Behavior of Concrete with Composite Reinforcement

by

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Abstract

Steel reinforcement for concrete structures has been commonly used in construction for years. However, in some environments steel may not be an ideal choice for reinforcement because it may deteriorate due to chloride ion penetration and it may interfere with the magnetic fields of sensitive equipment. To avoid using steel in those environments, Fiber Reinforced Plastics (FRPs) may be used in place of steel as concrete reinforcement. FRPs are typically stronger and lighter than steel, but they are not as stiff as steel rebar and do not bond as well to concrete. While the flexural behavior of FRP-reinforced beams has been tested, the shear behavior of FRP beams has not been studied as much. Composite materials may also have applications in retrofitting of structural columns and strengthening columns against earthquake loading. Rather than using steel confinement, composites wraps may be used to improve the strength and ductility of concrete columns.

In this research program, concrete beams were cast with FRP reinforcing bars. Plain and fiber-reinforced normal strength and high strength concretes were tested. No shear stirrups were used, and the beams were designed to fail in shear, not in bending, to test the effectiveness of FRP reinforcing bars with the different types of concrete. In addition, composite wraps were tested for their effectiveness in increasing the strength and ductility of concrete cylinders under compression.

The FRP reinforcing bars performed well as flexural reinforcement, though they did not provide as much shear resistance as steel bars. The addition of steel fibers increased the shear capacity and post-peak ductility of the beams. High strength specimens showed more of an improvement with the inclusion of fibers than normal strength beams did. Fibers also improved the strength of FRP-reinforced beams more than the ones reinforced with steel rebar. Polypropylene fibers increased the shear capacity at the expense of post-peak ductility. The composite column wraps provided huge increases in concrete column ultimate stress and strain for all the tested specimens.

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Introduction

1.1 Background

Concrete has been widely used as a construction material for many years because of its high compressive strength and low cost. Though ten times stronger in compression than in tension, concrete may be used in beams and slabs with the addition of reinforcing bars. Typically made of steel, these bars take the tensile load while the concrete resists the compressive load on the beam. However, reinforced concrete beams will inevitably crack in flexure. As the crack opens, salt and chloride ions can penetrate into the cracks. This is especially a problem in coastal environments, where salty air from the ocean carries chloride ions, and northern environments, where deicing salts may penetrate cracks in concrete. To combat this problem, fiber-reinforced plastic (FRP) reinforcing bars may be used to carry the tensile load acting on the concrete. Though not as stiff as steel, FRP bars can resist tensile load, but they will not deteriorate in a salty environment. Thus, the study of the feasibility of composite reinforcing bars in concrete may yield valuable benefits to construction in regions where structural members are subject to chloride penetration.

In addition to the tensile and compressive forces that a beam in flexure must withstand, concrete and its reinforcement must be able to resist shear cracking within the concrete. The diagonal tension in the shear span of a beam creates a diagonal crack between support and load point. Shear is resisted by the concrete, the reinforcement, and any additional reinforcement that is added, such as shear stirrups or fibers. Reinforcing fibers made of steel, kevlar, polypropylene,
glass, and carbon have been used to toughen concrete. These fibers may be short or long, and may be straight or hooked. They make crack propagation much more difficult, increasing the toughness of the concrete. However, they make the concrete mix much less workable, possibly causing the compressive strength of the concrete to drop. Fibers may also increase the ductility of the concrete. Because of the improved shear strength fibers provide, though, fibers may be used instead of additional stirrup reinforcement to improve the shear behavior of concrete.[4]

Concrete is also used in structural columns to take advantage of its excellent behavior in compression. However, a concrete column may be strengthened by a confining wrap. Especially popular in retrofitting of structures, wrapping usually involves fabricating a steel jacket and installing it around the column. However, if steel is used for this type of reinforcement, the column will be vulnerable to deterioration due to corrosion. Composite wraps may be used for these situations. Since composites do not yield, they behave differently from steel wraps. Composite reinforcement will keep providing additional resistance until it fractures. Though this will improve the performance of the concrete, it also leads to spectacular, catastrophic failure. Retrofitting has become increasingly popular in areas subject to seismic damage, as concrete columns have failed or suffered severe damage in recent earthquakes. California’s Dept. of Transportation has begun to use composite wraps in practice already.[13,17] However, few studies have been done on the behavior of composite-confined concrete.
1.2 Research Rationale

Though FRP reinforcing bars are being used now in construction practice, no comprehensive study has been done to study the shear behavior of fiber-reinforced FRP-reinforced concrete beams. Because the bond between the FRP and the concrete is not as strong as the steel-concrete bond, shear cracks may be able to propagate more easily along the reinforcement-concrete interface. To improve the shear behavior of these beams, fibers may be used. The first part of this research seeks to examine the shear behavior of normal and high strength concrete beams using FRP tensile reinforcement, as compared to steel reinforcement. Steel fibers and polypropylene fibers were used in some of the concrete mixes to determine the effect of fibers on shear behavior.

Similarly, little research has been done on the effect of composite wrap for concrete cylinders, as opposed to steel confinement. No studies involving confined high strength concrete or fiber-reinforced concrete have been carried out. Confined cylinders fail at much higher loads and strains than unwrapped ones. Steel has been the primary reinforcing material for jacketing, but composites may prove very useful as a confining material. Because composite does not yield, the concrete cylinder is expected to exhibit more ductility and higher strength. Tests to determine the behavior of normal strength and high strength plain and fiber reinforced concrete were conducted to test the effectiveness of the composite wrap in confining concrete.
1.3 Organization of the Thesis

This thesis has been divided into 6 chapters, as described below.

Chapter 1 presents a very general overview of the behavior of concrete and describes the objectives of the research program.

Chapter 2 describes the shear behavior of concrete beams and presents past research conducted on FRP-reinforced concrete and fiber-reinforced concrete, and highlights the differences between high strength concrete and normal strength concrete.

Chapter 3 describes the experimental work done in this research.

Chapter 4 presents the data on the shear behavior of FRP-reinforced beams and provides an analysis of the collected data.

Chapter 5 presents the data on the confined cylinder tests and provides an analysis of that data.

Chapter 6 presents brief conclusions with recommendations for further research.
Chapter 2

Research Background

2.1 Background

Shear behavior in concrete is dependent upon a number of factors. Shear cracks are diagonal cracks that propagate in the shear span. Some of the material factors that affect shear failure are the strength, size, and bond of the tensile reinforcement, the type of concrete, type of shear reinforcement, spacing of shear reinforcement, and prestressing. Geometrically, the shape of the concrete cross-section, depth to shear span ratio of the specimen, depth and width of the section, and type of loading (three point bending, four point bending, etc.) all affect the shear behavior.[1] In this research, the effect of main flexural reinforcement and fiber reinforced concrete were the subjects of interest, so the other parameters were held constant in all tests.

2.2 Shear Transfer in Concrete

If concrete is not adequately reinforced for shear, a diagonal crack will propagate quickly, leading to catastrophic failure. This can occur suddenly and without warning, especially in shallow beams - those with span-depth ratios of 6 or greater. The so-called shear crack really does not result from the shear stresses placed on the beam. However, there is a diagonal tension
stress that results from the combination of the shear and longitudinal stresses. When that combined tensile stress exceeds the tensile strength of the concrete, shear cracks occur. Diagonal tension occurs in areas of high shear force, initiating cracks at 45° to the axis of the specimen when no axial force is present.[2]

Concrete resists the shear force through several mechanisms, as illustrated in Figure 2-1. The figure shows one half of a beam with a diagonal crack. The total shear transferred across the crack is $V$, which is made up of three components:

1. The shear force resisted by the uncracked concrete ($V_a$) undergoing compression above the neutral axis. This accounts for 20 - 40% of the total shear resistance.

2. The shear force resisted by the flexural reinforcement ($V_d$) through dowel action. This accounts for 15-25% of the total shear resistance.

3. The inclined shear stresses transmitted by aggregate interlocking ($V_a$) along the inclined crack. This accounts for 35-50% of the total shear resistance.[1]

The force resisted by dowel action is quite small. Since the reinforcing bars are supported against the vertical load by the thin concrete layer below, the bearing pressure creates vertical tension stresses in the concrete. This often results in splitting of the concrete along the flexural reinforcement, further reducing the dowel force. This reduction in force allows the crack to widen, decreasing the amount of resistance provided by the uncracked concrete. The dowel force is usually the first to reach its full capacity. Then, the aggregate interlocking reaches its capacity. Finally, the concrete in the compressive area fails, causing the entire beam to fail.[2]
Beam failure may be classified into five types, based on the shear arm to effective depth (a/d) ratio.

(1) a/d > 6

Beams will usually fail in bending.

(2) 3.5 < a/d < 6

Beams will fail shortly after the diagonal cracking load has been reached, as the diagonal crack spreads rapidly, resulting in collapse by splitting the beam into two pieces.

(3) 2.5 < a/d < 3.5
The diagonal crack stops due to concrete arch action, many cracks form and eventually the diagonal crack widens and propagates along the level of the tension reinforcement. The steel then debonds from the concrete, causing the beam to collapse.

(4) $1 < \frac{a}{d} < 2.5$

The diagonal crack forms independently of a flexural crack, until the load is increased enough to cause the crack to penetrate the compressive zone at the loading point. This continues until the concrete fails in compression at the load point.

(5) $\frac{a}{d} < 1$

A beam with this ratio would be classified as a deep beam. A diagonal crack forms because of the compression force between the load and support. Crushing of concrete occurs at either the load point or, more typically, the support point.[2]

For beams undergoing four point bending tests, shear failure will usually be the governing parameter for specimens with $1.5 < \frac{a}{d} < 6$.

2.3 Advantages of Fiber Reinforced Concrete

Fiber reinforced concrete may improve concrete behavior in a number of ways. Fibers bridge cracks that are forming in the concrete and resist their propagation. They can also increase ductility in both concrete columns and beams. If enough fibers are used, steel stirrup requirements may be lowered or even dropped altogether. Fibers may be preferable to steel
stirrups in seismic zones, because they resist stresses in all directions equally. Also, the use of fibers may decrease the total labor involved in construction as they do not need to be bent or fixed in place. Fibers can also be placed in irregular sections where it may be difficult to use stirrups.[3] With FRP reinforcing bars, because the bond between the FRP and concrete may not be as strong as the concrete-steel bond, cracks may tend to open more widely. The addition of fibers may serve the important need of arresting those cracks, thereby improving the total concrete performance.

One drawback of the inclusion of fibers into a mix is that fibers decrease the workability of the concrete. Fibers tend to clump around mixing blades and also will ball up and not mix well into the concrete unless care is taken in the mixing process. Longer fibers ball up more than short ones. Mixing techniques to accommodate the inclusion of fibers, such as using superplasticizer or using mixers without blades have been used to allow for the workability problems associated with fibers.

