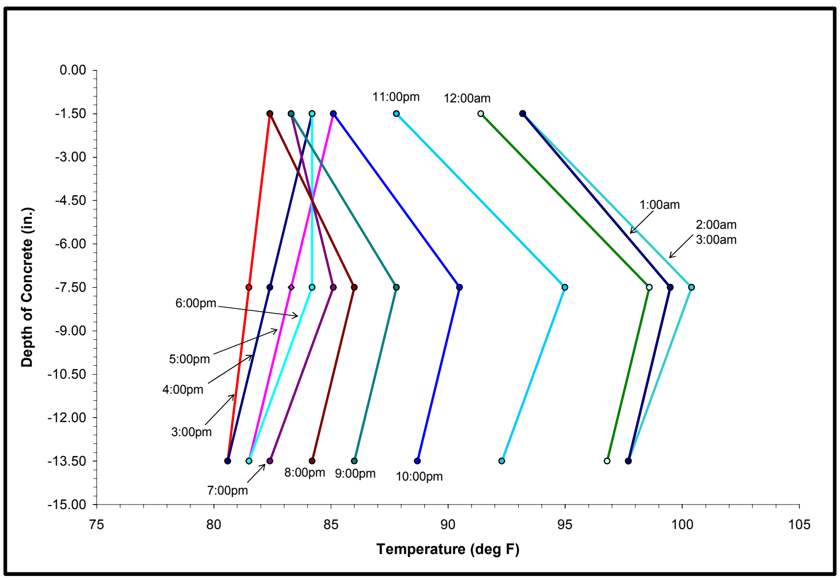


Figure 5.49: Typical Gradient Placed in Afternoon—2 Feet from Edge

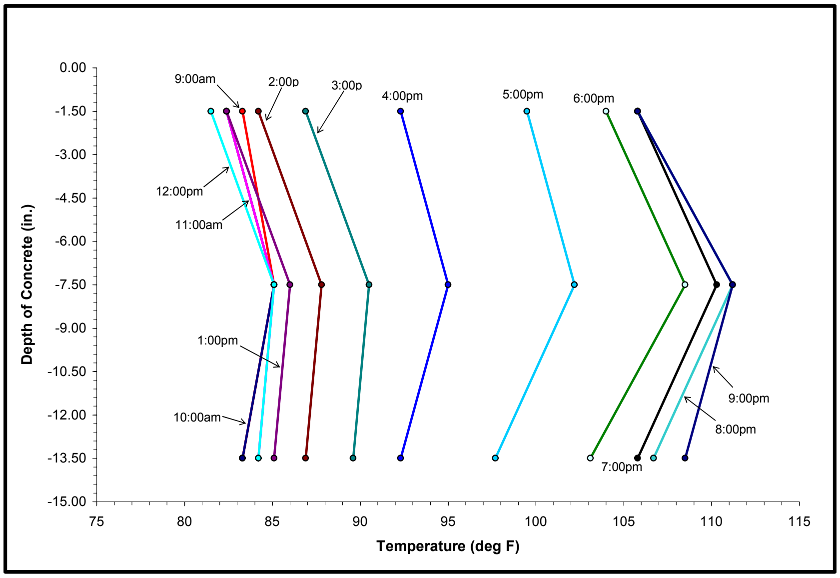


Figure 5.50: Northbound Thermal Gradient—Middle Section

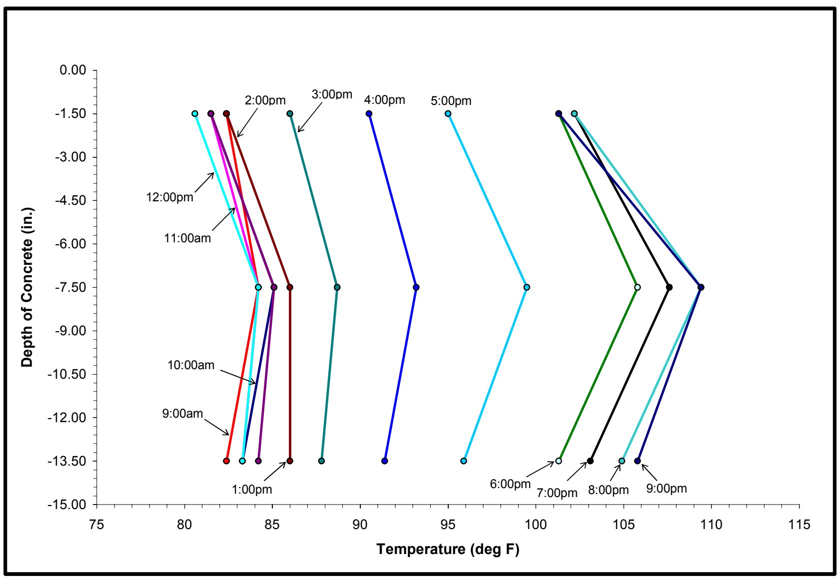


Figure 5.51: Northbound Thermal Gradient—2 Feet from Edge

5.7 Materials

The materials chosen for the field test were typical materials used for CRCP in the Houston area and known to have poor long-term spalling resistance. Because this field test was added on to an existing project already underway, the only materials capable of being changed were the fiber types. The project was already utilizing poor performing river gravel, so modifying any of the other raw materials was deemed unnecessary. As stated in the previous chapter, the materials used in the laboratory study are the same as those used in the field evaluation in order to provide more accurate correlations between properties measured in the lab and the actual performance monitored in the field.

5.7.1 Cementitious Material

The cement used in this field test was classified as a Type I/II, as specified by ASTM C 150. Class C fly ash was also used and conforms to ASTM C 618. Both materials are common for CRCP construction in Texas and come from the same sources as the materials used in the lab study. The chemical compositions of these materials are presented in Table 5.5.

Table 5.5: Chemical Compositions of Cementitious Materials

Chemical Analysis	**Type I/II Cement (%)**	**Class C Fly Ash (%)**
SiO	20.1	-
SiO_2	-	32.44
Al_2O_3	5.0	19.06
Fe_2O_3	3.7	6.83
CaO	64.0	28.26
MgO	0.6	4.15
SO_3	-	2.14
Na_2O	NA	1.46
Loss on Ignition	1.3	0.10
Free Lime	0.9	-
Moisture Content	0.14	0.11
Physical Analysis		
Fineness	95.6	17.0
Specific Gravity	-	2.70
Autoclave Expansion	0.00	0.04
Potential Compounds		
C_3S	59.0	-
C_3A	7.1	-

5.7.2 Aggregates

The aggregates used in this study were typical materials used in the Houston area for CRCP construction. The coarse aggregate used was siliceous river gravel and a TxDOT designated Grade 2 coarse aggregate that also conforms to ASTM C 33 (Size 467). This aggregate has been known to provide poor spalling resistance for CRCP, making it an excellent material to be incorporated into this study. The fine aggregate used was a TxDOT designated Grade 1 natural river sand that also conforms to ASTM C 33. Table 5.6 lists the properties of the coarse and fine aggregates used in the field evaluation as well as the respective aggregate specifications found in TxDOT Item 421.2.

