CREATION OF AN ODOT SPECIFICATION FOR PATCHING OR OVERLAY OF BRIDGE DECKS

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16 . ABSTRACT

Bridge deck cracking is a huge problem in the United States, and various agencies have sponsored research endeavoring to determine the underlying problems. A number of causes have been identified, including thermal movement, plastic shrinkage, and early age settlement, as well as a number of other issues. Polymer fibers are a possible solution to many of the causes of bridge deck cracking: they have been shown to help early age properties like shrinkage and movement, and as a bonus, fibers improve post-cracking behavior. More understanding of the benefits and uses of polymer fibers in concrete is needed.

This study researched the properties of four polymer fibers; two of the fibers were macrofibers, and two were microfibers. Each fiber was tested at several dosage rates to identify optimum dosage levels. Early age shrinkage, long term shrinkage, compressive strength, and tensile strength were investigated.

Macrofibers and microfibers were found to have different impacts on concrete behavior, with different optimal dosage rates. Microfibers greatly dried out the concrete mixture, hindering workability. However, the microfibers substantially reduced plastic shrinkage and improved concrete strength at early age. Macrofibers, while not hindering workability, did not provide benefits as great as the microfibers to the concrete strength.

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SI (METRIC) CONVERSION FACTORS

Approximate Conversions to SI Units

Symbol	When you know	Multiply by	To Find	Symbol
LENGTH				
in	inches	25.40	millimeters	mm
ft	feet	0.3048	meters	m
yd	yards	0.9144	meters	m
mi	miles	1.609	kilometers	km
AREA				
in^2	square inches	645.2	square millimeters	mm
ft^2	square feet	0.0929	square meters	m^2
yd^2	square yards	0.8361	square meters	m^2
ac	acres	0.4047	hectares	ha
mi^2	square miles	2.590	square kilometers	km^2
VOLUME				
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.0283	cubic meters	m^3
yd^3	cubic yards	0.7645	cubic meters	m^3
MASS				
oz	ounces	28.35	grams	g
lb	pounds	0.4536	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
TEMPERATURE (exact)				
°F	degrees Fahrenheit	(°F-32)/1.8	degrees Celsius	°C
FORCE and PRESSURE or STRESS				
lbf	poundforce	4.448	Newtons	N
lbf/in^2	poundforce per square inch	6.895	kilopascals	kPa

Approximate Conversions from SI Units

Symbol	When you know	Multiply by	To Find	Symbol
LENGTH				
mm	millimeters	0.0394	inches	in
m	meters	3.281	feet	ft
m	meters	1.094	yards	yd
km	kilometers	0.6214	miles	mi
AREA				
mm^2	square millimeters	0.00155	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.196	square yards	yd^2
ha	hectares	2.471	acres	ac
km^2	square kilometers	0.3861	square miles	mi^2
VOLUME				
mL	milliliters	0.0338	fluid ounces	fl oz
L	liters	0.2642	gallons	gal
m^3	cubic meters	35.315	cubic feet	ft^3
m^3	cubic meters	1.308	cubic yards	yd^3
MASS				
g	grams	0.0353	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams	1.1023	short tons (2000 lb)	T
TEMPERATURE (exact)				
°C	degrees Celsius	9/5+32	degrees Fahrenheit	°F
FORCE and PRESSURE or STRESS				
N	Newtons	0.2248	poundforce	lbf
kPa	kilopascals	0.1450	poundforce per square inch	lbf/in^2

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Abstract

Bridge deck cracking is a huge problem in the United States, and various agencies have sponsored research endeavoring to determine the underlying problems. A number of causes have been identified, including thermal movement, plastic shrinkage, and early age settlement, as well as a number of other issues. Polymer fibers are a possible solution to many of the causes of bridge deck cracking: they have been shown to help early age properties like shrinkage and movement, and as a bonus, fibers improve post-cracking behavior. More understanding of the benefits and uses of polymer fibers in concrete is needed.

This study researched the properties of four polymer fibers; two of the fibers were macrofibers, and two were microfibers. Each fiber was tested at several dosage rates to identify optimum dosage levels. Early age shrinkage, long term shrinkage, compressive strength, and tensile strength were investigated.

Macrofibers and microfibers were found to have different impacts on concrete behavior, with different optimal dosage rates. Microfibers greatly dried out the concrete mixture, hindering workability. However, the microfibers substantially reduced plastic shrinkage and improved concrete strength at early age. Macrofibers, while not hindering workability, did not provide benefits as great as the microfibers to the concrete strength.

In general, several key results were identified, and it is suggested that many of these impacts can be explained by considering that the polymer fibers have a modulus of elasticity well below that of the hardened concrete matrix. Fibers were found to greatly reduce early age shrinkage, with the effect increasing with increasing dosage levels. Long term shrinkage is not affected by the addition of polymer fibers. Early age concrete strength is improved with the addition of fibers, but long term strength is sometimes reduced with high dosages of fibers. It is noted that these characteristics of polymer fibers indicate that they will be very useful in combating the bridge deck cracking problem.

Chapter 1: Introduction

Bridge decks have many problems with cracking. More than 100,000 bridge decks, nearly half of the bridges in the United States, showed transverse cracking at early age (Krauss and Rogalla, 1996). Early age cracking is the most common deck distress reported by the State Highway Agencies. In all , 97% of state Departments of Transportation indicated that they have problems with early age cracking (Aktan et al., 2003).

Numerous studies have been performed on these problems, and several of the primary causes have been isolated. These include thermal movement, early age shrinkage, and early age settlement (Krauss and Rogalla, 1996; Babaei, 2005). These causes may all be counteracted by the addition of polymer fibers. Polymer fibers have been shown to be beneficial to the early age properties of concrete, as well as to crack mitigation (Kao, 2005).

Research presented here analyzes a number of fibers and dosage rates for their strength and shrinkage properties. Four types of fibers are tested; each one is tested at three to five different dosage rates. The results indicate that long term strength is not strongly impacted by polymer fiber addition, but early age shrinkage is greatly decreased and early age strength is increased.

Chapter 2: Literature Review

There has been considerable research work done on both ends of the field: bridge deck cracking and fiber reinforcement. General reviews of the bridge deck cracking problem have been conducted by the National Cooperative Highway Research Program (NCHRP) and several Departments of Transportation (DOT's). These reviews analyze the problems statistically, and provide a summary of many variables important to the problem. Fiber reinforcement has typically been regarded as a simple crack reducer, but there is research investigating many aspects of its impact on material properties. Fibers impact the bridge deck cracking problem on several fronts, not simply by bridging cracks. A review of research done on both bridge deck cracking and fiber-reinforced concrete is presented here.

2.1 Bridge Deck Cracking

Bridge deck cracking is a problem throughout the United States, as several surveys indicate. A number of state departments of transportation, including Michigan, Texas, Oregon, Utah, New Jersey, Minnesota and Colorado have launched studies on the problem (Brooks, 2000; Brown et al., 2001; Xi et al., 2003; Aktan et al., 2003; Linford and Reaveley, 2004), and in 1996 NCHRP conducted a major project entitled "Transverse Cracking in Newly Constructed Bridge Decks". This project, undertaken by Krauss and Rogalla, was a comprehensive analysis of the cracking problem at that point, and set out the problems in great detail. Since then a number of projects have conducted research according to the recommendations of that report. The departments of transportation performed similar analyses, researching the problem statistically through surveys, and then identifying the primary causes of cracking. Applicable laboratory research and extensive field studies on new bridges were done to test various methods of mitigating the problem.

An interesting aspect of the present cracking problem is that it has increased as the strength of the concretes used has increased. This may indicate that something about the newer high-performance concretes encourages cracking, unless some other variable such as workmanship or curing is becoming worse during the same period of time. This literature review will investigate why that may be, and what to do about it (Xi et al., 2003).

2.1.1 Scope of the Problem

A large proportion of the bridges in the United States crack at early age. Aktan et al. (2003) found that early age cracking is the single most prevalent deck distress reported by the State Highway Agencies. More than 100,000 bridge decks in the United States showed transverse cracking at early age, according to Krauss and Rogalla (1996); this is nearly half of the bridges. Their survey included 52 DOT's in the United States and Canada. Sixty-two percent of these agencies considered transverse cracking a problem; fifteen percent believed that all of their bridges suffered from transverse cracking. The respondents stated that, on average, forty-two percent of bridge decks cracked in the first week.

In the report for the Utah Department of Transportation (Linford and Reaveley, 2004) a database of 71 newly-constructed bridges in the I-15 reconstruction project was created. The bridges were constructed between April 1998 and March 2001. The bridges were each ranked with a Cracking Severity Index Number (CSIN). Cracking was found on 70 of the 71 bridges. Diagonal cracking was found on 87% of the bridges, primarily near abutments or interior bents. Transverse cracking was found on 67% of the bridges; according to the report, this, they postulate, was caused by concrete shrinkage. Only 11% have visible longitudinal cracks.

The report for the Colorado Department of Transportation (Xi et al., 2003) analyzed 72 structures built between 1993 and 2000. These were inspected in 2002. At that time, 82% of the bridges had deck cracking. In addition, the report declared that the Nevada Department of Transportation stated that 75% of all new bridges have a significant cracking problem. The Kansas Department of Transportation indicated that their cracking problems have been mostly resolved; they attribute their success to the implementation of a wet burlap 7 day curing procedure, which cut deck cracking by 50%.

Michigan conducted a survey of the state Departments of Transportation in 2002 (Aktan et al., 2003). Thirty-one states responded. Of these, 97% indicated that they had an early age cracking problem in reinforced concrete bridge decks. Nearly all of those first observed bridge deck cracking within the first year, and most within the first few months. Seventy-eight percent of the respondents stated that transverse cracking was the most prevalent, with 16% citing longitudinal cracks, and 6% diagonal.

2.1.2 Mechanics of Cracking

Krauss and Rogalla (1996) carefully considered the mechanics of the cracking problem in their report on transverse cracking in bridge decks. Concrete bridge decks develop cracks when the tensile stress in the concrete exceeds the tensile strength of the concrete at that time. The tensile stresses come from concrete shrinkage, temperature changes in the concrete, and sometimes from self-weight or traffic loads. The stresses develop in the bridge decks because the girders restrain the natural thermal and shrinkage movement of the deck, thus translating the strain into stress.