Much research has been done on the effect of fiber reinforcement on the shear strength of concrete beams. Increases in strength of 10-170 percent have been recorded.[4] However, the effect of fibers on ductility of concrete beams is still unclear. Fibers should increase the amount of displacement before failure, providing ductility on the load-deflection curve. They also may alter the failure of the concrete so that when the beam fails it shows post-peak ductility; the load does not drop catastrophically, and the beam is able to carry load after failure. Fibers have been shown to increase concrete resistance to crack formation and propagation, and spalling. [5]
In beams with a/d less than 2.5, arch action is evident when the diagonal crack extends to the supports, separating the tension and compression zones of the shear span. Fibers improve the splitting strength of the beams, and may lead to crushing at the support or sudden ejection of the upper part of the shear span due to failure of the compression zone under combined shear and compression, together with sliding along the diagonal crack faces.[3]

In beams with a/d greater than 2.5, diagonal cracks propagate along the compressive stress path toward the load point and the support. Flexural-shear cracking occurs, followed by diagonal shear cracking forming as an extension of a flexural crack. Some cracks propagate along the reinforcement. Failure occurs either by excessive cracking along the rebar that reduces the dowel force or by failure in the compression zone under combined shear and compressive stresses. Because fibers add resistance to crack propagation and allow greater tensile stress capability across existing cracks, they increase the load to failure. In addition, the fibers increase the dowel force by preventing crack propagation along the rebar. Failure will occur at a higher load because of the increased shear capacity across the cracks and along the dowel.[3]

As the a/d ratio increases, the effectiveness of fiber reinforcement also increases. This is because fibers cannot improve arch action in a beam as much as they can improve beam action. Arch action depends on compressive forces developing between load point and support. The forces depend on fiber pullout behavior across diagonal cracks, compressive strength of the material, and material behavior under combined shear and compression near the load point. Fibers only increase the behavior across the cracks. For a/d greater than 2.5, though, failure depends on
tensile capacity across cracks and dowel forces, which may both be improved by the addition of fibers.[3]

When used in NSC, fibers have been shown to improve the shear strength of weaker mixes more than it does stronger ones, because the strength contribution of fibers is based on the matrix-fiber bond and not concrete strength. When concrete quality is improved, the mix strength increases more than the matrix-fiber bond strength. Also, composite tensile strength improvement depends on the ratio between fiber-bridging toughness and matrix toughness. A higher quality mix reduces the ratio and lessens the improvement in tensile strength. For higher strength concrete, better bonding or higher fiber fractions will thus be necessary to achieve the same improvements in shear strength.[5] Beam size also has an effect on the shear strength of fiber-reinforced beams. Because increasing beam size decreases the shear strength of beams, larger beams will show more of an improvement with the incorporation of reinforcing fibers. Therefore, the fibers will have more of an effect on a weaker, larger beam.[3]

When a fiber reinforced beam is placed under load, the fibers may fail by either breaking or pulling out (debonding) from the concrete matrix. The tensile strength of the concrete is improved whichever failure mode is prevalent. However, if the fibers fail by debonding, the toughness of the specimen is also increased. Because high strength concrete bonds better to embedded fibers than normal strength concrete, the addition of fibers improves shear performance of high strength concrete more than the performance of a normal strength mix. This is also why fibers with rough surfaces or with hooked ends are often used. However, if the bond is too great, the fiber may fail by fracture. The shear strength of the beam will still increase
significantly, but at the expense of very low post-peak strength. However, if high strength steel is used, failure can still occur by pullout, rather than fracture.[2]

Fracture occurs differently in fiber-reinforced concrete than it does in normal concrete. Because of the fibers, the width and length of the fracture process zone are larger in fiber-reinforced concrete, due to crack bridging, fiber debonding and pullout, crack deflection, and multiple cracking.[3] These mechanisms increase the energy dissipation and ductility of the material. Fiber-reinforced concrete has been shown to lower the rate of crack growth in concrete, as well as provide a higher ultimate bending strength. Crack widths are also reduced in fiber-reinforced specimens. Because cracks are not as wide, strains in beams have also been shown to be reduced.[6]

While the inclusion of fibers improves the tensile and shear strength, it does not significantly affect the compressive strength of concrete. The major change fibers have on concrete in compression is the change in failure mode from a double cone to a shear band. Distributed fibers turn the failure mode from diagonal shear cracking to bending-shear failure with multiple flexural and shear cracks. Fibers also increase the effectiveness of the rebars by preventing splitting cracks and debonding along the rebar.[5]

When large amounts of fibers are included, over 4% by volume, alternate casting techniques must be used, such as casting with vibration, compaction, extrusion, and injection molding. Distributed cracks without macrocracking is usually exhibited. Flexural strengths of 5-10 times
ordinary concrete, with increases in deformation capacity of 15-30 times have been shown for specimens with 6% steel fibers, conventional rebar, and the inclusion of silica fume.[5]

There are several types of fibers from which one can choose for inclusion in concrete. These include carbon fibers, glass fibers, kevlar, steel, and polypropylene. Carbon fibers are very stiff and have a high tensile strength. They also resist alkaline deterioration. However, they are expensive. Glass fibers have good tensile strength and stiffness and are also relatively cheap. However, they deteriorate quickly in alkaline environments, so they may not be very effective for long term strength. Kevlar (aramid) fibers are very strong and stiff, but they are even more expensive than carbon. Their durability in concrete has yet to be verified.[1]

Steel fibers are quite stiff and strong. Though they corrode in harsh environments, they are more durable than steel stirrups and rebars. They are the most commonly available fibers and are also inexpensive, so they are used more often than any of the other types of fibers. They are available as straight fibers, hooked-end fibers, or deformed fibers. Straight fibers generally debond easily. The other fibers, due to the deformations, provide much higher bridging stresses when they debond and pull out. In some cases, fiber fracture may occur due to strong anchorage.[1]

Polypropylene fibers have been used as concrete reinforcement since the early 1960’s. They are fairly compliant and they debond more easily than steel fibers. However, they are extremely durable in alkaline environments and are also relatively inexpensive. They may come as fibrillated (accordion) fibers, monofilaments, or continuous films.[7] They are much more difficult to mix than steel fibers, as they often clump together and resist separation. They have
been mixed in volume fractions of 0.1%, 0.3%, and 0.5% in pan mixers in previous research.[8] However, the mixes that were effective in breaking up the fibers and having them distributed throughout the concrete were very specific. The mix was quite large for a laboratory cast, weighing over 100kg. The mix consisted of air-entrained concrete mixed with a 0.69 w/c ratio and superplasticizer. Thus, it was a low-strength, high workability mix uncommon in practice.[9] As denoted in Chapter 3, the polypropylene fibers required a different mixing procedure whenever they were used. For this research project, steel and polypropylene fibers were used in conjunction with composite reinforcing bars.

2.4 FRPs and Concrete

Fiber Reinforced Plastics may play a large role in concrete construction in the future. Though more expensive and more compliant than steel, FRPs have many uses as reinforcement for concrete. These include use as reinforcing bars, column wraps, and laminates used to retrofit deteriorated structures. In fact, FRP reinforcing bars have already been used in some pavement and bridge construction in the US. FRPs have also been widely used in Japan and some other countries.[10] When used as reinforcing bar, FRP rods are only marginally more expensive than epoxy-coated steel bars, which have become widely used recently for their corrosion prevention.[11]

There are several potential benefits to using FRPs. FRPs do not deteriorate in the presence of chloride ions. Coastal environments contain free chloride ions that attack steel. Likewise, deicing salts in northern environments provide ions that can attack and corrode steel. Since concrete always cracks under tension, steel rebar can be exposed to these damaging ions. As the
steel corrodes, it expands in volume and causes the underlying concrete to spall. If FRPs are used, the spalling can be avoided, and the concrete structure will have better durability. FRPs also may be useful in structures subject to magnetic fields, since unlike steel stirrups and reinforcing bars, they will not interfere with a magnetic field. One example of this type of use for FRP rods is use in reinforcement of concrete guideway structures on the MagLev train system.[12] Another example is rooms housing medical equipment that cannot have magnetic interference from the steel.

FRP rods may be used as prestressing tendons for pretensioning or post-tensioning concrete, and because their modulus is lower than that of steel rods, creep becomes less of a problem. However, the ultimate shear and flexural strengths of FRP-pretensioned beams are lower than those pretensioned with steel.[12] FRP rods may also have use as shear stirrups. However, FRP rods that are bent after setting weaken greatly at the bends, and fracture almost always occurs at those bends. A bend of 5mm may weaken the rod 50%, while one of 25 mm may weaken a rod by 65%.[13]

In retrofitting of structures, FRP laminates have proven useful due to their light weight, easy attachment via epoxies, and their high strength. These laminates may be attached to beams or wrapped around columns. After earthquake damage in Kobe, Japan, retrofitting of columns with composite jacketing became widespread in Japan.[12,13] A similar retrofitting trend, albeit on a smaller scale, took place after the recent earthquakes in California. Thus, jacketing with composites is an up-and-coming method in civil engineering. Confinement may be applied by wrapping composite bars, composite sheets, or composite meshes around the columns.[12,13]
Composite jackets afford several potential benefits over steel jacketing. One benefit is the resistance to weathering from salt and water. Another is the different behavior of the confined column. With steel jacketing, the concrete reaches a peak stress and then tapers off as the steel yields. With composite jacketing, yielding does not occur. Thus, the confined concrete reaches its maximum stress and strain at the same point, resulting in significantly improved ductility. In fact, the addition of composite wrapping may increase the failure strain by a factor of 6 or more. Typically, the stress/strain curve of the concrete remains linear up to the point where it would fail if unconfined. After that, the curve bends and the load increases with a lower stiffness, as the concrete cracks inside the column. The column finally fails when the composite fractures. The stiffer the confinement, the higher the final fracture load. Another benefit of using a composite wrap is that no mechanical destruction to the concrete column is required, as anchoring and bolting is replaced by epoxying or mechanically coupled belts. Also, there is the potential for pretensioning concrete columns with composite wraps that does not exist with steel jackets. Thus, composite jacketing has great potential for use in civil engineering structures.[12,13]