Table 5.6: Aggregate Properties

Property	Sieve Size	Coarse Aggregate (%)		Fine Aggregate (%)	
		TxDOT Spec. For Grade 2	Siliceous River Gravel (SRG)	TxDOT Spec. For Fine Agg.	Natural River Sand
Cumulative Percent Passing for each Sieve:	2"	100	100.0	-	-
	1 1/2"	95-100	100.0	-	-
	1"	-	92.0	-	-
	3/4"	35-70	84.6	-	-
	1/2"	-	62.5	-	-
	3/8"	10-30	26.7	100	100
	No. 4	0-5	3.3	95-100	99
	#8	-	-	100-80	92
	#16	-	-	50-85	69
	#30	-	-	25-65	38
	#50	-	-	10-35	11
	#100	-	-	0-10	2
Bulk Specific Gravity		-	2.58	-	2.62
Fineness Modulus		-	NA	-	2.89
Absorption Capacity (%)		-	0.7	-	0.6

5.7.3 Admixtures

Admixtures used in this field study included a water reducer/retarder and an air entraining agent. These admixtures are typical for use in CRCP in Texas. The low range water-reducing/retarding admixture conforms to ASTM C 494 (Type A or D). It is a refined lignosulfate salt suspended in an aqueous solution, and no calcium chloride was added throughout the manufacturing process. The air entraining admixture meets the requirements of ASTM C 260. It is formulated with a stabilized, modified resin surfactant that has a chloride content less than 0.05 percent.

5.7.4 Fibers

As with the lab study, the fibers to be used in the field study were chosen to encompass the most typical fibers currently available on the market. The fibers used vary in material, geometry, and cost to provide an unbiased, comprehensive evaluation of fiber reinforcement capabilities. The fiber types chosen are also the same as the ones implemented in the laboratory evaluations to provide consistency between the two studies. Table 5.7 lists the fiber types used in this study, as well as the material properties and geometries. This table also supplies the fiber designations which will be referenced throughout this chapter. Refer to Section 3.2.4 for images of the fiber types that were used in both laboratory and field investigations. SnF3 was the only fiber type that was evaluated only in the field, and an image of this fiber is presented in Figure 5.52.

Table 5.7: Fiber Designation

Fiber Designation	Description	Length (in)	Aspect Ratio
SF1	Steel—Collated Hooked-End	2.36	65
SF2	Steel—Corrugated	1.97	44
SnF1	Synthetic—Monofilament	1.57	90
SnF2	Synthetic—Collated-Fibrillated	< 1.18*	NA*
SnF3	Synthetic—Microfilament	0.79	5440

*The SnF2 fiber is graded and does not conform to a specific length or aspect ratio.

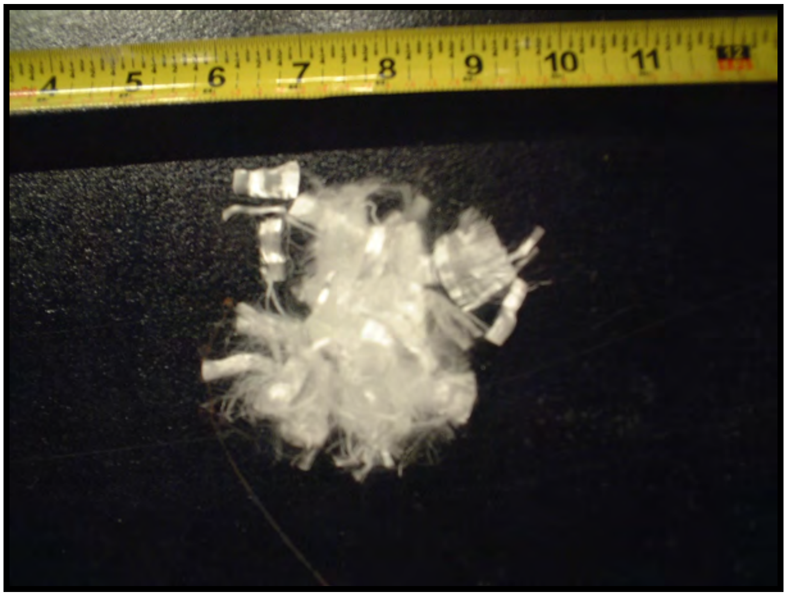


Figure 5.52: SnF3 Fiber Type

5.8 Mixture Proportions

The mixture proportions used for this field investigation were taken from the designs already being used for the construction previously completed on the field site. The proportioning designates the concrete as a TxDOT specified Class P mixture design per Item 421.9 in the TxDOT Standard Specifications Manual. This type of concrete is solely used for pavement construction and is typical for CRCP. The target slump was 1.5 to 3 in. and the target air content was 5 percent. This base mixture was then modified to better accommodate the addition of fiber reinforcement and ensure good workability and finishability. The main proportioning modification was replacement of 100 lbs of coarse aggregate with 100 lbs of fine aggregate whenever fiber reinforcement was used. This increase in fine aggregate content improves the overall workability and finishability of in-place concrete containing fibers.

Table 5.8 lists the average mixture proportions that were used for each test section. Unlike the laboratory study, the amount of water was changed per batch to obtain the desired workability. While this is not ideal, because it means each mixture has a different water-to-cement ratio, this is what is normally practiced in the field because using additional water is much cheaper than modifying the amount of water reducer used. Aside from the variation in water content, the mixtures utilized in the field sections were very similar to the mixture proportions used in the laboratory study.