Brown et al. (2001) endeavored to further isolate the mechanical causes of bridge deck cracking. Figure 1 shows the flow chart they created showing the primary factors in the cracking problem.

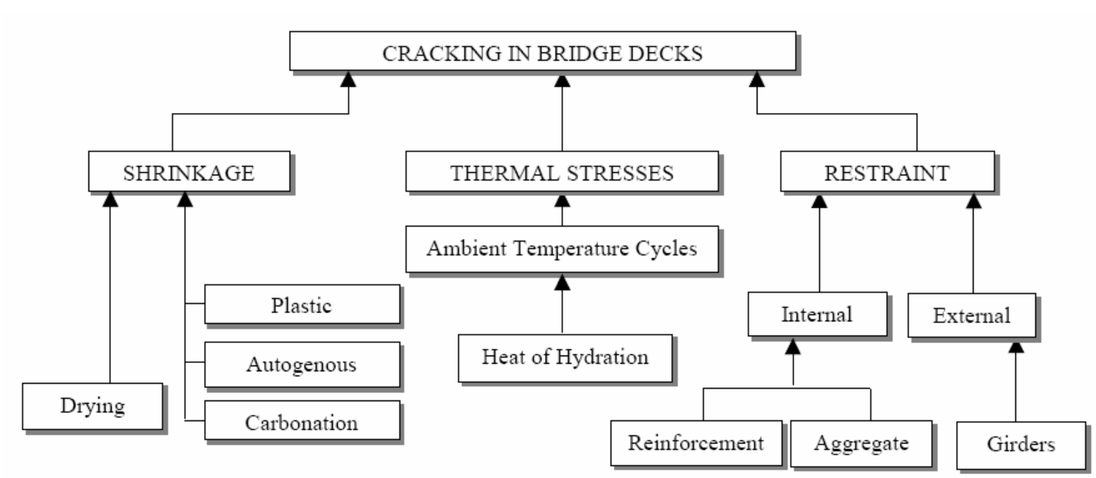

Figure 1: Causes of bridge deck cracking (Brown, et al., 2001)

As seen in this figure, Brown et al. consider shrinkage, thermal stresses, and restraint to be the primary factors in cracking. Later in this literature review, each of these factors will be considered in greater detail.

Shrinkage of concrete is a primary source of strain in bridge decks, and can produce enough strain to crack concrete without additional strain from temperature sources (Krauss and Rogalla, 1996). It is considered by many to be the greatest culprit in the cracking problem (Krauss and Rogalla, 1996).

Temperature effects are the other important source of strain in the concrete matrix. The concrete sets at a specific temperature, locking the matrix to zero temperature stress at that temperature. However, the deck changes temperature, seasonally, daily, from cooling off after the heat of hydration subsides, and from solar radiation on the top surface. These four sources cause significant temperature movement, which occurs according to the coefficient of thermal expansion of the concrete. The stresses induced can both be high and significantly non-uniform (Krauss and Rogalla, 1996).

The final sources of strain are the dead loads and live loads on the structure, along with formwork deflection issues. These strains are less significant, but of concern nonetheless. Several state departments of transportation considered these to be a source of cracking (Krauss and Rogalla, 1996).

In an unrestrained system, strain does not cause cracking, but when the system is restrained, the strain translates to stress and causes cracking. The restraint of the deck's movement converts the strain into stress, according to the modulus of elasticity. Both external and internal sources can provide the restraint. The chief external source is the girders that the deck rests upon. Since the girders will not shrink at the same rate as the deck unless they are cast at the same time of the same material, the girders restrain the deck's movement. In addition, material differences can cause differential restraint of temperature movements. Internally, rebar, aggregate, and fibers are some of the sources of restraint (Krauss and Rogalla, 1996).

There are several other factors that influence the mechanical cracking problem. Stress relaxation or "creep" of concrete is another key issue, as it is the one factor that can reduce the stresses on the concrete. Altoubat and Lange (2002) analyzed this factor in considerable detail. They found that creep can reduce shrinkage stresses by 50% (depending on the mix design), thus doubling the strain capacity at failure.

Krauss and Rogalla (1996) consider the modulus of elasticity to be another important factor in the cracking problem. The modulus of elasticity of the concrete determines the rate of conversion from strain to stress. Therefore, the stress in the concrete will be higher with a higher modulus of elasticity given the same strain conditions.

The geometry of the bridge deck and girders can also have significant impacts on the cracking behavior of the concrete. Krauss and Rogalla (1996) analyzed different designs analytically and found that the geometry of the deck significantly impacted the shrinkage and thermal strain fields.

Corrosion of reinforcing steel is a well known factor; however, it typically does not become important for several years. Since the present cracking problems usually show up within a year, the corrosion issue will only be considered in passing.

The final factor in the cracking process is the tensile strength of the concrete itself. After the stresses are created by the factors mentioned previously, whether the concrete finally cracks or not is determined by comparing the stress to the tensile strength of the concrete. As shown in Figure 2, both the stress and the tensile strength of the concrete change with time and it is when the stress finally exceeds the tensile strength of the concrete that cracking occurs (Brown et al., 2001 after Mehta, 1993).

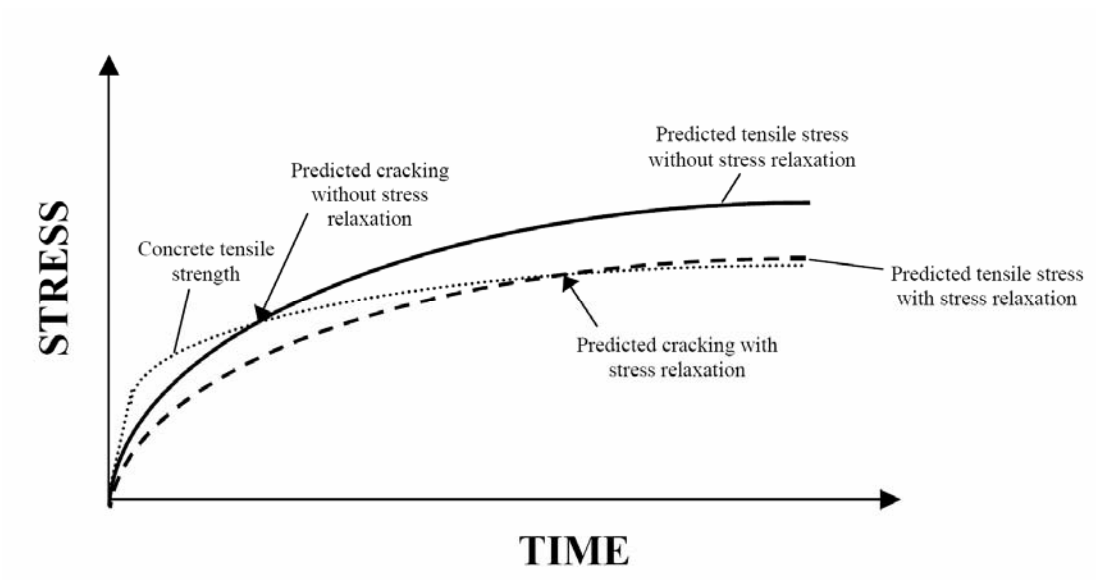

Figure 2: Time dependence of restrained shrinkage and creep (Brown et al., 2001 after Mehta, 1993)

In summary, the literature indicates that the mechanical process that creates the cracking is as follows. Shrinkage and thermal movement, along with deflections to some extent, put a strain on the deck. This strain would cause no stress if it was unrestrained, but restraint is provided both by the girders and by the reinforcement. This restraint converts some 80% of the strain to stress, depending on the degree of restraint. The actual conversion rate is the modulus of elasticity of the concrete. The creep of the concrete reduces stress by a significant but hard to quantify amount. This stress field is modified by the geometry of the deck, and finally the stress and the tensile strength of the concrete may be compared to see whether cracking is likely to occur. This view of the cracking problem, while probably somewhat simplistic in some areas, gives a reasonable picture of the issues involved in cracking of bridge decks (Krauss and Rogalla, 1996).

Here is a simple example of the mechanics in action, from Krauss and Rogalla (1996):

> …If the concrete has a free-shrinkage of 500 microstrain (με), but it is restrained and allowed to shorten only 250 με, the restraint is 50 percent. A concrete with a modulus of elasticity of 4×10^6 psi might have an effective modulus of only 2×10^6 psi, because of creep. The resultant stress would be the product of the strain (500 με) times the restraint (50 percent) times the effective modulus of elasticity (2×10^6 psi) for a resultant tensile stress of 500 psi. If the tensile strength of

> the concrete is greater than 500 psi, cracking will not occur. However, additional tensile stresses from thermal gradients or loading could crack such a concrete. Therefore, effects of shrinkage and temperature changes, effect concrete modulus, restraint conditions, tensile strength, and loading conditions must be considered. (Krauss and Rogalla, 1996)

Figure 3 shows the factors affecting cracking in bridge decks that are covered in this literature review.