FRPs have been tested as reinforcing rods in concrete in recent years. They have even been used in the construction of several bridge decks. FRP bars fail at about twice the tensile load as steel rods, yet their stiffness is only 1/4 that of steel. The bars are much lighter than steel, with a specific gravity of about 2.2 for the C-bar tested in this project. In contrast, steel has a SG of 7.8. Early FRP bars that were used were smooth and did not bond well to concrete at all. Now, bars are ribbed similar to steel so the slippage problem is mitigated. FRP bars still do not bond to concrete as well as steel bars do. The bond strength is about 60-90% of the bond strength of
steel, but is adequate for flexural purposes. The mechanism of the bond between concrete and steel is not fully understood, but the bond is due more to the adhesion between the concrete and the steel than the deformations in the steel bar. FRPs do not have this same adhesion to concrete. In addition, since FRPs do not possess the same high shear strength and rigidity as steel, the bond strength for FRP bars is lower than that of steel bars. FRP bars develop their bond to concrete because of the deformations inherent in the bars. Because of the lower bond strength of FRP rods, beams fail in shear at lower stresses, as the dowel action contribution to shear strength is reduced. In addition, the shear cracks are more likely to propagate along the bar, rather than going around it, further reducing the dowel force. In fact, the maximum dowel capability has been determined to be only 70% of that of steel rebar.[13,14]

Many tests have been performed on the flexural behavior of beams reinforced with composite rods. They have shown that the composite bars perform well and can achieve full moment capability, as long as proper development length is provided. However, because FRP bars are typically 4 times more compliant than their steel counterparts, crack widths in FRP-reinforced beams are approximately 4 times as large.[12] The ultimate shear and flexural strengths of FRP-reinforced concrete are lower than those in steel-reinforced concrete for comparable reinforcement ratios. This may cause a serviceability or creep problem, but if enough reinforcement is provided, beams should still be structurally sound. [15,16]

FRP bars have also been tested for their resistance to alkali agents. Glass FRP bars showed slight weakening of their tensile strength, while Carbon FRP bars and Aramid FRP bars did not show any damage after exposure to alkali agents.[13] FRP bars have also been tested for their
extreme temperature behavior and resistance to cyclic loads. The glass bars showed some weakening at high temperatures, losing 31%, 45%, 60%, and 71% of their initial strength at 100, 200, 300, and 400°C, respectively. Steel bars, in comparison, typically lose only 14% of their strength at 400°C.[16] On the other hand, carbon FRP bars showed excellent behavior in cold environments. This is especially important, as cold weather climates are one of the major markets for composite bars, as mentioned above.[18] Under cyclic loading, glass bars showed excellent behavior and fatigue resistance.[16]

2.5 High Strength Concrete

High Strength Concrete (HSC) behaves quite differently than Normal Strength Concrete (NSC). Aside from the obvious failure at a higher load, HSC is much more brittle and fails in a more catastrophic manner. HSC is produced by reducing the water content of the concrete, as well as by adding silica fume to the mix. Silica fume consists of fine particles of silicon dioxide. It reacts with Ca(OH)$_2$ to form more gel. This fills the pores that would otherwise exist in concrete, thus increasing the density and compacting the mix. By eliminating the pores, the strength of the concrete is increased dramatically. While NSC typically reaches compressive strengths of 6000 psi, HSC has been known to reach strengths of up to 20,000 psi. Typically, though, one can achieve strengths of 9-13,000 psi.

The addition of silica fume also improves the bond between the concrete matrix and the aggregate. This increase in bond strength, coupled with the increase in the strength of the
concrete, forces cracks to propagate through the aggregate, rather than around the aggregate (along the concrete-aggregate interface) as in NSC.[1]

The danger in using HSC in practice, however, is its lack of ductility. Failure is typically sudden and catastrophic in tension and in shear. Thus, the addition of fibers into the mix may drastically improve the performance of HSC. Though they decrease the workability of the mix, once fibers are incorporated into HSC, its ductility improves, and the failure mode also changes. Rather than failing catastrophically and making pieces of concrete fly around the testing lab, the concrete merely cracks. The crack propagates until the concrete can take no more load, but it usually does not fail catastrophically or spall.

Fibers do not alter the compressive strength of HSC or NSC greatly. However, the shear behavior of fiber reinforced concrete beams is dramatically improved in HSC. Because of the improved bond between the concrete and fibers due to the addition of silica fume, the fibers do not pull out as easily as they do in NSC, and thus shear performance is greatly enhanced. While doubling the volume fraction of fibers does not double the increase in shear strength, the addition of more fibers (up to about 3% volume fraction) does increase the shear strength.[9]

Fiber reinforced HSC may perform better than plain HSC due to its high strength and improved ductility over plain HSC. HSC is quite stiff, thus reducing deflection over NSC. It creeps less and is less susceptible to fatigue failure. Since HSC has so many uses as a construction material, the use of composite reinforcement with HSC may enhance the use of HSC in northern and coastal environments. Also, since debonding is a concern with FRP reinforcement, the use of
HSC may be able to mitigate this problem, since there may be improved bonding with the bar.
Chapter 3

Experimental Program

3.1 Beam Design

To test the effectiveness of composite tensile reinforcement, both normal strength and high strength beams were cast with varying fiber content. For each type of concrete and fiber volume fraction, 2 beams were cast. The beams measured 3” x 4.5” x 21” overall, with effective size and span of 3” x 3.75” x 15”. The NSC mix was designed with $f'_c$ of approximately 6,500 psi, and the HSC was designed to have a strength of 10,000 psi. Each beam was tested in four point bending. The shear span was 5.5”, providing a shear span to effective depth ratio (a/d) of 1.47.

The flexural reinforcement provided was 2 #4 bars for NSC and 2 #5 bars for HSC, as shown in Figure 3-1. For the FRP rebar, the design tensile strength was 103 ksi for the #4 bars used in the NSC and 99 ksi for the #5 bars used to reinforce the HSC. The steel reinforcement had a failure stress of 75 ksi. However, while the stiffness of the steel bars measured 29,000 ksi, the stiffness of the FRP bar was only 6,500 ksi. The density of the FRP rebar was 138 lb/ft$^3$, while the density of the steel rebar measured 490 lb/ft$^3$. Thus, though stronger than steel, the composite was not as stiff nor as dense as the steel reinforcement.
Different fiber volume fractions were also used to test their effect on shear. Steel fibers, as shown in Figure 3-2, were used in volume fractions of 0.0%, 0.4%, and 0.8% for two beams each of NSC and of HSC. Similarly, polypropylene fibers (Figure 3-3) were used as reinforcement. 5 grams of fibers and 10 grams of fibers were used for two beams each of HSC and NSC. Based on a specific gravity of 0.9, this corresponds to polypropylene volume fractions of 0.05 and 0.1%. Thus, a total of 20 composite-reinforced beams were cast for shear testing. In addition,
Figure 3-2: Steel Fibers

Figure 3-3: Polypropylene Fibers
NSC and HSC beams reinforced with steel rebar were tested as a basis for comparison. Beams were prepared for each of the three steel fiber volume fractions. The beams were designed as deep beams, to develop some arch action. Enough tensile reinforcement was placed, however, to guarantee shear failure. In addition to the beams, three cylinders were cast from each mix to determine the compressive strength of the mix. This enabled a more accurate determination of the shear strength of each mix.

The chief purpose of these tests was to determine the shear behavior of beams reinforced with FRP tensile reinforcement. Another purpose was to investigate the effect of different fibers and fiber volume fractions on the shear behavior of concrete beams.

### 3.2 Wrapped Cylinder Design

To test the performance of the composite wrap on concrete cylinders, cylinders 6" in height and 3” in diameter were cast. Both normal strength and high strength mixes were tested. For each type of concrete and fiber fraction, three cylinders were cast. The cylinders were then compressed until failure. For these tests, only steel fibers were used as additional reinforcement. Volume fractions of 0.0%, 0.4%, and 0.8% were used, as with the beams. Three unwrapped control specimens were also cast from each mix. 30 cylinders were cast and tested to determine the efficacy of composite wrapping of concrete cylinders.
3.3 Mix Components

The following materials were used in the testing program:

1. Portland Cement -
   Type III portland cement, purchased from Waldo Bro., Co., Boston, MA, was used.

2. Aggregates -
   Pea gravel of maximum size 3/8” was used, purchased from Waldo Bro.

3. Sand -
   Mortar sand purchased from Waldo Bro. was used.

4. Silica fume -
   Condensed silica fume slurry donated by W.R. Grace & Co. of Cambridge, MA was used. It is a microsilica-based liquid admixture designed to increase concrete compressive and flexural strengths, increase durability, reduce permeability, and improve hydraulic abrasion-erosion resistance. Named Force 10,000, it contains 5.5 pounds of microsilica and weighs 11.5 pounds per gallon. By weight, it contains 48% silica fume, 50% water, and 2% foreign particles.

5. Superplasticizer -
   WRDA-19, an admixture donated by W.R. Grace, was used. It is an aqueous
solution of a modified naphthalene sulfonate. It was used to produce a more workable mixture in high strength concrete, without compromising concrete strength.

6. Steel Fibers -

Hooked-end steel fibers donated by Bakaert were used due to their strength, bonding properties, and ductility. They were 30mm long, had a diameter of 0.5mm, and a failure strength of 170 ksi.

7. Polypropylene Fibers -

Fibrillated polypropylene fibers, donated by W.R. Grace were used. The fibers had a specific gravity of 0.9 and length of approximately 0.75”.

8. Steel Reinforcement -

Reinforcing steel in the form of deformed #4 and #5 bars was purchased from Somerville Lumber, Somerville, MA. The bars had a yield strength of 75 ksi.

9. FRP Reinforcement -

Reinforcing #4 and #5 C-Bar GFRP bars were donated by Marshall Composites, Inc., Jacksonville, FL. Data on the FRP bars is included in Table 3-1. C-bar is produced by the thermosetting of the glass-reinforced urethane-modified vinyl ester resin composite into its final shape (straight or bent). It is a deformed bar that matches epoxy-coated rebar in appearance and is only slightly more expensive. Its tensile strength is twice that of steel, while its weight is 1/4 that of steel.
### C-BAR™ Reinforcing Rod Designation Number:

<table>
<thead>
<tr>
<th></th>
<th>#12</th>
<th>#15</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cross Section Barrel Diameter:</td>
<td>12 mm</td>
<td>15 mm</td>
</tr>
<tr>
<td>Area of Fiber Reinforcement:</td>
<td>113 mm²</td>
<td>176 mm²</td>
</tr>
<tr>
<td>Weight:</td>
<td>0.25 kg/m</td>
<td>0.37 kg/m</td>
</tr>
<tr>
<td>Water Absorption:</td>
<td>0.25% max.</td>
<td>0.25% max.</td>
</tr>
<tr>
<td>Barcol Hardness:</td>
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<td>60 min.</td>
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<tr>
<td>Average Ultimate Tensile Strength (f_u):</td>
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<td>746 N/mm²</td>
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<tr>
<td>Standard Deviation (σ)</td>
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<td>22 N/mm²</td>
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<tr>
<td>Design Ultimate Tensile Strength (f_u = f_u - 3σ)</td>
<td>713 N/mm²</td>
<td>680 N/mm²</td>
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<tr>
<td>Modulus of Elasticity (Average):</td>
<td>42,000 N/mm²</td>
<td>42,000 N/mm²</td>
</tr>
<tr>
<td>Bond Strength:</td>
<td>17 N/mm²</td>
<td>18 N/mm²</td>
</tr>
</tbody>
</table>

Table 3-1: Data on FRP Rebar

#### 3.4 Mix Design

<table>
<thead>
<tr>
<th>Ingredients</th>
<th>Normal Strength Mix</th>
<th>High Strength Mix</th>
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</thead>
<tbody>
<tr>
<td>Sand/Cement</td>
<td>2.0</td>
<td>2.0</td>
</tr>
<tr>
<td>Gravel/Cement</td>
<td>2.0</td>
<td>2.0</td>
</tr>
<tr>
<td>Silica Fume/Cement</td>
<td>0.0</td>
<td>0.1</td>
</tr>
<tr>
<td>Superplasticizer/Cement</td>
<td>0.0</td>
<td>0.03</td>
</tr>
<tr>
<td>Water/(Cement + Silica Fume)</td>
<td>0.5</td>
<td>0.3</td>
</tr>
</tbody>
</table>

Tested steel fiber volume fractions: 0.4%, 0.8%.
Tested polypropylene weight fractions: 5g/60.5# (volume fraction of 0.05%, using polypropylene SG = 0.9), 10g/60.5# (volume fraction of 0.1%).