Table 5.8: Average Mixture Proportions for Each Test Section

Section	Mixture Designation*	Cement	Fly Ash	Coarse Aggregate	Fine Aggregate	Water	Water Reducer	Air Dosage
1	SnF1-4	406 pcy	135 pcy	1836 pcy	1277 pcy	198 pcy	24 oz/yd3	1.7 oz/yd3
2	SF1-25	406 pcy	135 pcy	1836 pcy	1277 pcy	201 pcy	24 oz/yd3	1.7 oz/yd3
3	SF2-27.5	406 pcy	135 pcy	1836 pcy	1277 pcy	197 pcy	24 oz/yd3	1.7 oz/yd3
4	Control	406 pcy	135 pcy	1936 pcy	1173 pcy	193 pcy	24 oz/yd3	1.7 oz/yd3
5	SF1-40	406 pcy	135 pcy	1836 pcy	1270 pcy	195 pcy	24 oz/yd3	1.7 oz/yd3
6	Control	406 pcy	135 pcy	1936 pcy	1167 pcy	186 pcy	24 oz/yd3	1.7 oz/yd3
7	SnF2-1.5	406 pcy	135 pcy	1836 pcy	1270 pcy	198 pcy	24 oz/yd3	1.7 oz/yd3
8	SnF3-0.5	406 pcy	135 pcy	1836 pcy	1270 pcy	197 pcy	24 oz/yd3	1.7 oz/yd3

*Indicates Fiber Type and Dosage (i.e., SF1-25 contains 25 pcy of steel collated, hooked-end fibers)

5.9 Testing Program

This section presents the results of the field specimen testing and focuses on properties that have not been presented in any other documentation. Fresh properties such as slump, air content, and unit weight were evaluated for each test section but were documented and discussed in the thesis written by Ryan Turner (2003). The following sections of this chapter will discuss the various hardened concrete properties evaluated in this field study and the impact fiber reinforcement has on typical CRCP mixture proportions.

5.9.1 Hardened Concrete Properties

The proceeding sections will cover each hardened concrete property individually. These properties include compressive strength, elastic modulus, splitting tensile strength, flexural strength, and flexural toughness. The purpose of this testing is to provide data for quality control of the concrete used and for the correlation between lab evaluations and actual field conditions. All specimens used to evaluate these properties were cast from the same batch of concrete to ensure accuracy. Also, the concrete was sampled from the middle of each test section to ensure that any modifications that needed to be made between test sections had already been completed. This is largely in reference to the mix plant obtaining their desired workability by modifying water content. All testing followed ASTM specifications.

5.9.2 Compressive Strength

For each test section, eight 6-in. x 12-in. cylinders were cast for compressive strength testing. Specimens were tested at 7 and 28 days. The results of the compressive strength testing are presented in Table 5.9. As has been explained and demonstrated in previous chapters, fiber reinforcement will have minimal impact of static concrete properties where cracking has not yet initiated. This phenomenon holds true for compressive strength and is verified by the results of the field specimens. While compressive strengths are not directly incorporated into TxDOT specifications for Class P concrete, the values found in this field study are typical for CRCP construction in Texas. The differences in strength between test sections were minimal, and any deviations can be attributed to differences in water content.

Table 5.9: Summary of Compressive Strength Test Results

Section	Mixture Designation	Average Compressive Strength (psi)	
		7-day	28-day
1	**SnF1-4**	**3,960**	**5,120**
2	**SF1-25**	**3,780**	**4,830**
3	**SF2-27.5**	**3,750**	**5,070**
4	**Control**	**3,730**	**5,160**
5	**SF1-40**	**4,090**	**5,480**
6	**Control**	**4,070**	**5,160**
7	**SnF2-1.5**	**3,620**	**4,830**
8	**SnF3-0.5**	**3,980**	**5,120**

5.9.3 Elastic Modulus

Of the eight 6-in. x 12-in. cylinders cast for compressive strength testing, four were first tested for elastic modulus. Elastic modulus tests were conducted at 7 and 28 days, and two compressive strength tests were always conducted first in order to determine an average compressive strength. From this value, the upper bound of the linear-elastic range (0.4 f ’c) was calculated. Stress and strain were then evaluated at 50 microstrain and 0.4 f’c to determine the elastic modulus. The results of the elastic modulus testing are presented in Table 5.10. As expected, fibers have minimal impact on elastic modulus, and no correlation between fiber type and/or dosage can be formulated. Any variation in elastic modulus can be attributed to variations in compressive strength and statistical scatter.

Table 5.10: Summary of Elastic Modulus Test Results

Section	Mixture Designation	Average Elastic Modulus x 10^3 (psi)	
		7-day	28-day
1	**SnF1-4**	**4,500**	**4,950**
2	**SF1-25**	**4,400**	**5,400**
3	**SF2-27.5**	**4,350**	**5,000**
4	**Control**	**4,750**	**5,100**
5	**SF1-40**	**4,300**	**5,350**
6	**Control**	**4,600**	**5,700**
7	**SnF2-1.5**	**4,200**	**4,600**
8	**SnF3-0.5**	**4,600**	**5,400**

5.9.4 Splitting Tensile Strength

For each test section, six 6-in. x 12-in. cylinders were cast for splitting tensile strength testing. Specimens were tested at 7 and 28 days. Results of the splitting tensile strength tests are provided in Table 5.11. It is difficult to obtain accurate splitting tensile results using fibers because it can be difficult to differentiate the actual initiation of cracking from the additional load carrying capacity contributed by the fibers after cracking. This is particularly true for steel reinforcement. Often during the testing of the specimens containing steel fibers, the cracking of the concrete matrix could be heard even though the load applied continued to increase in value. This behavior has more in common with toughness than an actual increase in strength, and therefore splitting tensile strengths should be viewed with caution. While care was taken to avoid this phenomenon during testing, steel fibers did appear to offer an increase in splitting tensile strength after variations in water-to-cementitious ratio had been noted. However, there is no real correlation between fiber reinforcement and splitting tensile strength. While this statement is valid for the field specimens, it is better supported by tests conducted in the laboratory study because of increased control of mixture proportions and water addition.

Table 5.11: Summary of Splitting Tensile Strength Test Results

Section	Mixture Designation	Average Splitting Tensile Strength (psi)	
		7-day	28-day
1	**SnF1-4**	**485**	**595**
2	**SF1-25**	**435**	**590**
3	**SF2-27.5**	**430**	**570**
4	**Control**	**425**	**505**
5	**SF1-40**	**520**	**565**
6	**Control**	**430**	**465**
7	**SnF2-1.5**	**440**	**445**
8	**SnF3-0.5**	**450**	**485**

5.9.5 Flexural Strength

Flexural strength was determined using three 6-in. x 6-in. x 20-in. prisms, and the specimens were tested at 28 days after casting. The specimens used to determine the flexural strength were actually tested for flexural toughness, but the peak load values determined using ASTM C 1018 (flexural toughness) are also valid for ASTM C 78 (flexural strength). Test results are presented in Table 5.12. As expected for static properties, flexural strength is not affected by fiber type or dosage. Differences in strength can be attributed to variances in water content and statistical scatter, which are typically high for a field study.