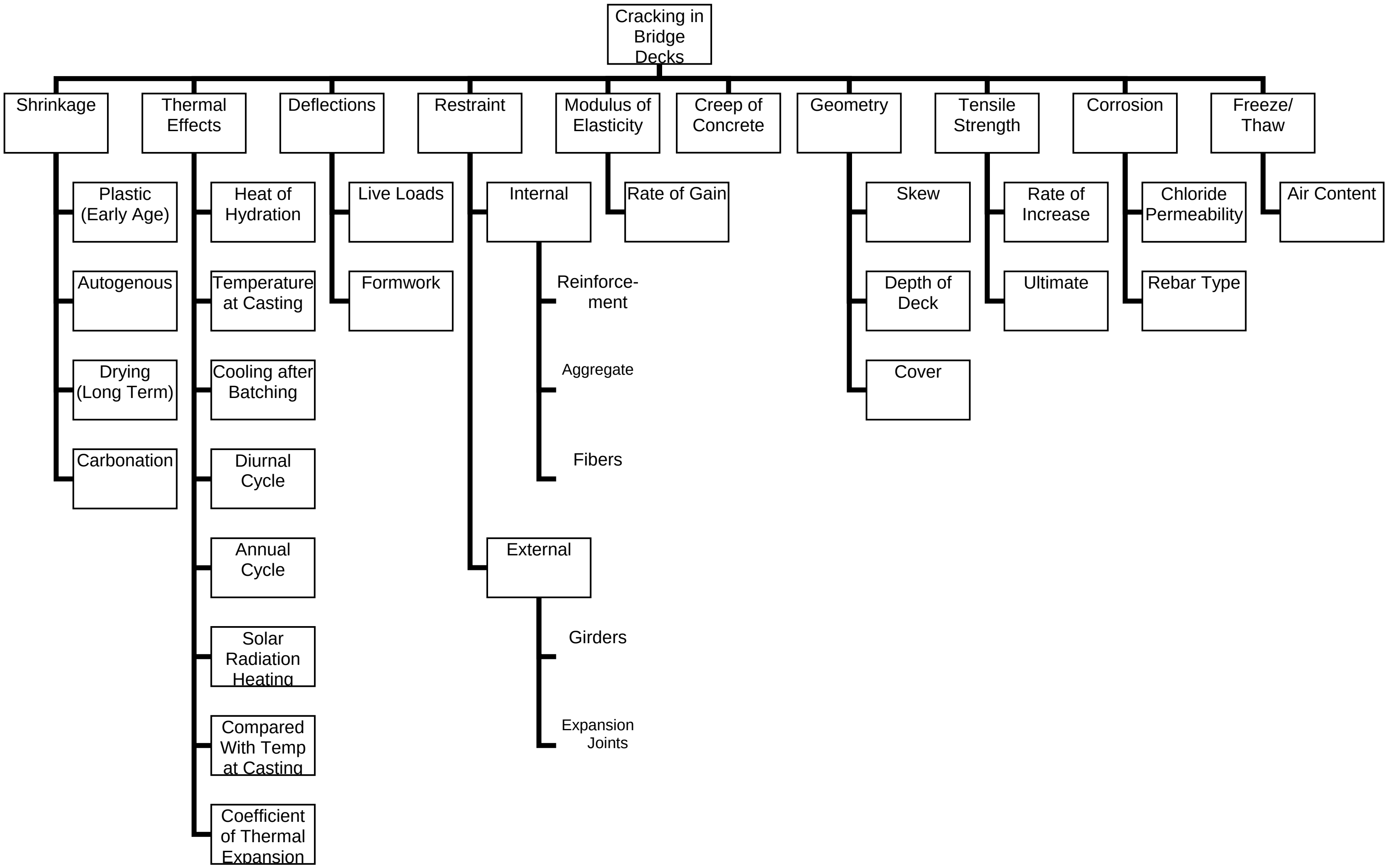

Figure 3: Factors affecting cracking in bridge decks

2.1.3 Shrinkage

Shrinkage is thought to be one of the greatest causes of cracking in bridge decks (Krauss and Rogalla, 1996; Phillips et al., 1997). Restrained shrinkage alone can create tensile stresses sufficient to crack the deck. If the deck shrinks 500 microstrain, the deck can easily see tensile stresses exceeding 1000 psi, depending on the material properties and geometric constraints (Krauss and Rogalla, 1996).

There are four types of shrinkage of note. Plastic shrinkage occurs at early age, before the concrete has hardened. This type of shrinkage typically occurs because of poor curing conditions leading to evaporation of water and hence high capillary stresses. Autogenous shrinkage is based on the loss of water due to chemical consumption in the setting chemical reactions, and potentially the actual formation of the crystal structure. Drying shrinkage is the primary long-term shrinkage type, again based upon water loss. Carbonation shrinkage is a long-term shrinkage that occurs when there is a high CO_2 concentration in the air around the concrete.

It must be noted that shrinkage as a whole is not well understood. The types of shrinkage can be isolated by using specific tests, but the actual mechanisms by which these shrinkage types proceed are open to argument.

2.1.3.1 Plastic (Early Age) Shrinkage

Plastic shrinkage occurs at early age. It is listed by Issa (1999) as the most important cause of bridge deck cracking. Plastic shrinkage depends on two primary factors: the rate at which surface water forms (bleeding) and the evaporation rate of the surface water (Wang et al., 2001). When the evaporation rate from the top surface of the concrete exceeds the bleed rate at which water rises from the concrete, the top surface dries out. At this point, the free water surface in the concrete drops within the concrete, yielding menisci between the particles. These menisci exert a tensile force due to surface tension on the particles, a suction of sorts. This and a low concrete strength due to top surface desiccation cause cracking (Mindess and Young, 1981; Cheng and Johnston, 1985; Holt, 2001; Brown et al., 2001). Since this type of cracking occurs because of forces near the surface of the concrete, the cracks are typically shallow in depth and originate from the top surface. These cracks, however, are sufficient to assist water and chloride penetration, and to provide stress concentration points for long-term shrinkage cracking. Plastic shrinkage does not require external restraint on the member to create stresses, as the majority of the member is not shrinking, and it is solely the surface that shrinks. Thus, the surface alone will crack. Typical cracks are no more than 2 or 3 feet long and are 2 to 3 inches deep (Xi et al., 2003, Krauss and Rogalla, 1996) and exhibit a typical "turkey track" configuration.

2.1.3.1.1 Curing conditions

Curing conditions are the overriding cause of plastic shrinkage cracking. It is the most common reason cited by transportation agencies for the transverse deck cracking (Krauss and Rogalla, 1996). Curing conditions are blamed by most departments of transportation for the early-age cracking problem. In many cases, the department of transportation's specifications on bridge deck placement and curing may be ignored, greatly intensifying the problem.

There are several procedures that are important for limiting the plastic shrinkage cracking problem, all revolving around limiting evaporation from the fresh concrete. If possible, the evaporation rate should be measured or estimated, and the evaporation rate limited to 0.20 lb./ft^2/hr for normal concrete and 0.10 lb./ft.^{2}/hr. for concrete with a low water to cement ratio (Shing and Abu Hejleh, 1999). Evaporation counter measures are almost mandatory if the evaporation rate exceeds 0.20 lb./ft.^{2}/hr, and cracking is possible even with an evaporation rate of only 0.10 lb./ft.^{2}/hr (Cheng and Johnston, 1985). Nomographs are available to calculate the evaporation rate based on environmental conditions.

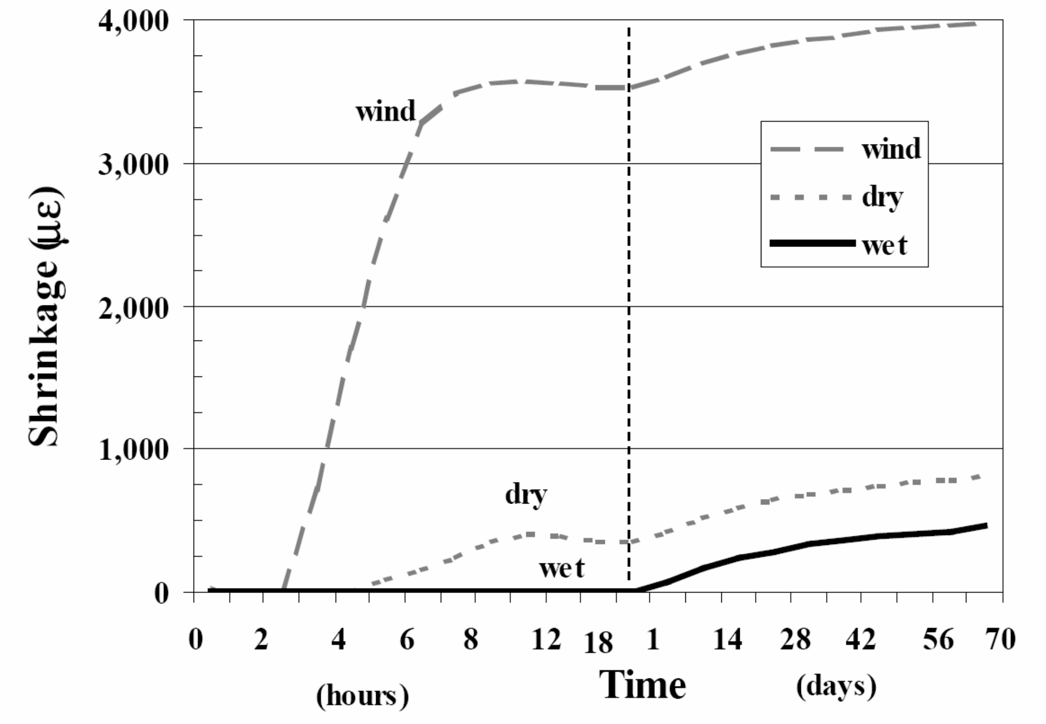

Figure 4: Accumulation of early age and long term shrinkage, with various curing environments during the first day. Wind = 2 m/s (4.5mph), dry = 40% RH, wet = 100% RH. (Holt, 2001)

Figure 4 gives shows just how significant the curing conditions are in the shrinkage of concrete. Wind can greatly increase the shrinkage of concrete, and the level of wind shown (some 4.5 miles per hour) is often found on a jobsite. Dry conditions (like 40% relative humidity) are similarly commonly found, and proper precautions must be taken to prevent the drying shrinkage shown in the figure from occurring. Interestingly, it has been shown that there is no correlation between curing conditions in the first 24 hours and shrinkage at later times; they are essentially decoupled (Holt, 2001).

Moist curing for an extended period of time is highly recommended (Mindess and Young, 1981). Using a wet burlap system has long been considered the best method, but wind and heat can dry burlap rapidly, necessitating a method for keeping the burlap moist. The moist curing must start within a few minutes of the finishing to get the best results. Fogging during the time between strike-off and the application of the burlap helps reduce early-age plastic cracking as well, and is highly recommended (Xi et al., 2003; Shing and Abu-Hejleh, 1999; Cheng and Johnston, 1985).

Curing compounds can significantly reduce the number of small deck cracks, but this method is not as good as using wet burlap for several days. The film applied is difficult to make continuous, and the moisture from the wet curing aids the strength of the very top of the concrete.