3.5 Specimen Design

The specimens were designed to fail in shear, rather than in flexure or compression, to test the shear capability provided by the C-bar. Research done by Amjad Shahbazker on shear behavior of fiber-reinforced HSC beams was used as the basis for this project. Following his reinforcement selection to insure no tensile failure, 2 #5 bars were used as tensile reinforcement for the HSC, and 2 #4 bars were used as tensile reinforcement for the NSC specimens. For the HSC beams, using ACI design provisions for HSC and fiber-reinforced concrete[19,20], the predicted flexural strength was approximately 49.0 kips for every specimen, while the predicted shear capacity (Vc) was approximately 10-15 kips, providing a failure load (Pu) for shear of under 30 kips. Again using the ACI design provisions for steel fiber reinforced concrete[20], the NSC beams, the flexural failure load was approximately 28 kips, while the predicted shear failure load was approximately 10 kips, yielding a failure load of approximately 20 kips. Therefore, shear failure was assured.[1]

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Vc (kips)</th>
<th>Pu (kips)</th>
<th>Calculated Flexural Strength (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NSC-00</td>
<td>9.21</td>
<td>18.42</td>
<td>27.6</td>
</tr>
<tr>
<td>NSC-S4</td>
<td>10.53</td>
<td>21.06</td>
<td>27.7</td>
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<tr>
<td>NSC-S8</td>
<td>11.86</td>
<td>23.72</td>
<td>27.8</td>
</tr>
<tr>
<td>HSC-00</td>
<td>10.78</td>
<td>21.56</td>
<td>48.9</td>
</tr>
<tr>
<td>HSC-S4</td>
<td>13.26</td>
<td>26.52</td>
<td>49.0</td>
</tr>
<tr>
<td>HSC-S8</td>
<td>15.75</td>
<td>31.50</td>
<td>49.1</td>
</tr>
</tbody>
</table>
3.6 Mix Procedure

The mold used by Amjad Shahbazker for his shear tests was used for this project. The mold is made of 1/2” thick plexiglas. It is held together by 1 inch bolts. The mold is designed to hold two beams at once, each of size 3” x 4.5” x 21”. The mold is pictured in Figure 3-4.

To make the concrete, a standard rotating drum mixer (Figure 3-5) was used. This mixer was chosen because it has fewer blades than other mixers. There are gaps underneath the blades to allow fibers and concrete to flow through. Also, two of the four blades do not reach the back of the mixer, so they do not interfere with the mix.
To make NSC, the mix procedure was fairly simple: the sand, gravel, and cement were added in that order. The components were mixed together for one minute. Then, water was added and the contents were mixed for 9 minutes.

For HSC, the procedure was similar. For plain HSC, the sand, gravel, and cement were placed in the mixer in their proper proportions, and the mixer was run for one minute. Then, water was measured out in a plastic bin. Superplasticizer was added to that, and the two were mixed together so the superplasticizer did not just sit on the bottom. The silica fume slurry was stirred to form a uniform mixture and then the proper amount of that was added to the water-superplasticizer mixture. The water content of the silica fume was accounted for in the determination of the required amount of water. The conglomeration of water, superplasticizer,
and silica fume was then mixed together until uniformity was achieved. A small amount of this mixture was placed in the mixer, and the mixer was run for several seconds. The mixer was stopped and more of the mixture was added. The mixer was again run for several seconds and stopped. This continued until all of the mixture was added, upon which the mixer was run for 9 minutes. The mixture was added in this manner to prevent the silica fume from spilling onto the mixing drum and sticking to it, rather than mixing into the concrete mix.

For steel fiber reinforced concrete, the sand was first added to the mixer. 1/3 of the fibers were then sprinkled by hand onto the sand to prevent balling. The sand and fibers were then mixed together for one minute. The gravel was then added, and another 1/3 of the fibers were sprinkled on, and the mixer was again run for one minute. Then, the cement and remainder of the fibers were added to the mixer and the mixer was run for one minute. Any fibers that may have balled or clumped were separated by hand and sprinkled back in the mixer. The water/plasticizing mix was then added and the mixer was run for 9 minutes.

Polypropylene fibers, however, posed a much more difficult problem for mixing. Because they were fibrillated, it was desired that they be separated prior to mixing. Test mixes were tried to determine the best method to separate the fibers and prevent them from clumping together in balls or around the blades of the mixer. One attempt was to add the polypropylene to all the other components, but the fibers did not separate from their initial fibrillated state. In one trial, an aggregate-only mix was used to break up the fibers and that only served to allow the fibers to ball around the aggregate. Finally, a mix of the fibers with only sand and cement was tried. This was the most successful. However, to make it work best, the fibers had to be hand separated.
This was very time-consuming, but it made the mix much easier to work with. It also allowed for a small amount of fibers to provide a noticeable improvement in shear strength (see Chapter 4 for details). Thus, the fibers were broken up by hand and then placed in the mixer with the sand and cement. The mixer was run for 3 minutes and then the aggregate was added. The water/plasticizing mix was added to that and the mixer run for 9 minutes.

Upon completion of the mixing, the concrete was placed in their appropriate mold: the plexiglas mold for beams and plastic cylindrical molds for the cylinders. The reinforcing bars were suspended at their proper height by wires that were affixed to the mold by duct tape. The molds were lubricated with WD-40 to ensure easy specimen removal. Once concrete was placed in its appropriate mold, the specimen was vibrated on a shaking table for 10 minutes. The vibration was especially necessary for HSC, as the concrete was not very workable and it was vital that the concrete not have air voids in it. Tamping by hand was not an option as that would orient the fibers, and they had to be randomly distributed. As the concrete was poured into the mold, the steel rebar would sometimes sink to the bottom, so care had to be taken to ensure that the steel would stay at the proper height. The C-bar had the opposite problem. Since it was lighter than the concrete, it tended to rise to the top during vibration. Several times, it had to be pressed down with a screwdriver to keep the reinforcement from sliding to one side.

Once the concrete was vibrated, the molds were placed in water for the first 24 hours. Then, the concrete was removed from the molds and cured underwater for 7 days. Upon reaching 7 days, the concrete was removed from the water. The beams were tested on that day, as were the control compression test specimens pertaining to those beams.
For the wrapped cylinder tests, the control specimens and the specimens to be wrapped were also removed from the bath on the 7th day. The control specimens air-cured for 7 days until testing. However, the specimens to be confined had to be wrapped. A two-part epoxy, donated by Hexcel Fyfe of San Diego, California, was mixed together in the proper proportions with a hand-held electric kitchen mixer. The fiberglass-reinforced wrap was saturated in the pre-mixed epoxy. The excess epoxy was squeegeed off the wrap. After a primer coat of epoxy was applied to the dried concrete cylinder, the cylinder was wrapped with a fiberglass-reinforced composite, TYFO-S, also donated by Hexcel Fyfe (Figure 3-6). The fiberglass fibers were oriented in the hoop direction, so as to resist the expansion of the concrete under compressive load. Care had to be taken to ensure that there were no gaps between the wrap and the cylinder. Any air pockets would immediately create an air gap not providing confinement, and the specimen would fail at a

Figure 3-6: TYFO-S Fibrwrap
much lower load. Rubber bands were placed around the cylinders to prevent the wrap from debonding before the epoxy set. The epoxy was allowed to cure 7 days. Thus, the wrapped specimens were tested 14 days after the concrete was cast, and 7 days after being wrapped with the composite.

3.7 Testing Apparatus and Procedure

3.7.1 Beam Tests

For all of the beams that were tested for shear failure, the 60 kip Baldwin testing machine (Figure 3-7) was used. The test setup is shown in Figure 3-8. The specimens were tested in 4-point bending. The supports consisted of semi-cylindrical steel rods. The supports were placed on hollow steel blocks to allow for deflection. A Linear Variance Displacement Transducer (LVDT) was placed under the center of the beam to measure displacement. Load was transferred from the machine to the specimen via a steel I-beam that transferred the load to two rectangular steel plates. The plates connected to a roller that transferred the load to a pad at each of the load points. The roller ensured that the load would be placed on the specimen without any eccentricity. The steel plates were 1 inch wide, along the longitudinal direction of the beam, to place the load over a wider area and avoid local crushing of the beam.

A data acquisition machine was connected to the load cell from the testing apparatus and the LVDT, to measure load and displacement. The recorded voltage readings were converted into raw data on a spreadsheet. Data was recorded every second, the quickest time between recordings that the data acquisition system could handle. The specimens were loaded until failure and then unloaded.
Figure 3-7: 60 kip Baldwin Testing Machine

Figure 3-8: Beam Shear Test Setup
Control cylinders also had to be tested to determine the compressive strength of the concrete. The NSC specimens were tested with the 60 kip Baldwin machine. The failure load was manually recorded from the readout on the dial gauge on the testing machine. Before testing, the cylinders were capped with hydrostone to ensure an even, smooth, and level surface between the cylinder and the testing machine. Capping also reduces the friction between the loading apparatus and the cylinders.

The HSC specimens, however, were too strong for the 60 kip machine. They were tested on the 200 kip Baldwin machine (Figure 3-9). This machine has a digital readout of the current and

Figure 3-9: 200 kip Baldwin Testing Machine
maximum loads. The maximum load was manually recorded from this readout. As with the NSC cylinders, the HSC cylinders were also capped prior to testing.