Table 5.12: Summary of Flexural Strength Test Results

Mixture	Mixture Designation	Average Peak Load (lb)	Average Flexural Strength (psi)
1	**SnF1-4**	**8,380**	**695**
2	**SF1-25**	**8,844**	**715**
3	**SF2-27.5**	**9,242**	**750**
4	**Control**	**9,317**	**745**
5	**SF1-40**	**8,588**	**680**
6	**Control**	**9,229**	**730**
7	**SnF2-1.5**	**8,381**	**655**
8	**SnF3-0.5**	**8,556**	**670**

5.9.6 Flexural Toughness

For each test section, flexural toughness testing was conducted on three 6-in. x 6-in. x 20-in. prisms 28 days after casting. As per ASTM C 1018, all specimens were tested using a closed-loop deflection controlled system in which midspan deflections were monitored using a Japanese yoke mounted through the neutral axis of the prism. More information on this type of control system can be found in Section 3.4.2. It should be noted that the prisms were not tested at UT Austin, because the test frame and data acquisition system were still under construction at the time of the field test. The specimens were tested by W.R. Grace in Cambridge, Ma, using test equipment similar to that developed at UT Austin. This situation also provided the opportunity to ensure that adequate inter-laboratory results could be obtained.

As previously discussed in the laboratory study, flexural toughness is the key parameter for quantifying the ability of fiber reinforcement to improve the post-cracking behavior of concrete. It is especially important in relation to CRCP because the performance of these pavements is directly related to controlling crack widths and maintaining adequate load transfer across these cracks. The post-cracking behavior of the various types of fiber-reinforced concrete tested is presented qualitatively in Figure 5.53. It can be seen from this figure that fiber reinforcement can provide significant improvements in post-cracking load carrying behavior when compared to plain concrete. However, the degree of improvements that can be made is heavily dependent on the fiber type and dosage. The level of improvement fibers are capable of producing is quantified by toughness indices and residual strength factors. These values are presented in Table 5.13, and graphical comparisons of these calculated parameters are provided in Figure 5.54 and Figure 5.55.

The toughness and residual strength values determined from the field specimens are very similar to the values determined in the laboratory evaluation. This verifies that the desired fiber performance can be achieved under the lower level quality control that is typical in field applications. As had been documented previously, steel fibers typically provide greater improvements in toughness than synthetic fibers. The differences in toughness behavior between steel and synthetic fibers are much more significant as deflections are increased. While these statements are generally true, specific issues need to be addressed. SF2 provided good levels of toughness at smaller deflections; once significantly larger deflections were reached, the load carrying capacity decreased drastically. This is expressed quantitatively by the decreased values for toughness indices and residual strength factors. This phenomenon is attributed to insufficient anchorage between the fibers and concrete matrix, leading to excessive pullout of the fibers as deflection demand increases. It also shows the impact fiber geometry can have on toughness performance. Synthetic macrofibers, like SnF1, provide much greater improvements in post-cracking behavior than synthetic microfibers such as SnF2 and SnF3, primarily because of optimized fiber properties and higher volume loadings. The synthetic macrofibers performed more like steel fibers than synthetic microfibers.

Toughness and residual strength do quantify which fiber types and dosages offer the greatest relative improvement in CRCP performance, but the extent of this improvement can only be determined by monitoring the actual long-term improvements fiber reinforcement produces. Only after quantifying the actual field performance can true correlations between toughness, residual strength, and CRCP performance be created.