2.1.3.1.2 Consolidation

It has been shown that inadequate consolidation contributes to early age cracking, as well as other issues. Typically, the department of transportation specifications are sufficient to prevent this problem, but are not always carried out in the field.

2.1.3.1.3 Finishing Procedures

Early finishing reduces the size and number of cracks. In addition, double-floated decks seem to have less cracking. In order to allow curing to commence earlier, it is recommended to saw cut the grooving rather than use rake tining of plastic concrete. Rake tining of plastic concrete damages the surface of the hardened concrete. Hand finishing should not be allowed except at the edge of the pavement (Krauss and Rogalla, 1996; Xi et al., 2003; Shing and Abu-Hejleh, 1999).

2.1.3.1.4 Mix Design

The mix design of a concrete influences the plastic shrinkage. High water to cement ratios and high cement content increase plastic shrinkage (Aktan et al., 2003; Krauss and Rogalla, 1996). Interestingly, a high water to cement ratio would seem to lead to a higher bleed rate, which according to the accepted model of plastic shrinkage is a good thing. A lower water to cement ratio concrete would probably have its top surface dried out more readily. Early age cracking has become more prevalent as high performance concretes (with a low water to cement ratio) have become more common. Perhaps some further investigation of the relationship of water to cement ratio and plastic shrinkage is in order.

2.1.3.1.5 Admixtures

There are several admixtures that can impact the plastic shrinkage of concrete. Shrinkage reducing admixtures reduce the surface tension of the water in the capillary pores, thus reducing the stress from the pore water. This reduces the plastic shrinkage, but this mechanism also reduces air entraining, which may be problematic. Set retarders can actually increase plastic shrinkage simply by keeping the concrete plastic before setting for a longer period of time (Xi et al., 2003; ACI 212, 1989). Water reducing admixtures can help decrease the shrinkage as well by reducing the water to cement ratio.

2.1.3.1.6 Air temperature

The air temperature at batching directly influences the evaporation rate of the concrete, and thus the plastic shrinkage. It is typically recommended to batch when the air temperature is below 80° F (Xi et al., 2003; Krauss et al., 1995, Shing and Abu-Hejleh, 1999).

2.1.3.1.7 Wind

Several investigators and transportation departments consider wind to be the most significant factor affecting cracking (Krauss and Rogalla, 1996). Wind significantly increases evaporation, which is the main cause of plastic shrinkage cracking (Xi et al., 2003). Most sources recommend setting up temporary wind breaks during casting to limit evaporation until appropriate curing methods can be applied. Some curing procedures are adversely affected with wind, particularly any that have plastic sheeting placed, as the wind can blow under the plastic if the edges are improperly secured. If necessary, casting under a high wind condition should be avoided to reduce plastic shrinkage (Xi et al., 2003; Mindess and Young, 1981).

2.1.3.1.8 Humidity

Humidity decreases evaporation; to increase humidity around the concrete, foggers are often recommended. If the humidity in the air is very low, there can be high evaporation rates even without wind (Xi et al., 2003). More cracking has been observed for concrete cast during low humidities (Krauss and Rogalla, 1996).

2.1.3.1.9 Silica Fume Concrete

Silica fume increases the density of the concrete, decreasing porosity, and thereby also decreasing the bleed rate of the concrete. This inability of water to move within the mix increases the concrete's susceptibility to plastic shrinkage and plastic shrinkage cracking. It has been shown that silica fume concrete is significantly more likely to crack if improper curing procedures are followed. However, studies have also shown that if appropriate curing procedures are adhered to, the silica fume does not increase plastic shrinkage cracking (Shing and Abu-Hejleh, 1999).

2.1.3.2 Autogenous Shrinkage

Autogenous shrinkage is defined as the macroscopic volume change occurring with no moisture transferred to the exterior surrounding environment, and thus is related to the actual chemical reactions of the concrete. Autogenous shrinkage occurs even when the concrete is completely submersed in water, thus having 100% humidity on the surface. It also occurs even when the surface is made completely air and water proof with some curing agent. Thus its mechanism is not related to surface tension of water at the surface, but rather to the surface tension in pores, a reduction in relative humidity as the pore water is chemically consumed, and the actual volume change from the reactants to the products (Xi et al., 2003; Holt, 2001; Brown et al., 2001; Lura, 2003). The higher performance concretes move the reaction more in favor of lower volume products, increasing the importance of the last mechanism mentioned.

Autogenous shrinkage is usually insignificant compared with plastic and drying shrinkage, but for high-strength concretes with low water-to-cement ratios, it has been shown that autogenous shrinkage becomes important. Most research indicates strength exceeding 6000 psi and water-to-cement ratios below 0.4 are most susceptible to autogenous shrinkage (Xi et al., 2003; Holt, 2001; Brown et al., 2001; Lura, 2003).

Autogenous shrinkage is a chemical shrinkage, but not all of the chemical shrinkage translates into autogenous shrinkage, which is an external measurement. Some of the chemical shrinkage ends up as voids in the concrete, as illustrated in Figure 5.

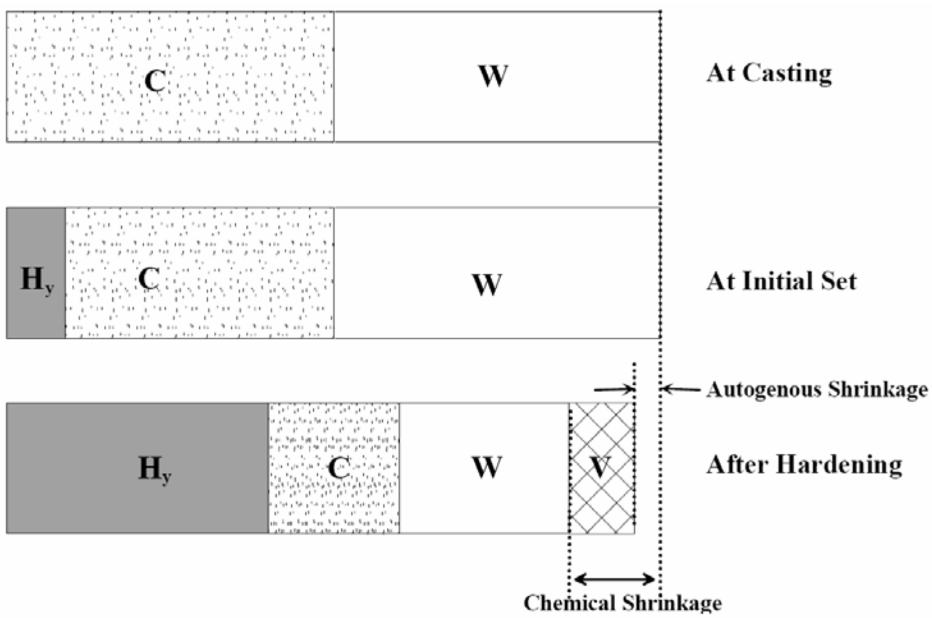

Figure 5: Reactions causing autogenous and chemical shrinkage (Holt, 2001 from Japan, 1999) C = unhydr ated cement, W = unhydr ated water , H_y = h ydration pr oducts, and V = voids generated by hydration.

The first source of the chemical shrinkage is from volume reduction of the reaction products. This is dominant at very early age, when the concrete is still liquid. At this age, the chemical and autogenous shrinkage are equivalent. In addition, because the concrete is still liquid, the shrinkage does not result in stress, as the concrete is unrestrained and simply settles.

After the skeleton of the concrete begins to be formed, there are several mechanisms in play. Figure 6 below illustrates the formation of empty pore volume due to chemical shrinkage, which results in a decrease of the radius of curvature of the menisci and in bulk shrinkage due to increased tensile stresses from the pore water. This is self desiccation shrinkage.

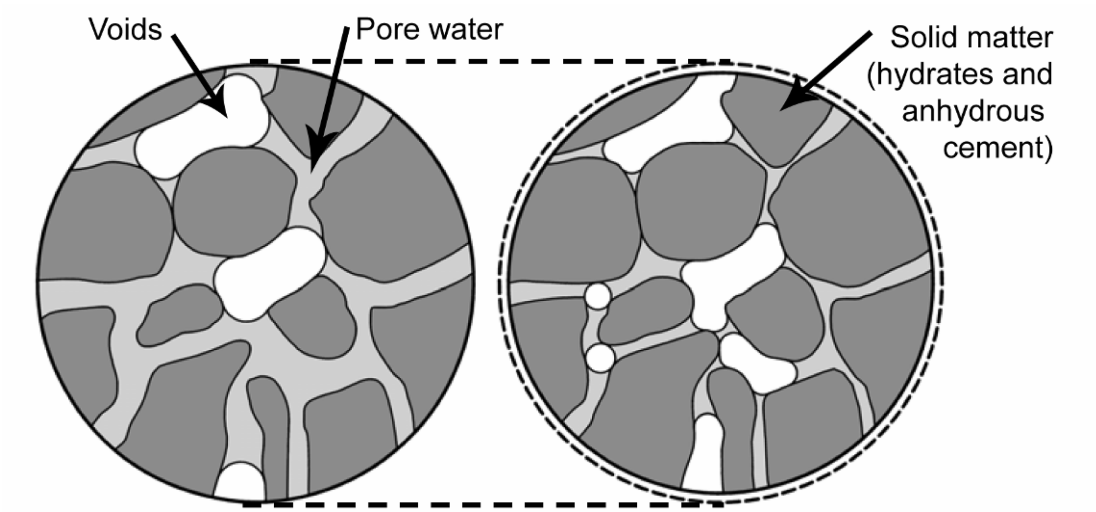

Figure 6: Schematic of a cross-section of hydrating cement paste (Jensen and Hansen, 2000). Left: low degree of hydration. Right: high degree of hydration.

Self-desiccation is the most commonly cited mechanism, where the pore water is consumed by the hydration process. As the pores dry, the water menisci in the pores produce significant suction forces on the crystalline structure. Chemical shrinkage is still in play as the chemical reactions proceed and the products of the reaction form. These products are slightly less in volume than the reactants.

There is a third mechanism theorized that relates more to the concrete microstructure and gel formation. Surface tension of the gel particles has been proposed as the mechanism, but it could only be a small part of the autogenous deformation.