3.7.2 Confined Cylinder Tests

The unconfined cylinders could be capped before testing to ensure a flat, level, and smooth surface to transfer load from the machine to the specimen. However, the displacement from the capping would add an uncontrollable amount to the measured strain of the cylinders and this was not desirable. Also, it would be difficult to ensure that no capping material stuck to the composite wrap, so this procedure was rejected. Instead, the cylinders were cut with a diamond-tipped saw, as shown in Figure 3-10. The specimens were clamped down after being leveled, and the saw cut across the cross-section of the wrapped cylinder, cutting through the wrap and the concrete. Load was then applied to the specimen through a pair of cylindrical steel blocks. The steel blocks had a diameter of 2.8”, so the load was placed only on the concrete and not on the composite wrap.

The confined cylinders, as well as the control specimens that pertained to them, were tested on the 200-kip Baldwin testing machine. This testing apparatus has an automatic data acquisition system and converts the voltages to actual values, once all the transducers have been calibrated. For these compression tests, the load and crosshead displacement were recorded. In addition, two LVDTs were connected to steel blocks that transferred the load from the machine to the concrete. The testing apparatus is shown in Figure 3-11. The LVDTs measured the displacement of the concrete and part of the steel rod for the calculation of the applied strain; the displacement of the steel rod was subtracted out. These LVDTs were also connected to the data
acquisition system for the testing machine and were included in the machine’s data file. The
displacements recorded on either side were averaged to determine the actual strain. The data
acquisition system built into the machine was programmed to take 4 readings per second.
Figure 3-10: Saw Used to Cut Cylinders

Figure 3-11: Cylinder Test Setup
Observations and Results of Shear Behavior of Beams

4.1 Results

Because the beams were adequately reinforced against flexure and no steel stirrups were provided, all the beams failed in shear. For the FRP-reinforced beams, flexural cracks were evident, as were shear-flexure, but failure was solely due to the diagonal tension crack that propagated between the load point and the support. For the FRP rebar, the crack propagated along the bar to the end of the beam, while the crack propagated across the steel rebar towards the support in the steel-reinforced beams. This is due to the better bond between the reinforcement and the concrete with steel. The plain concrete specimens developed one shear crack that propagated until failure, where the beam split. Polypropylene fibers incorporated into the mix increased the final strength of the specimens but in every test, one crack led to sudden and catastrophic failure, with pieces of concrete sometimes flying across the room. When steel fibers were included, failure generally occurred with two shear cracks next to each other. The first one would not cause failure because fibers bridged it adequately, but the other one, which was usually closer to the support than the load point, would start less than half an inch away, and when it propagated across the beam, the specimen would fail and the load capacity dropped.
The data for concrete compressive strength, failure load, deflection at failure, and $V_u$ is presented in Table 4-1. $V_u$ is defined as $P_u/2bwd$.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Compressive Strength (psi)</th>
<th>Failure Load (pounds)</th>
<th>Deflection at Failure (inches)</th>
<th>$V_u$ (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NSC-00-C1</td>
<td>5,490</td>
<td>16,300</td>
<td>0.0844</td>
<td>725</td>
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<tr>
<td>NSC-00-C2</td>
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<td>16,900</td>
<td>0.100</td>
<td>751</td>
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<td>14,765</td>
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<td>8,500</td>
<td>37,100</td>
<td>0.0910</td>
<td>1,650</td>
</tr>
<tr>
<td>HSC-00-S2</td>
<td>8,500</td>
<td>29,440</td>
<td>0.1135</td>
<td>1,308</td>
</tr>
<tr>
<td>HSC-S4-S1</td>
<td>8,700</td>
<td>39,205</td>
<td>0.1044</td>
<td>1,742</td>
</tr>
<tr>
<td>HSC-S4-S2</td>
<td>8,700</td>
<td>39,435</td>
<td>0.1059</td>
<td>1,753</td>
</tr>
<tr>
<td>HSC-S8-S1</td>
<td>9,430</td>
<td>41,150</td>
<td>0.1541</td>
<td>1,829</td>
</tr>
<tr>
<td>HSC-S8-S2</td>
<td>9,430</td>
<td>31,940</td>
<td>0.0854</td>
<td>1,420</td>
</tr>
</tbody>
</table>

Table 4-1: Results of Shear Tests on Beams
4.2 Comparison of Steel and FRP Rebars

Both FRP and steel provided enough resistance to flexural cracking to insure failure by shear, rather than flexure. However, the behavior of each was slightly different. Because the FRP rebars are not as stiff as the steel rebar, the beam was allowed to deflect more. Also, because the C-bar did not bond to the concrete as well as the steel did, more cracking was allowed on the tensile side of the beam. In fact, no flexural cracking was evident in any of the beams reinforced with steel rods. However, tensile cracking started at approximately 10,000 pounds of load for the FRP-reinforced NSC beams, and 20,000 pounds for the HSC ones. In the FRP-reinforced NSC beams, shear-flexure cracking propagated from some of the flexural cracks. This, too, was not the case with steel. Thus, cracking was much more prevalent in C-bar-reinforced beams. However, failure did not arise from flexural cracks in any specimens. Instead, all the beams failed due to one or two shear cracks propagating from a load point to a support.

The weaker adhesion between the C-bar and the concrete, though, may contribute to the lower shear strength in the FRP-reinforced concrete. Because the C-bar doesn’t arrest cracks as well as a steel one, the cracks cause failure at a lower load. Also, when failure occurred in the composite-reinforced bars, many times, the crack propagated along the FRP bar, debonding the bar from the concrete completely. Because the steel adheres better to the concrete, cracks tended to go around the steel, and the concrete stayed together better. Thus, the final failure mode was slightly different between steel and FRP reinforced beams.

The chief difference between FRP-reinforced and steel-reinforced concrete beams is the reduced crack control efficiency with the FRP rebar. Though it develops an adequate bond to the
concrete and serves well in tension, the bars do not contribute as much as steel bars do to the shear strength of the beams. The results shown in Table 4-2 provide support to this argument. The % increase refers to increase in strength over the non-fiber reinforced specimens of the same type.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>FRP-Reinforced</th>
<th>Steel-Reinforced</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Vu (psi)</td>
<td>Increase (psi)</td>
</tr>
<tr>
<td>NSC-00</td>
<td>740</td>
<td>-</td>
</tr>
<tr>
<td>NSC-S4</td>
<td>715</td>
<td>-25</td>
</tr>
<tr>
<td>NSC-S8</td>
<td>990</td>
<td>250</td>
</tr>
<tr>
<td>NSC-P5</td>
<td>780</td>
<td>40</td>
</tr>
<tr>
<td>NSC-P10</td>
<td>825</td>
<td>85</td>
</tr>
<tr>
<td>HSC-00</td>
<td>950</td>
<td>-</td>
</tr>
<tr>
<td>HSC-S4</td>
<td>1,360</td>
<td>410</td>
</tr>
<tr>
<td>HSC-S8</td>
<td>1,270</td>
<td>320</td>
</tr>
<tr>
<td>HSC-P5</td>
<td>1,025</td>
<td>75</td>
</tr>
<tr>
<td>HSC-P10</td>
<td>1,085</td>
<td>135</td>
</tr>
</tbody>
</table>

Table 4-2: Effect of Fiber Reinforcement on FRP and Steel Reinforced Beams.

The inclusion of fibers increased the shear strength of most of the specimens. Though a discussion of the effect of fibers and the effect of the different types of concrete will be conducted later in the chapter, a comparison of the increase in shear strength between composite-reinforced and steel-reinforced beams is presented. Though it is difficult to draw conclusions from just 2 beams per mix, a noted improvement in shear strength existed. For NSC, the increase is similar, whichever flexural reinforcement was used, while for HSC, the FRP-reinforced beams had a much larger increase in strength.
The HSC beams reinforced with FRP bars all had much lower strengths than those reinforced with steel rebar. While the steel-reinforced NSC specimens were also stronger than the FRP-reinforced NSC beams, the difference between steel and FRP-reinforced specimens was much greater in the HSC specimens. This is why the inclusion of fibers had such a profound effect on the shear strength of HSC beams reinforced with FRP bars. With larger crack openings, the dowel and aggregate interlock components are also reduced. The fibers bridge cracks, thus increasing each of these components. Thus, there is a synergetic effect of fiber reinforcement and composite rebar, as crack openings are now better controlled.

Research should be conducted to determine why FRP-reinforced HSC is so much weaker than steel-reinforced HSC. One reason could be the brittleness of the HSC beams. HSC is very brittle, and cracking occurs much earlier in the beams reinforced with the more compliant FRP bars. Because of the weaker bond between FRP bars and concrete, cracking is not controlled as well as when steel bars are used. Because of this combination of earlier cracking that is not well controlled in a very brittle matrix, failure occurs earlier in FRP-reinforced HSC beams.

There were some anomalies, however. For example, the FRP-reinforced NSC-S4 specimens did not conform to the trend of fibers improving the beam’s shear strength. One of the two beams showed marginal improvement in shear behavior over the non fiber-reinforced specimens, while the other was weaker. This may have been caused by random material variation. Also, the HSC-S8-C1 and HSC-S8-C2 specimens did not show much as much increase in shear strength as expected. This was probably due to a mixing problem with the extremely low workability of
those specimens. The HSC-S8-C3 and HSC-S8-C4 beams showed results that are much more consistent with the rest of the data.

4.3 Effect of Fibers on Shear Strength of Beams

4.3.1 General Observations

As shown in Table 4-2, the inclusion of fibers did have an effect on the shear strength of the concrete. Although the shear strength was slightly lower for the 0.4% steel fiber reinforced NSC, this can be considered an anomaly, as every other fiber addition served to increase the strength of the beam. The increase was slightly larger for HSC than NSC for the FRP-reinforced beams, as expected, due to the better bonding between HSC and the fibers than with the NSC and the fibers. However, the increase was about the same for NSC and HSC for the steel-reinforced beams. The fibers also proved to have an effect on the ductility of the beams. Though not very dramatic, there was a noticeable plateau in the load-displacement curves of the fiber-reinforced specimens, as shown in Figure 4-1 (NSC-S8-C2). Also, many of the steel fiber-reinforced specimens did not fail catastrophically. They were still able to carry load after initial failure, showing post-peak ductility.