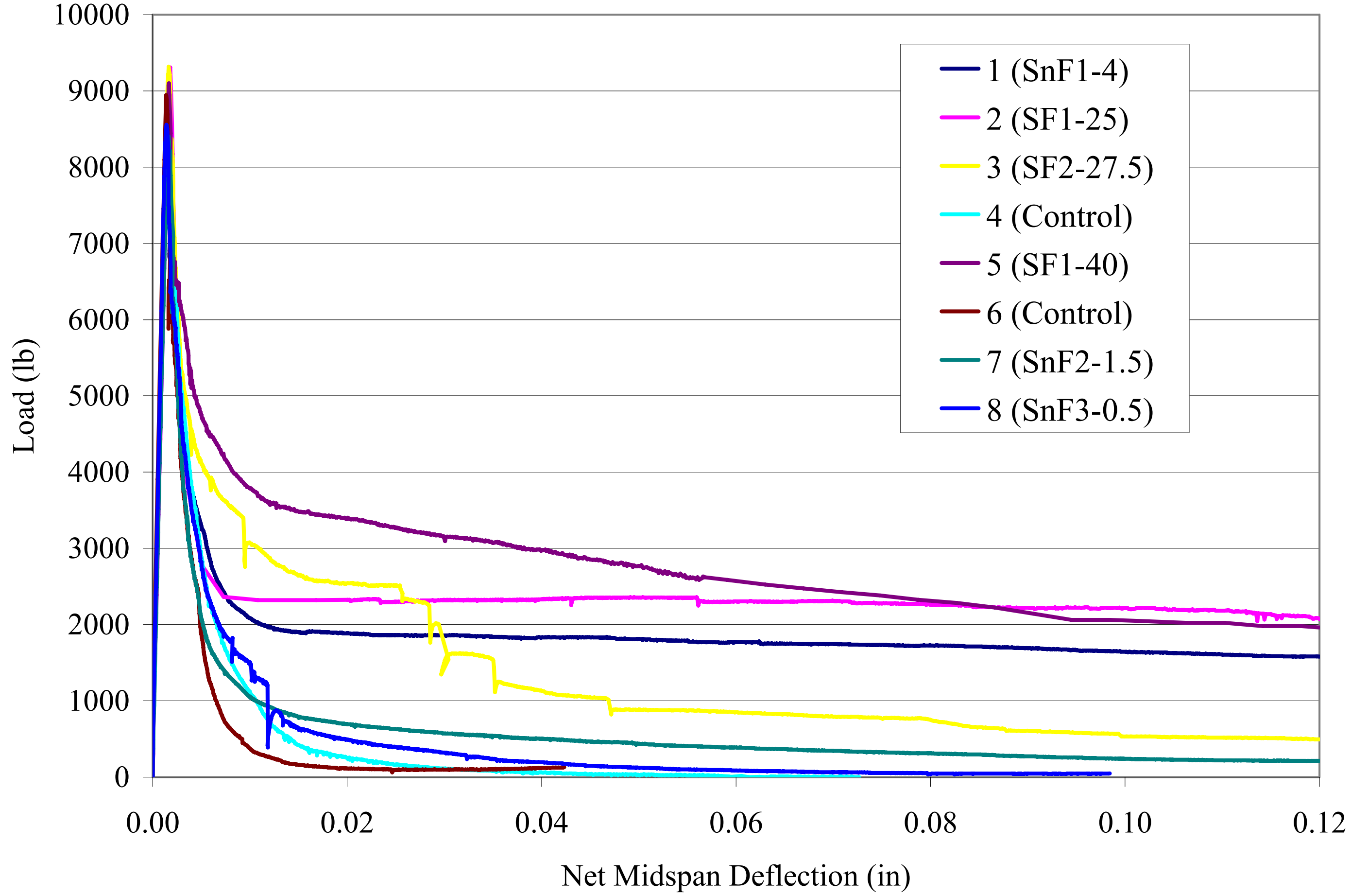


Figure 5.53: Summary of Load-Deflection Data

Table 5.13: Summary of Flexural Toughness Test Results

Section	Mixture Designation	Toughness Indices						Residual Strength Factors				
		I_5	I_{10}	I_{20}	I_{30}	I_{50}	I_{60}	$R_{5,10}$	$R_{10,20}$	$R_{20,30}$	$R_{30,60}$	$R_{10,50}$
1	**SnF1-4**	**2.9**	**4.2**	**6.0**	**7.6**	**10.7**	**12.2**	**25.2**	**18.0**	**16.0**	**15.2**	**16.2**
2	**SF1-25**	**3.0**	**4.2**	**6.8**	**9.5**	**14.5**	**16.9**	**23.3**	**26.1**	**26.6**	**24.9**	**25.7**
3	**SF2-27.5**	**2.7**	**3.7**	**5.2**	**6.5**	**8.4**	**9.1**	**19.0**	**15.1**	**12.8**	**8.9**	**11.9**
4	**Control**	**3.4**	**4.5**	**5.4**	**5.7**	**5.9**	**6.0**	**23.8**	**8.5**	**2.9**	**1.0**	**3.5**
5	**SF1-40**	**5.4**	**8.8**	**14.6**	**20.1**	**30.5**	**35.5**	**67.4**	**58.0**	**55.4**	**51.4**	**54.4**
6	**Control**	**3.4**	**4.4**	**5.1**	**5.4**	**5.7**	**5.8**	**21.3**	**7.0**	**2.7**	**1.4**	**3.2**
7	**SnF2-1.5**	**3.0**	**3.8**	**4.7**	**5.4**	**6.4**	**6.9**	**15.5**	**8.9**	**6.6**	**5.1**	**6.6**
8	**SnF3-0.5**	**3.3**	**4.5**	**5.8**	**6.3**	**7.0**	**7.2**	**24.4**	**12.6**	**5.5**	**2.9**	**6.2**

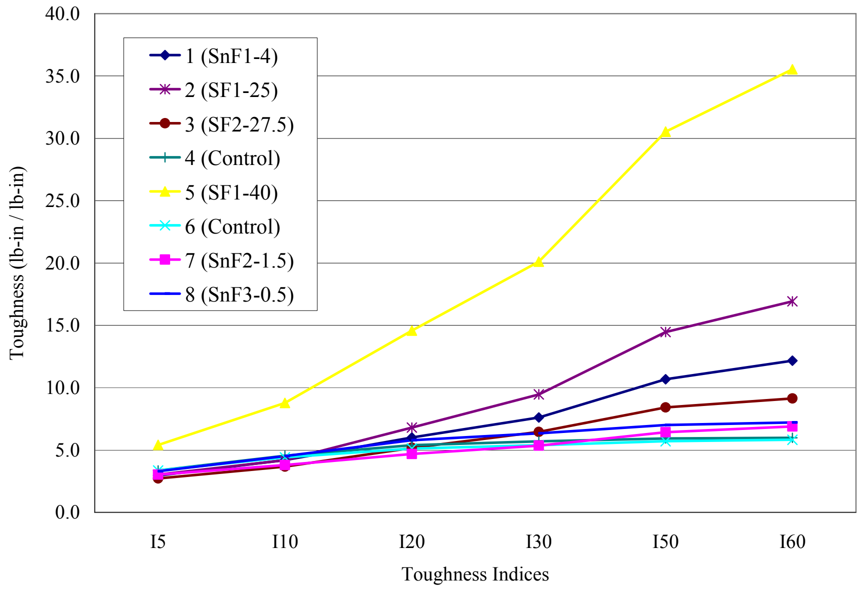


Figure 5.54: Effects of Fibers on Toughness

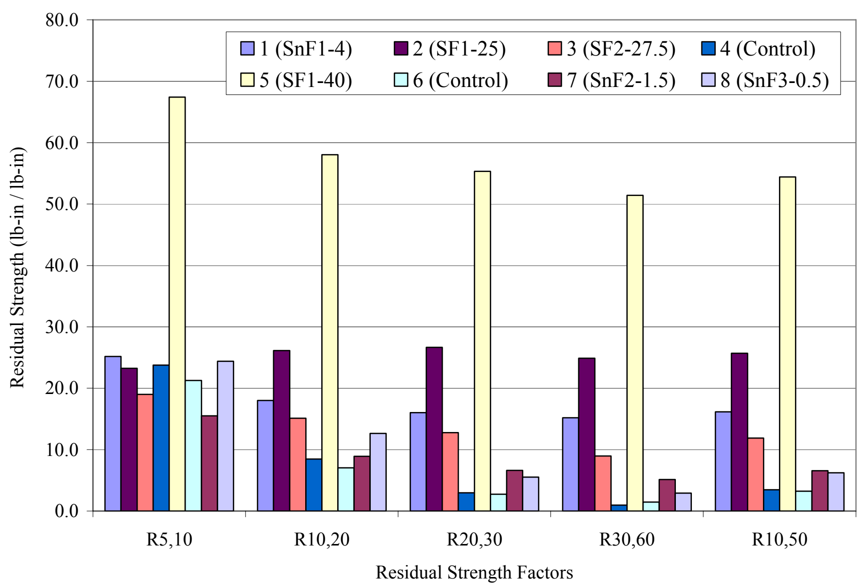


Figure 5.55: Effects of Fibers on Residual Strength

5.10 Monitoring of Test Sections

While properties such as flexural toughness are good indicators of the potential improvements fibers can impart on CRCP performance, correlations between toughness and spalling resistance can only be formulated with long-term monitoring of actual CRCP field performance. This section will cover the three performance parameters relevant to this study: crack spacing, crack width, and spalling.