The final mechanism proposed is disjoining pressure, where the adsorption of water to the gel particles is hindered. This occurs where the distance between the solid surfaces is less than two times the thickness of the free adsorbed water layer. The pressure is the result of van der Waals forces, double layer repulsion, and structural forces (Lura, 2003). This pressure is higher at higher relative humidity. When the relative humidity drops from water consumption, the disjoining pressure is reduced, causing shrinkage.

Autogenous shrinkage is hard to reduce without altering the actual water to cement ratio. If the autogenous shrinkage has to be reduced, it has been recommended that 25% of the coarse aggregate be replaced by a water-saturated lightweight aggregate (Xi et al., 2003). Holt (2001) agrees that the water to cement ratio is by far the most important factor in autogenous shrinkage, but lists three other factors that can influence it (shown in Figure 7). Holt was evaluating early age autogenous shrinkage for the most part, but noted three factors: bleed rate, chemical shrinkage, and time to hardening. A higher bleed rated decreases autogenous shrinkage, and earlier hardening does as well. Chemical shrinkage, the volume change when the hydration reaction progresses, directly influences autogenous shrinkage as well, but is generally not under the control of the engineer. Xi et al. (2003) lists these same factors as well.

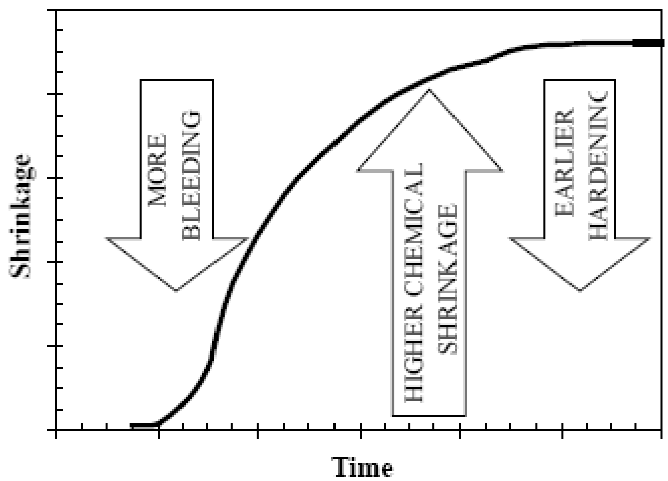

Figure 7: Direction of shift in early age autogenous shrinkage when influenced by other factors (Holt, 2001)

2.1.3.2.1 Mix design

Mix design is the factor with the largest influence on autogenous shrinkage. Autogenous shrinkage does not occur unless the water to cement ratio is below 0.42 (Holt, 2001). According to all sources, autogenous shrinkage increases as the water-to-cement level decreases, particularly below about 0.4 (Shing and Abu-Hejleh, 1999).

2.1.3.2.2 Cement type

Type K cement has a different crystalline structure than standard Portland cements. This shrinkage-compensating cement actually expands as the concrete sets, compensating for other types of shrinkage. Since this occurs inside the concrete, it is an autogenous movement type.

The Ohio Turnpike Commission (OTC) has used type K concrete for many years, and has over 500 bridge decks with type K concrete. The New York Thruway Authority

(NYTA) cast 47 decks in the early 1990s with this type of concrete. Linford and Reaveley (2004) reviewed the OTC and NYTA for their experiences with type K cement. The OTC has had good experience with type K decks, with most shrinkage cracking eliminated. They had to provide special treatment for the decks, including higher water to cement ratio, faster placement, faster implementation of curing, and continuous wet curing for 7 days. It must be noted that most of these are all well-known techniques for obtaining good shrinkage and cracking results, with or without the type K cement. NYTA had severe problems, and stopped using the cement. Overall, the benefits of type K are debated; some researchers show reduction in cracking, and others showed problems (Xi et al., 2003; Krauss and Rogalla, 1996).

2.1.3.3 Drying (Long Term) Shrinkage

Drying shrinkage is the most significant type of shrinkage in most concrete mixes, and has been called the most deleterious property of Portland cement composites (Zhang and Li, 2001). The mechanisms are similar to those of plastic shrinkage, but occur after the concrete has hardened. Drying shrinkage comes from the transfer of water from the concrete to the surrounding environment, thus increasing the surface tension in the pores. Eventually, the concrete will come to complete equilibrium with the surrounding environment. At that point the movement associated with moisture will simply follow the environmental conditions—if wet, then the concrete swells, if dry, it shrinks (Mindess and Young, 1981).

There are three mechanisms described in the literature: capillary stress, disjoining pressure, and surface tension. Each of these mechanisms is dominant in a different range of relative humidity. The most important mechanism in field conditions is the capillary stress, which is dominant from 45%-90% humidity. The three mechanisms all appear to be reversible, but a large portion of the drying shrinkage is irreversible. The reason for the irreversibility is not well known; it is thought that the stresses from those three mechanisms cause the calcium silicate hydrate particles to realign to a "matrix stable" configuration. This realignment seems to only occur during the first drying period; after that, subsequent wetting and drying does not have a large impact on the irreversible part of drying shrinkage (Xi et al., 2003; Mindess and Young, 1981; Brown et al., 2001).

It is thought by most researchers that the ultimate shrinkage values are not the most important facet of the drying shrinkage issue. The actual rate of shrinkage is more important, as this compared with strength gain, creep and other time-dependent factors actually determines whether there will be cracking. If the shrinkage occurs quickly while the strength gain occurs slower, the concrete may crack early even though at the fully-developed values of both the concrete would have been strong enough to handle the load. In addition, if the shrinkage occurs quickly, creep is unable to relieve the stress. Xi et al. (2003) cite the following example: "For a concrete prism fully restrained at both ends, cracks may develop at a shrinkage strain of around 200~250 με if not accounting for the creep effect of concrete. Under high shrinkage rate, 200~250 με could easily occur at the age of 10 days under normal room temperature and 50% humidity. Therefore, proper measures must be taken to reduce not only the ultimate shrinkage strain but also the shrinkage rate." It is generally perceived that reducing the shrinkage rate is more difficult than simply reducing ultimate shrinkage.

2.1.3.3.1 Curing methods

Curing of the concrete determines to a large extent the rate at which the drying shrinkage occurs (Krauss and Rogalla, 1996). If the concrete remains in a saturated condition, then drying shrinkage should be nearly eliminated for that period. Thus 7 day wet curing is very beneficial for letting the concrete gain strength before the shrinkage stresses cause cracking, and some even suggest 14 day. However, research by Holt (2001) shows that curing conditions for the first 24 hours do not affect shrinkage occurring at later ages. There seems to be some disagreement over how much curing conditions actually affect long-term behavior.

2.1.3.3.2 Mix Design

Mix design also has a significant impact on drying shrinkage. In particular, decreasing the water content decreases the drying shrinkage of the concrete. Interestingly, this is opposite to the results with autogenous shrinkage. The water to cement ratio has not been shown to have a conclusive effect on cracking, just on shrinkage. Decreasing the cement content decreases shrinkage, as the cement paste itself is the phase that causes the shrinkage. Essentially, high paste volume increases drying shrinkage. Many researchers have noted that high-slump concrete tends to increase cracking, which makes sense: high paste volume increases slump. Schmitt and Darwin (1999), for example, recommend that no more than 27% of the total volume of the concrete be cement and water (Schmitt and Darwin, 1999; Linford and Reaveley, 2004; Xi et al., 2003; Krauss and Rogalla, 1996; Cheng and Johnston, 1985).

Krauss and Rogalla (1996) list several other factors known to reduce drying shrinkage: maximizing the amount of aggregate (which reduces paste volume), using Type II cement, and using aggregate with low-shrinkage properties. A soft aggregate, such as sandstone, greatly increases the shrinkage of a concrete over a concrete using a hard aggregate (like dolomite); one researcher showed a 141 percent increase in that case. The absorption of the aggregate has been shown to reflect the drying shrinkage, but a quantitative relationship is not known (Babaei and Purvis, 1995; Cheng and Johnston, 1985). It is also known that cements from different sources can have widely different shrinkage characteristics; in some cases, one cement can have shrinkage over 100% higher than another (Babaei and Purvis, 1995).

2.1.3.3.3 Admixtures

Admixtures can modify the drying shrinkage. Shrinkage reducing admixtures reduce the surface tension in the pore water, reducing the driving force of the drying shrinkage, as well as the other types of shrinkage. Shrinkage reducing admixtures are very effective in reducing drying shrinkage (Xi et al., 2003). High range water reducers, retarders, and superplasticizers seem to have only a minor impact on drying shrinkage.

2.1.3.4 Carbonation Shrinkage

Carbonation shrinkage occurs when the concrete is exposed to air with high concentrations of carbon dioxide and about 50% relative humidity for long periods of time. The concrete behaves as if it were exposed to drying conditions with a relative humidity far below the actual humidity (Brown et al., 2001). The conditions mentioned above occur most often in structures like parking garages, while bridges seldom have these conditions (Mindess and Young, 1981). Therefore, this type of shrinkage is outside the scope of this work, and will not be discussed further.

2.1.4 Thermal Effects

Thermal effects are as important to the cracking problem as shrinkage is, but are often overlooked since they are largely outside the control of the engineer. Nevertheless, the strain applied by temperature changes alone can easily be enough to cause cracking (Krauss and Rogalla, 1996; Aktan et al.,2003).

The thermal stress-free condition is locked in at the time and temperature of the concrete's setting. From that time on, any temperature different than that experienced at the setting time will cause strain in the concrete. If this is restrained, then the strain is converted to stress. Differential stresses are created when the deck and the girders of a composite deck are expanding or contracting at different rates.

High early temperatures in the concrete can create early age cracks, as the thermal stresses act upon fresh concrete with low strength. Concretes that have high early strength usually also have a high heat of hydration, leading to more thermal cracking problems. To prevent excessive thermal gradients, the peak and placement temperatures of the concrete need to be limited, but how much is open to debate. There are numerous methods to reduce the heat related problems; they are discussed below.