4.3.2 Steel fibers vs. Polypropylene fibers

There were several noticeable differences between the behavior of beams reinforced with steel fibers and those reinforced with polypropylene fibers. Steel fibers were much more effective at bridging cracks and providing ductility. They provided approximately 3 times as much shear resistance as the polypropylene fibers did. However, it must be noted that only 5g or 10g of polypropylene were added to 60.5 pound mixes, corresponding to fiber volume fractions of
0.05% and 0.1%. Thus, there was approximately 1/8 volume fraction of polypropylene as compared to the 0.4% (356g) and 0.8% (705g) steel volume fractions. Thus, a perfect direct comparison between steel and polypropylene fibers is difficult. However, it is unclear whether an increase in the volume fraction would be practical, considering the workability problems associated with polypropylene.
Fibers had to either fracture or pull out for the specimen to fail. With the steel fibers, a metallic sound was audible as the load increased and the fibers were holding together the concrete. It was unclear whether they were breaking or pulling out from that, but upon inspection, most of the fibers seemed intact. Thus, most of the steel fibers pulled out upon failure. This would also explain why the steel fiber-reinforced beams possessed ductility after initial failure. There were enough fibers still embedded to allow the beam to carry some load. Some polypropylene fibers also pulled out rather than fracture. However, many of them broke at failure. Because of this fracture, the beam strength dropped dramatically and the beam failed catastrophically once the load capacity was reached.

The two types of fibers offered stark contrasts regarding ductility, as shown in Table 4-3. The steel fibers had a definite impact on the ductility of the beams. The beams reinforced with steel fibers improved the pre-peak ductility of the beams in some cases. In most cases, though, failure was still quite sudden, and plastic deformation was extremely minimal. However, the post-peak ductility of the steel fiber-reinforced specimens was markedly improved over the nonreinforced specimens. Post-peak ductility is the ability of a beam to carry load after beam failure occurs, as shown in Figure 4-2 (HSC-S8-C2). Aside from the FRP-reinforced HSC specimens, there was a definite trend of higher post-peak ductility with higher steel fiber volume. However, the polypropylene fibers had a very detrimental effect on the post-peak ductility of the beams. Though they increased the strength of the beams, the beams showed no plastic deformation before failing, and the failure was sudden and completely catastrophic. The beams could resist no more load, practically, after failure. Thus, a small amount of polypropylene fibers can
significantly increase the shear strength of concrete beams, but care must be taken when they are
used or beams will fail suddenly and catastrophically.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Load at Failure (pounds)</th>
<th>Deflection at Failure (inches)</th>
<th>Post-Peak Load (pounds)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NSC-00-C</td>
<td>16,600</td>
<td>0.0922</td>
<td>4,750</td>
</tr>
<tr>
<td>NSC-S4-C</td>
<td>16,080</td>
<td>0.1227</td>
<td>7,000</td>
</tr>
<tr>
<td>NSC-S8-C</td>
<td>22,225</td>
<td>0.16215</td>
<td>16,000</td>
</tr>
<tr>
<td>NSC-P5-C</td>
<td>17,520</td>
<td>0.105845</td>
<td>1,900</td>
</tr>
<tr>
<td>NSC-P10-C</td>
<td>18,560</td>
<td>0.11315</td>
<td>1,950</td>
</tr>
<tr>
<td>HSC-00-C</td>
<td>21,355</td>
<td>0.10355</td>
<td>14,750</td>
</tr>
<tr>
<td>HSC-S4-C</td>
<td>30,775</td>
<td>0.1154</td>
<td>950</td>
</tr>
<tr>
<td>HSC-S8-C</td>
<td>28,570</td>
<td>0.098275</td>
<td>13,025</td>
</tr>
<tr>
<td>HSC-P5-C</td>
<td>23,020</td>
<td>0.08995</td>
<td>2,500</td>
</tr>
<tr>
<td>HSC-P10-C</td>
<td>24,435</td>
<td>0.1036</td>
<td>2,150</td>
</tr>
<tr>
<td>NSC-00-S</td>
<td>20,005</td>
<td>0.07015</td>
<td>9,000</td>
</tr>
<tr>
<td>NSC-S4-S</td>
<td>24,100</td>
<td>0.0676</td>
<td>18,000</td>
</tr>
<tr>
<td>NSC-S8-S</td>
<td>26,285</td>
<td>0.1295</td>
<td>21,000</td>
</tr>
<tr>
<td>HSC-00-S</td>
<td>33,270</td>
<td>0.10225</td>
<td>3,600</td>
</tr>
<tr>
<td>HSC-S4-S</td>
<td>39,320</td>
<td>0.10515</td>
<td>14,000</td>
</tr>
<tr>
<td>HSC-S8-S</td>
<td>36,545</td>
<td>0.11975</td>
<td>25,000</td>
</tr>
</tbody>
</table>

Table 4-3: Beam Ductility

4.3.3 Comparison of Steel Fibers with Stirrups

The steel fibers used in this project had a length of 30 mm (1.18"). Their diameter was 0.5 mm
1/50", providing an area of 0.000314 sq. in. Thus, the volume of each fiber was .0037 cu. in.

The total volume of the fibers in the 0.4% beams was .07 cu. in. The steel fibers had a failure
stress of 170 ksi. If all of the steel fibers were replaced by steel stirrups of 75 ksi failure strength,
the expected increase in shear strength would be 300 psi, using the formula \( V_s = A_s f_y / s b_w \), where
the term \( A_s / s b_w \) is 0.004 and \( b_w \) is 3 in. For the 0.8% steel fiber reinforced beams, if the fibers
were replaced by 75 ksi steel stirrups, the increase in shear strength should be 600 psi. For the
stirrups, the area considered is the vertical area of the stirrups, not total area. Actual increases in
shear strength are shown in Table 4-4.
<table>
<thead>
<tr>
<th>Specimen</th>
<th>Shear Strength (C-bar)</th>
<th>Increase</th>
<th>Shear Strength (Steel Rebar)</th>
<th>Increase</th>
</tr>
</thead>
<tbody>
<tr>
<td>NSC-00</td>
<td>740</td>
<td>0</td>
<td>890</td>
<td>0</td>
</tr>
<tr>
<td>NSC-S4</td>
<td>715</td>
<td>-25</td>
<td>1070</td>
<td>180</td>
</tr>
<tr>
<td>NSC-S8</td>
<td>990</td>
<td>250</td>
<td>1170</td>
<td>280</td>
</tr>
<tr>
<td>HSC-00</td>
<td>950</td>
<td>0</td>
<td>1480</td>
<td>0</td>
</tr>
<tr>
<td>HSC-S4</td>
<td>1360</td>
<td>410</td>
<td>1750</td>
<td>270</td>
</tr>
<tr>
<td>HSC-S8</td>
<td>1270</td>
<td>320</td>
<td>1625</td>
<td>145</td>
</tr>
</tbody>
</table>

Table 4-4: Shear Strength Provided By Steel Fibers

Figure 4-2: Fiber Reinforced Beam Showing Post-peak Ductility
The increase in shear strength was typically lower than the predicted increase if stirrups had been used instead of fibers. However, it must be noted that this calculation assumes 75 ksi steel and not 170 ksi steel, the strength of the steel fibers. Aside from the HSC-S4 FRP-reinforced anomaly, all values were lower than the ones predicted for stirrups. The FRP-reinforced beams showed a larger increase than the steel-reinforced beams, but the values were still lower than those achieved with stirrups. Surprisingly, the HSC-S4 specimens showed larger increases in shear strength than the HSC-S8 specimens. This is likely due to the difficulty in mixing HSC with so many fibers included in the mix. Steel stirrups will be more effective than an equivalent volume of steel fibers.

4.4 HSC vs. NSC

As mentioned in Chapter 2, the two main differences between HSC and NSC are the increased strength of HSC and also its increased brittleness. The increase in strength is due to the inclusion of less water in the concrete, and the presence of silica fume, which increases the strength and bond strength of the concrete matrix. Most of the NSC specimens had a compressive strength of approximately 5,500 psi for the beams reinforced with composite rebar, and 6,250 psi for those reinforced with steel, though this has nothing to do with the type of rebar, only the specific mix. The HSC specimens had compressive strengths of approximately 8,500 psi. The failure strength of the concrete increased in close to this proportion. Thus, HSC is as effective as NSC in shear, relative to its strength. The major difference between HSC and NSC specimens in shear was the effect of fibers. In HSC, the increase in shear strength was much greater with the inclusion of fibers, as the fibers did not pull out as easily as they did in the NSC mixes. The HSC was more brittle than NSC, but failure modes did not change between the two types of concrete. Thus, the
only significant difference was due to the improved behavior of fibers with HSC rather than NSC.

4.5 Summary of Observations

There were two major parameters that were investigated in this part of the research project. One was the effectiveness of composite reinforcing bars. The second was the effect of fibers on the shear strength of the different beams. C-bar proved to be very effective as tensile reinforcement for concrete. However, because it is not as stiff as steel and does not bond as well to concrete as steel does, cracking was much more prevalent. However, those cracks did not lead to failure. Every beam, no matter how it was reinforced, failed due to diagonal tension cracking. Although failure occurred at a lower load for beams reinforced with C-bar, the difference was not very large. C-bar proved effective as concrete reinforcement.

The fibers introduced into the concrete improved the performance of the concrete. Steel fibers improved the strength and ductility of the concrete beams, while the polypropylene fibers increased only strength. The polypropylene fibers showed a marked increase in shear strength for such a small fraction added to a mix, but workability problems prevented the addition of higher volume fractions. Failure in beams reinforced with polypropylene fibers was always sudden and catastrophic, as opposed to the steel fiber-reinforced beams which failed after some softening, and usually the failure was not catastrophic.
Both HSC and NSC beams showed improvement in shear behavior with the incorporation of fibers. HSC beams showed more improvement with the incorporation of fibers with respect to load capacity and, especially, ductility. However, NSC also showed improvement with fibers. C-bar proved to be effective enough as tensile reinforcement to insure shear failure of all the beams. In a comparison of shear performance to steel reinforcement of the same size, C-bar proved to be more effective with NSC than HSC, which is surprising because of the better bond between HSC and whatever components are introduced into concrete. Further research should be conducted to determine the extent to which this is true.