While the information to be presented will provide some indication of potential improvements fibers can have on CRCP performance, the data is somewhat limited because of time constraints of the research. Another limitation was the inability to obtain the proper traffic control to monitor these test sections. Since the test sections are located in the middle of a heavily traveled highway, shutting down numerous lanes was too impractical. Because of these issues the pavement was monitored one time, 1month after placement. This is not a major concern because any significant performance distresses, such as spalling, would not become apparent during the time remaining for this study. This statement has been verified by monitoring conducted on the first test section, and was discussed in Ryan Turner's thesis (2003). Discussions of the long-term monitoring necessary to quantify the effects of fibers on long-term CRCP performance and spalling resistance are available in Section 6.4, *Recommendations for Future Research.*

5.10.1 Crack Spacing

Since CRCP is constructed without any joints, it is expected to crack at regular intervals due to the restraint of the reinforcing steel and sub-base. In the past, crack spacing was a major issue for CRCP performance because if the transverse cracks were too close, longitudinal cracks formed. These extras cracks increased stress concentrations and eventually lead to punchouts. This issue has been alleviated by tying the pavement into the shoulders. Even though crack spacing is not currently a major issue, it is still relevant because it can impact crack widths. Pavements that have larger crack spacing tend to have larger crack widths because the concrete will experience uniform shrinkage regardless of crack location. These larger cracks widths can lead to poor CRCP performance. For TxDOT CRCP design, cracks are expected to occur every 3 to 8 ft. Fiber reinforcement was expected to have little or no impact on crack spacing because the addition of fibers will only impact post-cracking behavior and not the initial crack formation.

The following figures present the crack spacing data acquired 1 month after the test sections were placed. Figure 5.56 provides the average crack spacing and standard deviation for each test section. Figure 5.57 provides the percentage of cracks falling within a specified spacing for each test section. The data collected was somewhat inconclusive because after 1 month there had not been a sufficient amount of time for all of the expected cracks to form into macrocracks. It typically takes approximately 3 months for the majority of cracks to form, and this is supported by the monitoring of the first test section constructed. This is why the average crack spacing shown is larger than what is typically expected. Section 1 and Section 2 had extremely high crack spacing, but this can be attributed to a lack of restraint at the end and the side of the test sections. This same issue was experienced in the end sections of the first field study. Despite these inconsistencies, the remainder of the data does imply that crack spacing is not significantly affected by fiber type and/or dosage.

Both Figure 5.56 and Figure 5.57 do show that there is significant scatter in the initial crack spacing data. More long-term evaluations are required to properly quantify crack spacing and the impact fiber reinforcement may have on it.