2.1.4.1 Heat of Hydration

The heat of hydration for the concrete sets the baseline upon which all other thermal effects work. A high heat of hydration, combined with an early set time, will lead to an elevated stress-free temperature, which will greatly exacerbate the thermal movement problems. The problems depend also upon the geometry of the member; a large member will retain the heat generated by hydration longer, making a higher temperature when the concrete hardens more likely (Brown et al., 2001). If the concrete sets at, perhaps, 100° F, and the concrete eventually reaches 20° F at some later date, that thermal movement will add over 200 psi of tensile stress to the deck (Krauss and Rogalla, 1996). It is beneficial, therefore, to reduce the heat of hydration and to keep down the temperature at setting.

The heat of hydration is impacted by several factors. The most important is the cement type. A cement heavy in tricalcium silicate will have a much higher heat of hydration than one heavy in dicalcium silicate. Type III cement has the highest heat of hydration, both because of the high tricalcium silicate and tricalcium aluminate percentages, and because the clinker particles are ground to a smaller size, increasing their reactivity. Type I cement has a somewhat lower heat of hydration, and Type IV, specially designed to reduce the heat of hydration, has by far the lowest heat of hydration. Typically, the faster the cement gains strength, the higher the heat of hydration, because of the concentration of reactions in time—more reactions at the same time means more heat at that time. It is recommended that cements with a lower hydration heat be used where possible (Xi et al., 2003, Shing and Abu-Hejleh, 1999). In particular, Type II cement, which has slightly lower heat of hydration than Type I, is recommended for general purposes (Krauss and Rogalla, 1996; Shing and Abu-Hejleh, 1999; Babei and Purvis, 1995; Aktan et al., 2003).

For the concrete, however, there are other factors than simply the type of cement. The cement volume in the actual mix design also determines the concrete heat of hydration. Increasing the cement volume in the concrete increases the amount of heat generated by hydration.

Finally, some admixtures alter the heat of hydration. Retarders decrease the maximum heat of hydration by spreading out the hydration reactions in time, giving more time for the concrete to lose heat to the environment. In addition, fly ash has been successfully used to reduce cracking by reducing the strength gain and early concrete temperature (Krauss and Rogalla, 1996; Shing and Abu-Hejleh, 1999).

2.1.4.2 Temperature at Casting

The actual temperature at the time of set determines the thermal behavior of the concrete from that time forward. Heat of hydration has a large influence on the setting temperature, but so do environmental conditions. The procedures used in the casting of the concrete can significantly modify the setting temperature as well.

If possible, the concrete should be cast at approximately the median temperature for the year; cracking is worse when the concrete is cast at either low or high temperatures (Krauss and Rogalla, 1996; Meyers, 1982; Cheng and Johnston, 1985). Obviously that is rarely possible, but it is possible to bring the temperature of the concrete close to that level. However, the temperature of the concrete at casting is rarely the temperature of the concrete at setting, because the concrete will quickly come to the temperature of the environment (Aktan et al., 2003). For this reason, it is unlikely that procedures such as cooling the mix with nitrogen actually have much impact on the setting temperature.

Agencies usually restrict batching temperature, both of the air and of the concrete itself. Concrete does not set properly at low temperatures; high temperatures cause problems with thermal movement. Air temperature at batching must be between 45° and 80° F (Rogalla et al., 2003). This is not practical in some regions of the country. Concrete mix temperatures must be above 50° F for the first 72 hours, and below 80° F (Xi et al., 2003, Shing and Abu-Hejleh, 1999; Krauss and Rogalla, 1996; PCA, 1970). This is very difficult to attain if the air temperature is outside that envelope, because concrete quickly approaches the ambient temperature (Aktan et al., 2003).

2.1.4.2.1 Weather

The weather at the time of the concrete setting is important to the temperature of the concrete at setting. It is often recommended to batch late in the day during the summer months; this allows the setting of the concrete to take place late in the evening as the ambient temperature decreases. Night batching has been shown to significantly reduce deck cracking (Krauss and Rogalla, 1996; Purvis, 1989). In the winter, casting should take place so that the concrete will set during the warmest part of the day. These procedures will minimize the effect of the annual temperature cycle on the concrete. It is usually recommended not to batch when the temperature is above 80°.

2.1.4.2.2 Heat of hydration

The heat of hydration, as discussed above, will raise the concrete's setting temperature. It is rarely feasible for the engineer to modify the mix to reduce the heat of hydration, as strength and shrinkage considerations dictate the mix proportions. Retarders are recommended to reduce the temperature gain from the heat of hydration (Xi et al., 2003).

2.1.4.2.3 Batching Temperature

During the winter and summer, the concrete is often warmed or cooled to meet department of transportation specifications on the temperature of the concrete at batching. In the winter, the aggregate is often heated through various means; in the summer, the water is chilled, ice is added, or the mix cooled with liquid nitrogen. Whether this does any good for the actual setting temperature is doubtful. Aktan et al. (2003) found that the concrete temperature at placement had little long term effect because the concrete quickly reached the ambient temperature.

2.1.4.3 Cooling After Batching

The first temperature change that the concrete will see is the actual cooling as the heat of hydration is released. This can very often cause cracking, because the concrete is still weak, but the matrix itself has already formed. The restraint provided by underlying beams and the forms themselves is sufficient to translate the strain into stress. Cracking from this source is usually formed above the uppermost transverse bars and is full depth (Xi et al., 2003).

Krauss and Rogalla (1996) give an example of the potential stress generated by the cooling of a deck that was 50° F above the temperature of the restraining girders:

> A 28° C (50° F) temperature change in the deck relative to the girders can cause stresses greater than 1.38 MPa (200 psi) when the concrete has an early effective modulus of elasticity of only 3.5 GPa (0.5 x 10^6 psi), and greater than 6.89 MPa (1000 psi) when the early effective modulus is 17.2 GPa (2.5 x 10^6 psi).

2.1.4.4 Diurnal Cycle

A concrete bridge deck's temperature will mirror to an extent the ambient conditions of the surrounding environment. The heat of a bridge deck will vary as much as 50° Fahrenheit during the course of a day. This type of thermal movement is too short-term to be alleviated by creep, and thus must be taken by the concrete itself (if restrained). This is the primary source of thermal stress, since the change is non-uniform on the structure; this non-uniformity is covered in the solar radiation section (Xi et al., 2003; Krauss and Rogalla, 1996).

Krauss and Rogalla (1996) give examples of the levels of thermal stress from the diurnal cycle that can be reached, from analytical analysis of the system. The assumption in these examples is of a linear temperature gradient in the bridge. With a 50° F temperature change, the tensile stresses can reach 1350 psi on simply-supported steel girders, and 1480 psi on simply-supported concrete girders. Over the interior supports of a continuous span bridge, the tensile stress could reach 2000 psi on concrete girders. Those numbers were calculated theoretically from the mechanics of the system; in reality, the concrete would probably fail long before those stresses were reached.

2.1.4.5 Annual Cycle

The annual temperature cycle also brings significant temperature fluctuations to the bridge deck. During a year, the high temperature during a day may go from 0° to 100° Fahrenheit. This type of fluctuation is less problematic, because it is uniform across the structure. Thus, the girders and deck will see precisely the same changes. If the deck and girders have the same coefficient of thermal expansion, little stress will be seen. However, when the girders are steel, the total temperature change is the source of the stress, rather than the differential change across the structure (Xi et al., 2003). When combined with the diurnal cycle, the annual cycle brings a temperature range of some 120°, and that is just the air temperature in the surrounding environment. This range is what has to be handled when the deck and girders are not the same material. The concrete itself is also likely to get hotter from radiation—but since that heating is non-uniform and non-linear, it is considered in the next section.

The annual temperature cycle is another of the factors that the engineer has no control over, but it is useful to consider it. Krauss and Rogalla developed equations to calculate the stress developed in a concrete bridge deck with various conditions. Obviously, the worst condition would have the concrete and the girders see different temperatures; if they differ by 50° after the stress-free temperature for the combination is when they are the same temperature, the tensile stress in the concrete can approach 1000 psi, far beyond the tensile capacity of the concrete. However, in most cases the stresses from the annual cycle are limited, since the concrete and steel have at least similar coefficients of thermal expansion (Krauss and Rogalla, 1996).

2.1.4.6 Solar Radiation Heat

This is one of the worst temperature impacts on the bridge deck. The sun heats the top surface, while the bottom surface remains relatively cool, particularly if over a large body of water. This yields very significant differential strains, causing curvature and stress in the deck; the free deck will try to curve convex upward. If the solar radiation heating is sustained for a full day, eventually the deck will increase in temperature significantly, while the underlying girders remain relatively cool. This can again put significant stress into the concrete (Krauss and Rogalla, 1996). Figure 8 (Figure 1 from Krauss and Rogalla) illustrates these different types of thermal movements. When these strains are translated to stresses (Figure 9), the stresses can be very large. Figure 9 is also from Krauss and Rogalla, and gives results of a typical calculation. They undertook a large number of similar calculations to determine the maximum stresses that could be seen by the girders and deck.