FRP-reinforced concrete performed adequately in shear, and the addition of fibers can further improve its shear resistance. The main concern about the performance of FRP-reinforced concrete would be the increased cracking and deflection that composite-reinforced beams suffer from, so methods to remedy that should also be investigated.
Chapter 5

Performance of Composite Wraps

5.1 General Observations

As expected, the composite wraps greatly increased the strength of the concrete cylinders. The cylinders were 6” long, with a 3” diameter. This provided a cross-sectional area of approximately 7 sq. in. The wrapped cylinders were cut to 5.5” in length to insure a smooth surface without the need of capping that could transfer compressive load to the wrap. The surface of the cylinder may still have had some roughness, though, as evidenced by the small seating error at the start of the load curves. The tensile strength of the composite wrap was 65,000 psi. Its thickness was .045”, and its failure elongation is 2-4%.. For a 5.5” cylinder of 3 inch diameter, the surface area is 51.8 sq. in. per wrap providing 103.7 sq. in. to resist the expanding concrete cylinder. If the wrap develops full strength and behaves according to the formula

$$\sigma = \sigma_{\text{unconfined}} + 4.1\sigma_{\text{confined}}$$

the wrap should provide 15,990 psi of additional compressive strength. The wraps provided excellent strength and ductility to the cylinders. Increases in strength of 200% were observed in some cases, and the ductility provided allowed for strains of over 4 times the plain concrete strain. For the confined NSC specimens, the curve would typically remain linear to just past where the plain concrete would fail, then the load increased at a lower slope until failure, as shown in Figure 5-1. For the wrapped HSC specimens, though, the curve remained nearly linear until failure, as shown in Figure 5-2. Often times several of the
fibers broke and the load dropped partially, but then the other fibers carried the extra load and
loading could continue until many more fibers broke. The fracture usually occurred at one
localized location, possibly revealing that a stress concentration or weak spot in the wrap
initiated failure. The results of the tests are summarized in Table 5-1. In this table, the maximum
stress is the highest stress the cylinder reached, while the failure stress was the point where the
cylinder failed and could not take any more load (See Figure 5-1). The strain at maximum stress
is the strain at the point of the maximum stress listed before, while ultimate strain is the strain at
the point of failure.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Failure Load (pounds)</th>
<th>Maximum Stress (psi)</th>
<th>Failure Stress (psi)</th>
<th>Strain at Maximum Stress</th>
<th>Ultimate Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>NSC-00-N1</td>
<td>33,500</td>
<td>4,740</td>
<td>4,250</td>
<td>0.0071</td>
<td>0.0082</td>
</tr>
<tr>
<td>NSC-00-N2</td>
<td>38,150</td>
<td>5,400</td>
<td>4,850</td>
<td>0.0072</td>
<td>0.0092</td>
</tr>
<tr>
<td>NSC-00-W1</td>
<td>94,110</td>
<td>13,310</td>
<td>13,310</td>
<td>0.0356</td>
<td>0.0356</td>
</tr>
<tr>
<td>NSC-00-W2</td>
<td>98,260</td>
<td>13,900</td>
<td>13,900</td>
<td>0.0410</td>
<td>0.0410</td>
</tr>
<tr>
<td>NSC-S4-N1</td>
<td>42,040</td>
<td>5,950</td>
<td>5,930</td>
<td>0.0108</td>
<td>0.0120</td>
</tr>
<tr>
<td>NSC-S4-N2</td>
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<td>6,845</td>
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<td>0.0091</td>
</tr>
<tr>
<td>NSC-S4-W1</td>
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<td>11,515</td>
<td>9,290</td>
<td>0.0231</td>
<td>0.0309</td>
</tr>
<tr>
<td>NSC-S4-W2</td>
<td>89,130</td>
<td>12,600</td>
<td>11,350</td>
<td>0.0282</td>
<td>0.0333</td>
</tr>
<tr>
<td>NSC-S8-N1</td>
<td>51,930</td>
<td>7,345</td>
<td>7,345</td>
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<td>0.0083</td>
</tr>
<tr>
<td>NSC-S8-N2</td>
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<td>18,075</td>
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<td>0.0300</td>
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<td>17,560</td>
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<td>0.0452</td>
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<td>7,290</td>
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</tr>
<tr>
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<td>18,385</td>
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<td>0.0302</td>
</tr>
<tr>
<td>HSC-S4-W3</td>
<td>128,230</td>
<td>19,220</td>
<td>18,140</td>
<td>0.0257</td>
<td>0.0257</td>
</tr>
<tr>
<td>HSC-S8-N1</td>
<td>79,380</td>
<td>11,230</td>
<td>11,230</td>
<td>0.0144</td>
<td>0.0144</td>
</tr>
<tr>
<td>HSC-S8-N2</td>
<td>81,110</td>
<td>11,475</td>
<td>11,475</td>
<td>0.0142</td>
<td>0.0142</td>
</tr>
<tr>
<td>HSC-S8-N3</td>
<td>72,780</td>
<td>10,295</td>
<td>10,295</td>
<td>0.0115</td>
<td>0.0115</td>
</tr>
<tr>
<td>HSC-S8-W1</td>
<td>150,230</td>
<td>21,350</td>
<td>21,250</td>
<td>0.0278</td>
<td>0.0278</td>
</tr>
<tr>
<td>HSC-S8-W2</td>
<td>122,190</td>
<td>17,850</td>
<td>17,285</td>
<td>0.0243</td>
<td>0.0243</td>
</tr>
<tr>
<td>HSC-S8-W3</td>
<td>145,490</td>
<td>20,665</td>
<td>20,580</td>
<td>0.0258</td>
<td>0.0258</td>
</tr>
</tbody>
</table>

Table 5-1: Results of Compression Tests of Cylinders
The values for strains in this chart were obtained by trying to correct for the seating problem at the beginning of the curve. Because the stress-strain curves were linear after the initial seating problem, a line was drawn connecting the linear part of the graph down to the zero stress line, using the slope from the linear part of the graph.

The non-wrapped cylinders had similar failure modes. They approached their peak value, showed slight nonlinearity, and then failed. The failure was quite catastrophic for most of the specimens that were not fiber-reinforced. Most of the fiber-reinforced specimens showed some post-peak ductility, though, with more ductility found in the normal strength mixes than in the high strength mixes. The wrapped composites, however, showed quite different behavior. The concrete followed the unconfined stress-strain curve until loads reached just above the normal concrete failure stress. Thereafter, additional load could still be added, albeit with more deformation per additional load for the NSC specimens, until some of the fibers snapped. The load would decrease slightly, then increase even more compliantly until many fibers simultaneously failed and the load would drop dramatically (Figures 5-1, 5-2). Because there could be several load peaks for each wrapped cylinder, post-peak ductility is hard to characterize for these specimens. There was great post-peak ductility after the first fibers snapped, but then there was catastrophic failure. The concrete was often crushed into dust inside the broken wrap. Thus, the wrap is highly effective at improving strength and ductility, but provides no load-carrying capability after fracture. In practice, either deformation or initial failure of fibers could be used as an identifier of approaching failure.
Figure 5-1: Typical Confined NSC Stress-Strain Curve
Figure 5-2: Typical Confined HSC Stress-Strain Curve
The strain in the system was measured via two LVDT transducers attached to a yoke, as shown in Figure 3-11. Above and below the concrete, a steel cylinder of diameter 2.8” was placed to transfer the load to the concrete from the testing machine. A groove was cut into these cylinders, 0.25” from the face touching the concrete. The yoke was placed there, thus measuring the strain in the concrete, as well as the 0.5” of steel in between the yoke and the concrete. Thus, to get the graphs and strain values, the strain contribution of the steel needed to be subtracted out. As mentioned above, there still may have been some roughness on the top of the specimens after being cut with the diamond-tipped saw, thus accounting for part of the seating error in the graphs. Figure 3-10 shows the setup of the saw.

5.2 Effect of Fibers on Cylinders

The addition of steel fibers into the mix had an effect on the performance of the concrete cylinders. When no wrapping was provided, the fibers served to significantly increase the maximum load that each of the cylinders could resist before failure. Surprisingly, fibers also slightly decreased the ultimate strain of most of the specimens. While fibers did not improve the pre-peak ductility of unconfined concrete under compression, they did improve the post-peak ductility of the cylinders.

The wrapped cylinders behaved differently from the unwrapped ones. Except for the NSC-S4 specimens, all the specimens with fiber reinforcement showed an increase in strength of the cylinders. However, the ultimate strain of all the wrapped specimens was approximately the same for specimens of the same type of concrete. The final fracture is dependent on the wrap reaching its failure strain, thus failing the enclosed concrete. The strength increases in the fiber-
reinforced wrapped cylinders were not as stark as those in the unwrapped cylinders. Thus, fibers improve the compressive performance of the unwrapped specimens more than they improve the performance of wrapped specimens. The higher concrete strength with fiber-reinforced concrete is the main factor in the increase in strength of the wrapped specimens, rather than a beneficial combination of fiber reinforcement and composite wrapping. Table 5-2 summarizes the increases in strength due to fiber reinforcement. In the table, % increase refers to the % increase in strength/strain due to the addition of fibers.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Unwrapped Cylinders</th>
<th>Wrapped Cylinders</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Ultimate Stress</td>
<td>% Increase</td>
</tr>
<tr>
<td>NSC-00</td>
<td>5,070</td>
<td>-</td>
</tr>
<tr>
<td>NSC-S4</td>
<td>6,400</td>
<td>26.2%</td>
</tr>
<tr>
<td>NSC-S8</td>
<td>6,735</td>
<td>32.8%</td>
</tr>
<tr>
<td>HSC-00</td>
<td>7,195</td>
<td>-</td>
</tr>
<tr>
<td>HSC-S4</td>
<td>10,035</td>
<td>39.5%</td>
</tr>
<tr>
<td>HSC-S8</td>
<td>11,000</td>
<td>52.9%</td>
</tr>
</tbody>
</table>