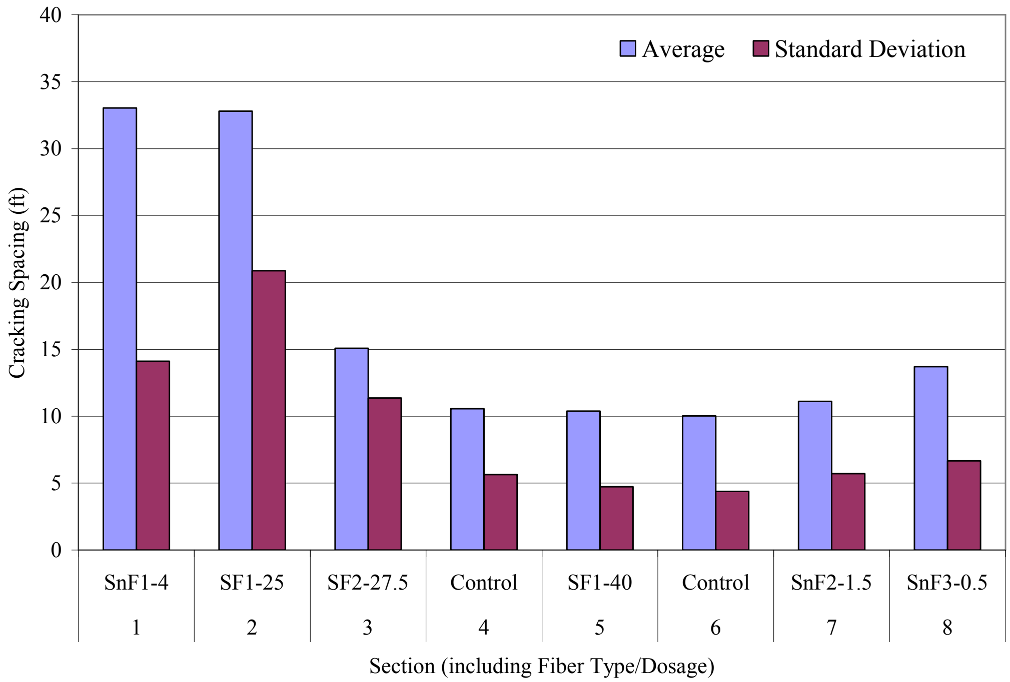


Figure 5.56: Average Crack Spacing for Each Test Section after One Month

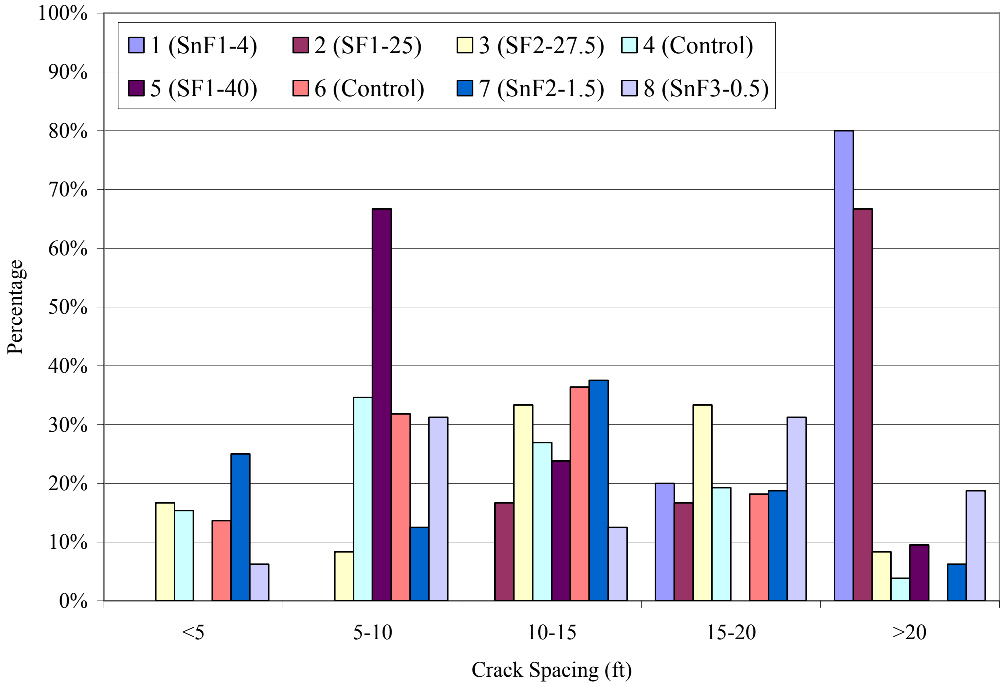


Figure 5.57: Percent of Cracks at Each Spacing after One Month

5.10.2 Crack Width

The potential improvements fiber reinforcement can have on controlling crack widths could lead to large increases in CRCP service life. This is because controlling crack widths is critical to ensuring good load transfer and the desired CRCP performance. Maintaining small crack widths is vital for CRCP for a variety of reasons. Cracks that are too wide allow water penetration, which can induce corrosion of the longitudinal steel and thus degradation in load transferring capabilities. Excessive crack widths also allow incompressible materials such as sand to infiltrate the openings. This type of material will cause stress concentrations in the crack when load is applied to the pavement and leads to premature pavement failures. Typical variables known to impact crack widths include pavement depth, percentage of longitudinal steel, and aggregate type. The impact of aggregate type is the most relevant for this study. In particular, the high coefficient of thermal expansion (CTE) of siliceous river gravel is believed to be the main cause for premature spalling of CRCP. Large temperature gradients, common in CRCP, lead to excessive expansions and contractions, larger cracks, and eventual failure. If fibers can reduce the stresses caused by this behavior, CRCP performance can be greatly enhanced.

Crack widths were determined for each test section by measuring the length of a given crack in three places using a crack comparator, then determining the average. Each of these average crack widths were then averaged over the entire length of a given test section. The average crack widths and standard deviations for each section are presented in Figure 5.58. The data is inconclusive for evaluating the effects of fiber reinforcement on crack widths because the pavement was monitored at such an early age (1 month), and the complete cracking of the sections had yet to occur. It takes much longer to observe more significant crack widths which would lead to performance issues. Once the pavement experiences more significant loading and seasonal changes, the effect of fiber reinforcement on cracks widths will become very apparent. To capture this data a long-term monitoring program needs to be prepared. More information on this plan will be outlined in Section 6.4, *Recommendations for Future Research.*

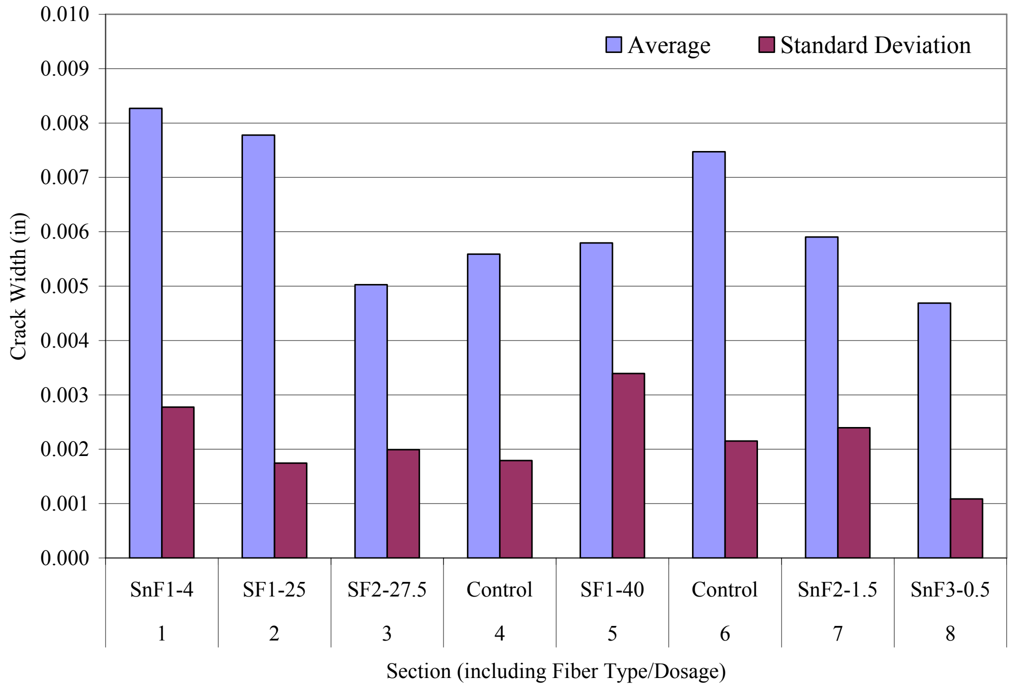


Figure 5.58: Average Crack Width for Each Test Section after One Month

5.10.3 Spalling

The actual mechanism that causes excessive spalling of CRCP is not completely understood, but controlling this failure type is currently a very important topic. It is known that siliceous river gravel is especially susceptible to this type of failure. Even though the mechanism is not well understood, fiber reinforcement has the potential to provide improvements in CRCP spalling resistance by controlling crack width and providing additional load transfer across cracks. Because the pavement was still so young and had not yet been subjected to traffic loading, no spalling was observed in any of the test sections. More rigorous long-term inspections will be necessary to comprehensively evaluate the potential of fiber reinforcement to improve CRCP spalling resistance.

5.11 Summary

The initial findings of this mainline fiber-reinforced CRCP field study have been very promising. CRCP containing low dosages of fiber reinforcement can be placed successfully using typical machinery, but small adjustments to water and/or water-reducing admixture dosage may be required to achieve the desired workability. The use of additional fine aggregate as a direct replacement for coarse aggregate was also found to improve workability and ensure proper consolidation. Excluding workability and finishability, low dosage fiber reinforcement had little or no impact on other fresh properties such as air content and unit weight. These findings also verify the fresh property testing conducted in the laboratory study, which are discussed in Chapter 3.

Fiber reinforcement had minimal impact on most typical hardened concrete properties including compressive strength, splitting tensile, elastic modulus, and flexural strength. This is as expected from results found in the laboratory study. Flexural toughness is the only hardened concrete property greatly affected by the addition of fiber reinforcement. The amount of toughness imparted onto a typical control mixture was highly dependent on the fiber type and/or dosage. Steel fibers provided greater levels of toughness than did synthetics and macrofibers. SnF1 had better performance than microfibers (SnF2, SnF3). Since toughness should correlate well with improvements in CRCP spalling resistance, the long-term monitoring of the field tests will be vital to properly quantify these improvements.