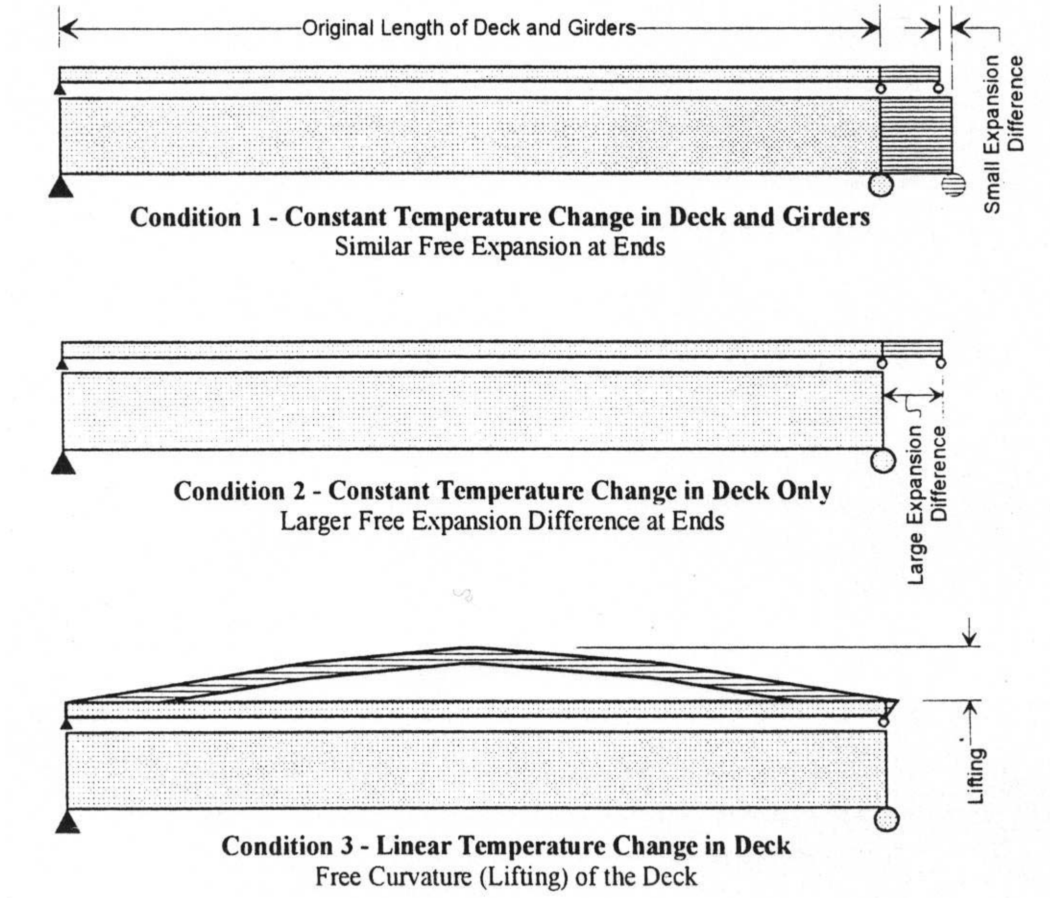

Figure 8: Strain effects of various temperature changes (Krauss and Rogalla, 1996)

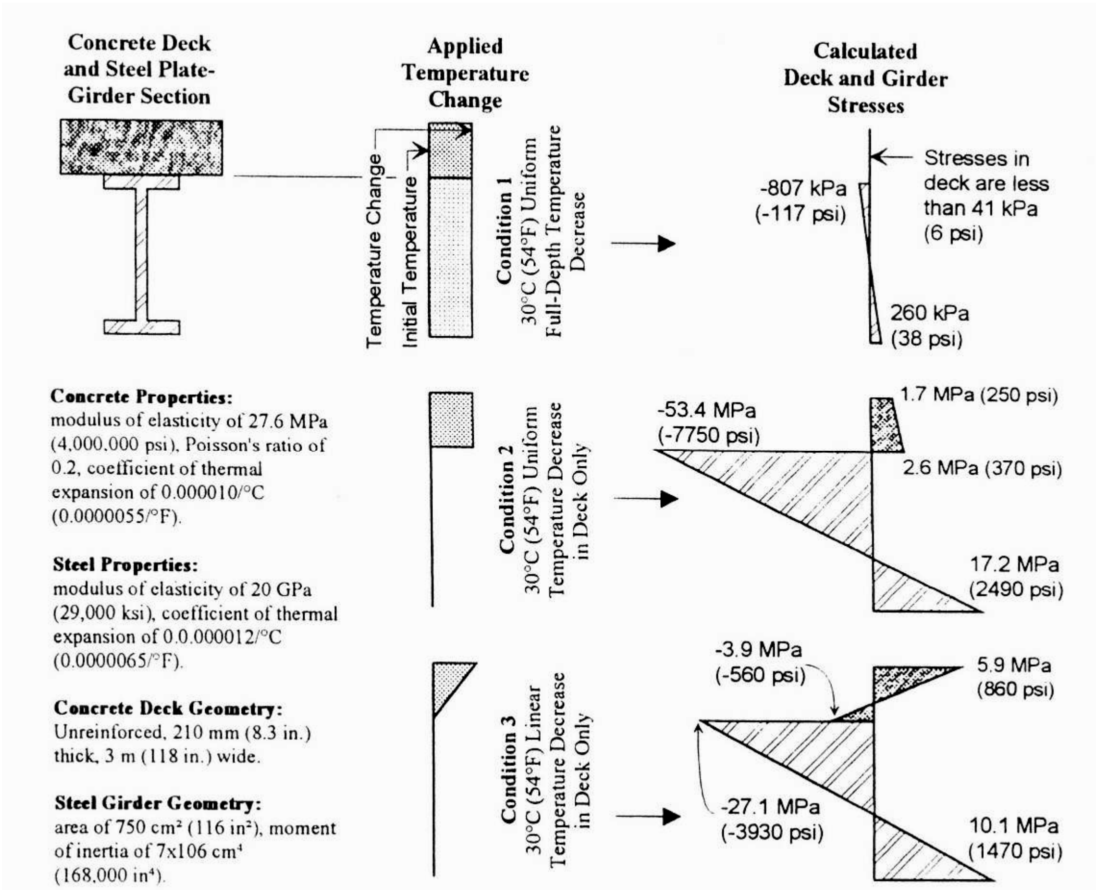

Figure 9: Example deck and steel girder stresses for various temperature changes (Krauss and Rogalla, 1996).

2.1.4.7 Compared with temperature at casting

The strain in the concrete depends on the difference between the concrete temperature and that at which the concrete set. The only thing the engineer can control to any degree is the batch temperature, which should be somewhere between the extremes to try to reduce the maximum strains seen.

2.1.4.8 Coefficient of Thermal Expansion

The coefficient of thermal expansion determines how large the strains are with the variation in temperature. This is essentially beyond the control of the engineer.

However, the differing thermal coefficients of concrete and steel may explain why it has been seen that steel girder bridges are somewhat more prone to cracking than concrete girder bridges. At the time of setting, the stress-free temperature is set, with the concrete usually at a slightly higher temperature than the girders. Then, as the annual and diurnal temperature cycles occur, the concrete deck and steel girders move at different rates, causing stresses to occur in the system.

The coefficient of thermal expansion of concrete is from 4 to 7 $\mu\varepsilon/°F$, while that for steel is 7 $\mu\varepsilon/°F$ (Xi et al., 2003; Shing and Abu-Hejleh, 1999; Mindess and Young, 1981). Concrete with a higher coefficient of thermal expansion is theoretically desirable on a steel girder bridge, in order to match the movement of the girders, but this also would increase the thermal stresses from other sources (like temperature gradients in the deck from radiation), reducing any benefit (Xi et al., 2003).

2.1.4.8.1 Aggregate

The aggregate used has a large impact on the coefficient of thermal expansion. However, it is rarely feasible for aggregates to be chosen based on the thermal expansion coefficient. The final coefficient of thermal expansion is a combination of the coefficients of the cement matrix and that of the aggregate; the paste coefficient is usually 2 to 3 times higher than that of the aggregate (Mindess and Young, 1981; Krauss and Rogalla, 1996; Xi et al., 2003).

2.1.5 Deflections

This is the third and least important source of strain in the concrete. It, like much of the temperature strain, is of short duration, so the strain cannot be relieved by creep.

2.1.5.1 Live Loads

These obviously produce both stress and strain in the concrete, both after curing and potentially during the curing process if the concrete feels vibrations induced by traffic. These loads are added to those from shrinkage and thermal factors, but it is typically considered that these loads are not significant in the cracking problem. This is because the stresses induced are usually much lower than those from other sources, and they are usually compressive for the deck as well. In addition, these are the loads that the decks are actually designed to carry. Traffic-induced vibrations during curing have not been found to be detrimental (Krauss and Rogalla, 1996).

2.1.5.2 Formwork

The formwork potentially can induce strain, as it is holding the concrete in a certain position during casting. When removed, the structure settles into its dead-load deflected shape, inducing tensile strain in the concrete. There has been some research done on types of formwork, with inconclusive results on whether there is a correlation between formwork type and cracking of the deck. Some advocate stay-in-place forms, while others say they increase the cracking (Krauss and Rogalla, 1996; Cheng and Johnston, 1985). Nothing conclusive has been determined.

The other type of strain associated with formwork comes from deflection of the formwork while the concrete is plastic. Cracking may occur over the supports of continuous deck bridges in this condition; this situation can be eliminated by using appropriate pour sequences to eliminate formwork deflection inducing tensile stresses in those locations (Krauss and Rogalla, 1996). It should be noted that this type of job sequencing may cause cold joints and construction difficulties.

2.1.6 Restraint

Without restraint, the strain would simply cause movement of the concrete. However, bridge decks are highly restrained systems, both internally and externally. When restraint is present, the strain is converted to stress according to the modulus of elasticity of the concrete (assuming linear elastic behavior). There are two classes of restraints: internal and external. The internal restraint on a bridge deck comes from the reinforcement in the deck, from the aggregate in the deck, and from any fibers in the deck. The external restraint comes from the girders and from any end restraints; the expansion joints are planned to reduce external restraint. However, if the girders and deck are composite, as is often the case, nearly all of the external restraint comes from the girders anyway (Krauss and Rogalla, 1996, Brown et al., 2001).

2.1.6.1 Internal

There are several sources of internal restraint to the concrete matrix. The reinforcing steel is chosen to carry load, but it also is a restraint to the concrete. When the concrete shrinks, the reinforcement does not, thus inducing tensile stress in the concrete and compressive stress in the reinforcement.

2.1.6.1.1 Reinforcement

The rebar imbedded in the concrete provides a significant degree of longitudinal restraint, and to some extent lateral as well. Since the loads are most significant longitudinally, where they can accrue along the length of the bridge, this is a problem for the bridge deck. Embedded reinforcement, to a lesser extent than girders, restrains the deck against shrinkage and thermal movement, as the coefficient of thermal expansion of the reinforcement is likely different from that of the deck. Of course, the engineer cannot remove the reinforcement from the deck, but there are a few factors that are under the engineer's control.

2.1.6.1.1.1 Epoxy coated

Epoxy coated rebar behaves differently in its interaction with concrete than does standard rebar. It has been shown that bridges with epoxy-coated rebar behave worse than those with standard black rebar. There is an increasing likelihood for cracking shown, and the epoxy-coated bars develop considerably less bond stress. The cracks tend to be larger with the epoxy-coated rebar (Krauss and Rogalla, 1996; Meyers, 1982). The epoxy rebar helps chloride-ion protection in the laboratory under ideal conditions, but in practice there has not been any benefit found. In addition, the epoxy sometimes delaminates from the steel, causing a failure zone to develop at the bonding surface (Linford and Reaveley, 2004).