Table 5-2: Effect of Fibers on Unwrapped and Wrapped Cylinders

5.3 Performance of HSC vs. NSC

As shown in Table 5-3, HSC specimens are quite different in behavior from NSC specimens. Under confinement, the NSC specimens were strengthened more than the HSC ones were. The wrapped NSC specimens reached 161-268% of the failure strength of the unwrapped specimens, corresponding to actual increases in ultimate stress of 4,000-11,000 psi. The NSC-S4 specimens improved in strength much less than the other NSC specimens, as a result of air gaps in the wrapping from improper wrapping during the first batch. The wrap on the NSC-S4 specimens
did not break, even though the concrete inside was crushed. However, the HSC specimens less than doubled in strength, though they increased in strength from 9,000-10,000 psi in strength. The non-fiber-reinforced specimen increased more than the other ones did. The major difference, though, was in the effect that the wrap had on the failure strain of the specimens. The wrapped NSC specimens reached strains over four times as large as their unwrapped counterparts before failing. Meanwhile, the wrapped HSC specimens only reached twice the ultimate strain as the unconfined ones. The larger increase in NSC specimens may be due to the extremely brittle nature of HSC specimens. It also may be a result of the compaction that occurs in the NSC that cannot occur with the HSC specimens. The failure of each of the specimens was due to the composite wrap failing well after the concrete had cracked. Failure may occur as a result of one of several factors. The wrap could fail in a uniform manner, or it could be subject to a stress concentration. Because the concrete inside is cracked, there may be one point where an edge is being pressed against the wrap. There also may be a weaker bond between the wrap and the concrete at a point that would cause a variation in stresses around the cylinder. This could lead to failure earlier than the material would be expected to fail. It is possible that the wrap approached its failure strain when it wrapped the NSC cylinders which is why it failed at such a higher strain but lower stress than when used in conjunction with HSC cylinders.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>NSC Stress</th>
<th>% Increase</th>
<th>NSC Strain</th>
<th>% Increase</th>
<th>HSC Stress</th>
<th>% Increase</th>
<th>HSC Strain</th>
<th>% Increase</th>
</tr>
</thead>
<tbody>
<tr>
<td>00-N</td>
<td>5,070</td>
<td>-</td>
<td>0.0087</td>
<td>-</td>
<td>7,195</td>
<td>-</td>
<td>0.0129</td>
<td>-</td>
</tr>
<tr>
<td>00-W</td>
<td>13,605</td>
<td>168%</td>
<td>0.0383</td>
<td>340%</td>
<td>17,371</td>
<td>141%</td>
<td>0.0259</td>
<td>101%</td>
</tr>
<tr>
<td>S4-N</td>
<td>6,400</td>
<td>-</td>
<td>0.01055</td>
<td>-</td>
<td>10,035</td>
<td>-</td>
<td>0.0121</td>
<td>-</td>
</tr>
<tr>
<td>S4-W</td>
<td>10,320</td>
<td>61%</td>
<td>0.0321</td>
<td>204%</td>
<td>19,585</td>
<td>95%</td>
<td>0.0257</td>
<td>112%</td>
</tr>
<tr>
<td>S8-N</td>
<td>6,735</td>
<td>-</td>
<td>0.0086</td>
<td>-</td>
<td>11,000</td>
<td>-</td>
<td>0.0134</td>
<td>-</td>
</tr>
<tr>
<td>S8-W</td>
<td>17,820</td>
<td>165%</td>
<td>0.0376</td>
<td>337%</td>
<td>19,955</td>
<td>81%</td>
<td>0.0260</td>
<td>94%</td>
</tr>
</tbody>
</table>

Table 5-3: Effect of Confinement on Ultimate Stress and Strain of NSC and HSC Specimens
A major difference between the behavior of composite-wrapped NSC and HSC specimens was the stress-strain relationship after the unconfined concrete failure load was reached. The stress-strain curve of the NSC specimens became less steep, meaning there was more deformation per increase in stress, as shown in Figure 5-1. In contrast, the stress-strain curve of the confined HSC specimens remained linear until it reached a peak of perfectly plastic deformation, as shown in Figure 5-2. Further research should be conducted to determine if this always happens for confined HSC specimens and, if so, why this occurs.

5.4 Conclusions

The use of a composite wrap on concrete cylinders has great potential benefit for use in practice. Composite wraps are easier to apply than steel ones, and require no mechanical fastening to the concrete. Care must be taken, though, to ensure a tight fit around the entire column. The composite wraps also resist much more load until they fracture than steel does until yield, so the concrete can reach a higher failure strength. The TYFO-S wrap increased the strength of specimens 61-168%, with only two wraps. More impressively, the failure strain of the concrete was increased 94-340%, for a maximum increase of over 3 times the unwrapped failure strain. While the addition of fibers seemed to affect the unconfined specimens, the confined ones were largely unaffected. HSC specimens remained linear far past the unconfined concrete failure load, unlike the NSC specimens. However, HSC showed lower percentage improvement under the wrapping than NSC did. There are valid reasons for this. One could be the more brittle and compact nature of HSC. Another may be the failure stress of the HSC already being higher so
only two wraps of confinement may not be enough. Further research should be conducted on the
effect of more layers of wrapping around HSC and NSC cylinders.

In practice, composite wrap should have many uses with the deterioration of columns in the
infrastructure. The TYFO-S wraps are easy to apply and should not be subject to weathering.
Research should also be done to determine the long term effect of weathering on the composite
wraps, as well as on the fatigue loading of wraps, to better understand the behavior of the wraps
in practice. Finally, the long term bond strength of the epoxy should be studied to insure the
wrap will remain structurally integral with the column.
Chapter 6

Conclusions

Composites have a bright future ahead of them in civil engineering. Because retrofitting of structures is often a more viable option than rebuilding, composite laminates and column wraps should play a vital role in repair. Composites should not deteriorate as much as steel does in foreign environments, and they possess a higher tensile strength than ordinary steel does.

Composites should also play a large role in new construction. Because steel corrodes under freeze/thaw action and chloride ion impregnation, it expands and spalls the surrounding concrete. If FRP rebar can be used instead of steel, weathering may become less of a problem.

The FRP rebar performed well as flexural reinforcement for concrete. While its strength is greater than steel’s, its bond strength is lower than that of steel, and its stiffness is only one quarter that of steel. The major drawback of using FRP rebar is that there is more cracking in FRP-reinforced beams than in steel-reinforced beams.

In the shear tests performed in this research project, the beams reinforced with FRP bars did not perform as well as those reinforced with regular steel rebar. However, they did perform reasonably well. Also, when fibers were included, there was a larger increase in strength for the FRP-reinforced beams than for those with steel rebar. This is likely due to the additional control of cracking by the fibers. Steel fibers, included at 0.4 and 0.8% by volume, provided some
ductility and proved to be very useful for improving the shear strength of beams not reinforced with stirrups. The inclusion of 0.05 and 0.1% volume fractions of polypropylene fibers decreased the ductility of the beams, while increasing strength. The fibers were also very difficult to mix into the concrete well. They provided an excellent increase in strength for the volume of fibers used, but the sudden and catastrophic failure in the beams could be very dangerous in practice if they are the only shear reinforcement provided.

Because of the success of FRP rebar as flexural reinforcement, FRP shear stirrups should also be tested. Though some companies manufacture them by bending hardened rods, which would decrease the rod strength by over 50% for even small bends, Marshall makes their C-bar shear stirrups by setting them in the bent position. This should improve their effectiveness over other FRP shear stirrups. Because FRP stirrups are not as stiff as steel stirrups, and FRP does not bond as well to concrete as steel does, more deformation will be required for them to reach full shear capacity, testing should be done to determine how effective FRPs may be as shear reinforcement. The bars may have to be coated in sand or grooved more to prove effective as crack arresters. Research should be conducted on these shear stirrups to determine how well they would work with FRP rebar, and also with steel fibers.

The use of HSC improved the shear behavior of all the beams tested. Obviously, the failure strength was higher for the HSC beams, but the main contribution HSC made was the better grip of the fiber reinforcement. This increased shear strength of the beams. Though the FRP reinforced beams cracked much more than the beams reinforced with steel rebar, cracking was
much less prevalent in HSC bars. In all cases, the failure of HSC beams tended to be more catastrophic, even for the beams containing steel fibers.

The TYFO-S composite wrap, with a failure stress of 65 ksi, proved extremely effective in improving the behavior of concrete cylinders. It increased the load that specimens could take to up to nearly 270% of the specimen’s initial compressive strength, for concrete of a compressive strength of approximately 6,000 psi. More impressively, the ultimate strain of the specimens before failure was increased over 400% for some types of concrete. Thus, the composite wrap showed tremendous potential for use in the field.

A comparison of the concrete strain and the wrap strain should be conducted in future research, as should a test involving more wraps around the cylinder. Although the wrap managed to increase the HSC strength more than it did for NSC specimens, the HSC specimens did not reach the same strain as did the NSC ones after being confined. Interestingly, the stress-strain curves of the HSC specimens remained linear until the ultimate stress was reached, as opposed to the NSC specimens where the curve bent over from its initial slope to a shallower one after the failure stress of the unconfined concrete was reached.

Fiber reinforcement proved effective in improving the ductility of the unconfined cylinders, as well as the aforementioned beams, but once the cylinders were wrapped the fibers did not improve the ductility of the composite column, as the failure of the fiber wrap was the cause of failure. Thus, the fibers did not prove to have a significant impact on the wrapped specimens, though they did for the beams.
As composites are used in more applications in civil engineering in the future, the safety of structures reinforced with these materials must be assured. Building and material codes must start to include guidelines for use of these materials, and standards for field use developed. Because these materials are proving themselves useful and successful at the laboratory test level, their use should become more widespread. More tests need to be conducted, but composites have thus far proven that they are worthy of consideration as a prime reinforcing material for the civil engineer.
Bibliography


[19] American Concrete Institute, Committee 363, State of the Art Report on High-Strength Concrete. ACI 363R-84


Appendix A:
Load-Deflection Plots of All Beams
HSC-S8-C2

Displacement (inches)

Load (pounds) Thousands
HSC-S8-C4

![Graph showing the relationship between load (pounds) and displacement (inches). The graph has a peak at a load of thousands and a displacement of 0.1 inches.](image-url)
HSC–P5–C2

Load (pounds)

50
40
30
20
10
0

Displacement (inches)

50
40
30
20
10
0

0.1
0.2
0.3
0.4
0.5
0.6
0.7
NSC-S4-S1

Load (pounds) Thousands

Displacement (inches)

0 0.1 0.2 0.3 0.4 0.5 0.6 0.7

0 10 20 30 40 50
HSC-S4-S2

![Graph showing load plotted against displacement.](image)
Appendix B:
Stress-Strain Curve of All Cylinders
NSC-00-W2

Stress ( Ksi ) Thousands

0  10  20  30

Strain

0  0.01  0.02  0.03  0.04  0.05  0.06
NSC-S4-W2

Stress (psi Thousands)

Strain

0 0.01 0.02 0.03 0.04 0.05 0.06
NSC-S8-W1

Stress (Psi) 
Thousands

Strain

0 0.01 0.02 0.03 0.04 0.05 0.06
Appendix C: Pictures of All Tested Beams
Figure C-1: NSC-00-C1

Figure C-2: NSC-00-C2
Figure C-11: HSC-00-C1

Figure C-12: HSC-00-C2
Figure C-17: HSC-S8-C3

Figure C-18: HSC-S8-C4
Figure C-33: HSC-S8-S1

Figure C-34: HSC-S8-S2
Appendix D:
Pictures of All Confined Cylinders
Figure D-1: Confined NSC-00 Specimens (Front View)

Figure D-2: Confined NSC-00 Specimens (Top View)
Figure D-3: Confined NSC-S4 Specimens (Front View)

Figure D-4: Confined NSC-S4 Specimens (Top View)
Figure D-5: Confined NSC-S8 Specimens (Front View)

Figure D-6: Confined NSC-S8 Specimens (Top View)
Figure D-7: Confined HSC-00 Specimens (Front View)

Figure D-8: Confined HSC-00 Specimens (Top View)
Figure D-9: Confined HSC-S4 Specimens (Front View)

Figure D-10: Confined HSC-S4 Specimens (Top View)
Figure D-11: Confined HSC-S8 Specimens (Front View)

Figure D-12: Confined HSC-S8 Specimens (Top View)