Initial monitoring of the test sections does provide insight into the effects of fibers on cracking spacing and crack width. Unfortunately, the data obtained is inconclusive because of the lack of time allotted for the remainder of the research project. It typically takes a few months for the initial cracking to complete, and therefore the long-term crack spacing cannot be properly evaluated until that time. Because the pavements were very young at the time of monitoring, they had experienced relatively few loading and temperature cycles and consequently had very small crack widths and no spalling. Because of this lack of pavement stressing, the impact of fiber reinforcement addition was minimal but is expected to increase significantly as time passes.

In order to properly complete this field evaluation, additional long term monitoring is necessary because of the relatively long service life of CRCP compared to the time allowed for this research project. Only after this more rigorous evaluation occurs can sound correlations between flexural toughness and CRCP performance be quantified. The construction of more test sections with a greater number of fiber types and dosages would provide more data in order to accurately evaluate fiber reinforcement effects. These issues will be discussed in greater detail in Section 6, *Guidelines for Using Fibers in CRCP*.

Chapter 6. Guidelines for Using Fibers in CRCP

6.1 Summary

One of the major objectives of this research originally was to determine whether fiber reinforcement could be a practical tool for mitigating spalling in CRCP constructed with SRG. At the beginning of this research study, TxDOT engineers were uncertain how fiber reinforcement would affect standard practices for placing and finishing CRCP. While they were confident that it was possible to construct CRCP containing fiber reinforcement, researchers were asked to determine any modifications necessary to ensure good workability and finishability. And of course, the major question that this research addressed was whether adding fibers to concrete can prevent spalling in CRCP. Given that this was only a two-year project, it is not possible to conclude about the long-term effects of fibers on spalling. In fact this project was originally approved as a three-year project but was inexplicably reduced to a two-year project after a substantial portion of the work was in progress. The extra year would certainly have been helpful in monitoring the performance of the test sections and in helping to develop guidelines and recommendations for using fibers to control spalling in CRCP. As such, a follow-up to this project or implementation study is essential to capture the field performance of the various test sections, and with this information in hand, more firm guidelines and economic evaluations can be developed.

Two extensive field investigations were performed that provide vast amounts of data that can be used to evaluate pavement performance. The first field test was conducted on a highway frontage road in August under hot temperatures to evaluate three types of fibers that varied in dosage. The second field test was conducted in cooler weather on a highway main lane to evaluate the first three fiber types. The two field tests provide different weather conditions for evaluating the effects of fiber reinforcement. The main performance parameters that were monitored in each study include constructability, crack spacing, crack width, and spalling development.

Each fiber type and dosage implemented in the field studies was also tested in the laboratory under a controlled environment to have a better understanding of the fiber reinforcement effects on hardened concrete properties. Research included standard tests such as compression, flexure, splitting tensile strength, and flexural toughness. In addition, early-age tensile strength was measured.

Observations documented in two field studies verified that it is indeed possible to place and finish CRCP mixtures containing fibers. Care must be taken, however, to ensure proper construction. Meaningful additions of fibers impact water demand and workability. This study showed that changes to water content and/or water-reducing admixture dosage will be required for adequate workability. The severity of these changes will be dependent on fiber type and fiber dosage rate. Since fiber dosages used in this study were relatively low, the required modifications were typically minimal. If higher fiber dosages are desired, the effects of the fiber on placement and finishing will have greater impact and require more consideration.

The finishability of CRCP can be significantly changed by the introduction of fiber reinforcement. While many fiber types and dosages finished well, SnF1 had problems with finishability because of the high fiber count and low specific gravity. SnF1-6 had so many problems being placed and finished that it was not used after the first field test. To ensure good consolidation and finishability, a portion of the coarse aggregate should be replaced with fine aggregate, and the pavement should be constructed using machinery and techniques used for slipform paving.

6.2 Performance

It has been shown from both laboratory and field specimen testing that fiber reinforcement has the potential to impart improvements on CRCP performance. While most hardened properties are not changed, the increases in toughness that fibers provide will undoubtedly decrease crack widths, improve load transfer across cracks, and lead to improved long-term spalling resistance. Toughness was found to be highly dependent on fiber type and/or dosage. Steel fibers were also found to typically provide greater improvements in toughness for the same volume replacement. However, determination of the degree to which fibers will improve long-term pavement performance is still preliminary because of the lack of allotted time for this research project. Spalling, even in poor-performing pavements, will take time to surface, and the field sections will need to experience more traffic loads and temperature cycles before real conclusions can be formulated. More significant time must pass before better correlations between toughness and CRCP performance are possible.

6.3 Economic Feasibility

While fiber reinforcement can make significant improvements in CRCP performance, the additional cost associated with using fibers will govern their feasibility for use in CRCP construction. The current cost of fibers ($0.30/lb–$0.50/lb for steel, $2.00/lb–$3.00/lb for synthetic) is a large additional cost to incorporate into pavements, which are governed by material costs. The economic feasibility of fiber reinforcement depends on a few issues. Cost-benefit analysis will need to be performed to determine which fiber most efficiently controls spalling. This can only be completed once it has been determined how fibers affect long-term performance, because it has yet to be determined what level of toughness is needed to mitigate spalling. Cost-benefit analysis can also be used to determine if fiber-reinforced CRCP has a lower cost over the length of its service life compared to CRCP, which needs repairs as a result of spalling. This type of analysis is synonymous with performance-based design, and this method of design would make the addition of fiber reinforcement more feasible. Finally, the use of fibers will become more and more attractive if the cost of fibers decreases, which will most likely occur over time as their use increases.

6.4 Recommendations for Future Research

While much knowledge has been gained throughout the duration of this research, continued efforts in the future will be crucial to obtaining the information needed to viably incorporate fiber reinforcement into CRCP design. Since this research project lacked the time required to properly monitor the field sections for long-term results, it is imperative that the test sections constructed for this project be monitored over the coming years to properly assess the long-term impact of fibers on CRCP performance. Creating additional test sections using a larger range of fiber types and dosages would also provide additional data to improve correlations between toughness and CRCP performance.

While the research conducted for this report focused on using fibers as a secondary reinforcement to control spalling, there is a potential to use fiber reinforcement to reduce longitudinal steel percentage and/or slab thickness. Using fiber reinforcement to control spalling without any design modifications will most likely lead to over-designed pavements that are rather costly. If research were conducted to optimize fiber dosage, steel percentage, and slab thickness, spalling could be controlled at a much lower cost.

Finally, a better understanding of the spalling mechanism and of why siliceous river gravel is particularly vulnerable needs to be addressed. Fiber reinforcement does provide a method of controlling this failure, but there may be simpler and more cost effective ways of stopping spalling if its mechanism were better defined. This is a very difficult task and has been investigated for quite some time. However, only until the failure is thoroughly understood can the best solution to stop spalling be formulated.

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