2.1.6.1.1.2 Rebar location

Some researchers felt like the rebar location, particularly how much cover was present, had an impact on the cracking. It has been shown that cracking tends to occur over the transverse reinforcing steel. It is possible that this occurs because of insufficient cover at those locations. As the concrete settles in the plastic phase, a zone of weakness tends to develop over the rebar, which fractures first under the stresses leading to cracking (Aktan et al., 2003; Issa, 1999; Linford and Reaveley, 2004; Babaei, 2005).

2.1.6.1.2 Aggregate

It has been shown that the aggregate types have a significant impact on all facets of concrete behavior. Aggregate provides a large measure of the concrete's internal restraint. However, it is rarely feasible to choose aggregate types based upon the measure of internal restraint provided. Aulia (2002) demonstrated that the type of aggregate had a significant impact on the properties of the concrete.

Clean, low shrinkage aggregate is important in getting a high quality concrete. It is well known that the type of aggregate has a significant impact on shrinkage of the concrete, and on the time to crack as well (Krauss and Rogalla, 1996).

Larger aggregate is recommended in a number of sources, in order to minimize the paste volume without sacrificing workability (Xi et al., 2003, PCA, 1970; Shing and Abu-Hejleh, 1999). In addition to minimizing paste volume, the larger aggregates tend to bear directly on one another, so shrinking paste cannot move them. This tends to channel the stress into microcracks within the cement paste, rather than shrinkage. As long as these microcracks to not turn into larger cracks, the effect is considered beneficial. It is commonly recommended to achieve the highest possible aggregate volume in the mix, as less paste decreases shrinkage and thermal problems (Xi et al., 2003). “In general, concrete mixes with good quality, clean, low shrinkage aggregate with high aggregate to paste ratio have been observed to perform better (Saadeghvaziri and Hadidi, 2002).”

2.1.6.1.3 Fibers

Fibers provide internal restraint as well, particularly against movement before curing. Steel fibers will continue to provide restraint after curing, as their high modulus of elasticity will continue to take load. Polymer fibers stop providing restraint once the concrete’s modulus of elasticity becomes higher than the fibers’. There is some question whether early restraint is beneficial or detrimental to the concrete. If the concrete is still in the plastic stage, there would not be any stress captured in the matrix, so it likely doesn’t hurt to have this early restraint.

2.1.6.2 External

The external restraint on bridge decks is also significant. The girders are the primary source of the restraint. The best-case scenario is if the girders and deck are cast monolithically; then the shrinkage stresses are equal, and the thermal effects are minimized as well (Krauss and Rogalla, 1996). Most bridges, however, have the deck cast independently from the girders, and are composite systems.

2.1.6.2.1 Girders

The girders are the portion of the bridge in contact with the deck, and thus their composition and design can influence the behavior of the bridge deck. As the deck contacts the girders all along the length of the deck, and shear systems such as shear studs are used, longitudinal movement of the deck relative to the girders is prevented. Girders restrain the deck movement whenever they do not have temperature or shrinkage strains identical to the deck. Because steel girders do not experience any long term drying shrinkage, they tend to exert greater restraint on the deck than concrete girders. Since only a portion of the deck is restrained, there are induced stresses from the eccentric restraint present as well (Krauss and Rogalla, 1996).

When large girders are used, they can restrain approximately 60% of the uniform free strain at the upper surface of the deck; smaller girders can restrain 35 to 45% of the free strain at the upper surface (Krauss and Rogalla, 1996). Of course, there are many other variables as well.

If the deck has a linear free strain rather than a uniform free strain, the deck tries to curve to alleviate this. This type of movement is restrained at a much higher percentage, from 75 to 95% (Krauss and Rogalla, 1996).

2.1.6.2.1.1 Concrete vs. steel

Due to the fact the steel has a different coefficient of thermal expansion than concrete, the degree of restraint placed by differential movement depends on the material of the beams. In addition, the steel has a higher modulus of elasticity, leading to a higher degree of restraint on any free strain in the deck. Finally, the steel girders do not shrink like the concrete deck; the concrete deck strain is completely restrained by the girders. This, combined with the thermal difficulties, explains why cracking is more common on steel girder structures (Xi et al., 2003, Aktan et al., 2003; Krauss and Rogalla, 1996; Meyers, 1982; Cheng and Johnston, 1985; Linford and Reaveley, 2004).

2.1.6.2.1.2 Continuous vs. Simply-Supported

It is thought that continuous-span structures are more susceptible to cracking than simple-span structures (Krauss and Rogalla, 1996; Meyers, 1982; Aktan et al., 2003; Linford and Reaveley, 2004). This is likely due to the negative moment regions over supports and to the longer stretches of deck without any expansion joints. The negative moment regions induce tension in the deck over the support, which a deck already in tension due to shrinkage and potentially thermal effects is ill-prepared to withstand (Cheng and Johnston, 1985; Perfetti et al., 1985).

2.1.6.2.1.3 Girder size and spacing

Research indicates that the size and spacing of the girders effect cracking, but as these are designed based on other issues, they cannot be altered simply to protect the bridge deck. Restraint is increased with larger girders, and with more girders; higher restraint increases the likelihood of cracking (Shing and Abu-Hejleh, 1999).

2.1.6.2.1.4 Composite deck/girder systems

Composite decks and girders are the norm in bridge design, as they greatly improve the efficiency of the load-carrying system. Most of the discussion of restraints thus far has assumed that the deck and girders act compositely. However, these systems are the source of much of the restraint upon the system. If the deck and girders did not act compositely, the deck would be free to move with shrinkage and thermal strains to a much greater extent (Krauss and Rogalla, 1996). It is not a coincidence that the cracking problem became much more pronounced as the use of a composite deck/girder system became common.

However, it would be premature to advocate the return to noncomposite systems. Further research into the relative merits of the systems is in order, however, particularly in light of the high cost of repairing and replacing cracked decks.

2.1.6.3 Expansion joints

The design and placement of expansion joints can affect how well movements are taken up by the bridge, but they cannot alleviate restraint placed on the deck by the simple presence of the girders.

2.1.7 Modulus of elasticity

The modulus of elasticity of concrete is poorly understood, in that the modulus of concrete changes both over time and with loading. According to Krauss and Rogalla (1996), the modulus of elasticity affects the stresses in the concrete more than any other property. The modulus of elasticity determines the conversion ratio of strain to stress in the concrete (Xi et al., 2003). As the strain is the given for both shrinkage and thermal movements, a lower modulus of elasticity will decrease the stress in the concrete. However, a lower modulus of elasticity comes from a concrete with a lower binder ratio, and thus usually a lower strength as well.

A concrete's modulus of elasticity approximately mirrors the concrete's strength (Xi et al., 2003). It is unclear if there is any net benefit from reducing the binder ratio, since the strength is usually reduced. Of course, the external loads apply a given stress to the system, so a lower modulus of elasticity will increase deflections--except that the effect will simply be a reduction of the load taken by the deck and an increase of the load taken by the girders (whose modulus of elasticity is a constant).

To reduce the modulus of elasticity without reducing the strength, the primary approach is to use aggregates with a low modulus of elasticity (Xi et al., 2003; Krauss and Rogalla, 1996). Aulia (2002) also found that the modulus of elasticity was largely dependent on the aggregate used, and demonstrated that the relationship held true in fiber-reinforced concrete as well. Whether choosing aggregate to give a low modulus of elasticity is practicable depends on the location where the concrete is batched.

2.1.7.1 Modulus gain

There is some research done of the modulus gain curves. These curves essentially mirror the strength-gain curves of the concrete. In order to get better crack performance, Xi et al. (2003) recommend that a concrete with low early strength and modulus of elasticity be used. However, the cracking performance depends on the relationship of tensile strength to the modulus of elasticity, and that relationship is very hard to determine, so attempting to avoid cracking by using a low modulus concrete may not succeed.

2.1.8 Creep of Concrete

Creep of concrete is one factor beneficial to the engineer. Creep occurs with when the concrete is under load for long periods of time. Over time, the concrete slowly moves away for the load, deforming according to the load. Essentially, concrete tries to alleviate stress by a restructuring of the matrix. There are two types of creep: basic creep, which occurs without moisture movement to or from the environment, and drying creep, which is the additional creep caused by drying. The differences between these types of creep, and the fact that there is no distinct separation between instantaneous strain and time-dependent strain, make quantifying creep difficult (Linford and Reaveley, 2004). Research has been done on how great a benefit can be expected from creep and what influences its behavior. Krauss and Rogalla (1996) list creep as one of the major factors effecting bridge deck cracking. Creep occurs in the cement paste; aggregates do not creep. However, lower modulus aggregates encourage creep, possibly by increasing the localized stress in the cement paste (Xi et al., 2003). The nature of creep itself is not well understood; the mechanism seems to be related to the response of calcium silicate hydrate to stress—calcium silicate hydrate has multiple phases it may switch between (Mindess and Young, 1981).

It has been shown that the tensile creep can relax shrinkage stresses by up to 50%, doubling the strain failure capacity. Both the magnitude and time history of the shrinkage stress influence the time of cracking. Altoubat and Lange (2002) showed that the tensile creep caused their sample mixes to crack at twice the expected failure time based on shrinkage analysis for high performance concrete, and three times the expected failure time for the standard mixtures. Interestingly, they found that the actual evolution of the stress greatly altered the creep behavior. Concrete in a restrained shrinkage test that was sealed for three days and then unsealed actually cracked earlier than unsealed concrete. This, they believe, comes from the higher modulus of elasticity of the sealed concrete, and the exposure shock acceleration of the shrinkage. In addition, they showed that periodic wetting increased the creep of the concrete.

The creep of concrete typically mirrors the compression strength of the concrete. The creep rate (the concrete's rate of relaxation) decreases at a faster rate than the modulus of elasticity and tensile strength increases. This allows the stress into the concrete to catch up to the tensile strength over time (Figure 10). Note the tensile strength curve is flatter than the stress gain curve (Brown et al., 2001).

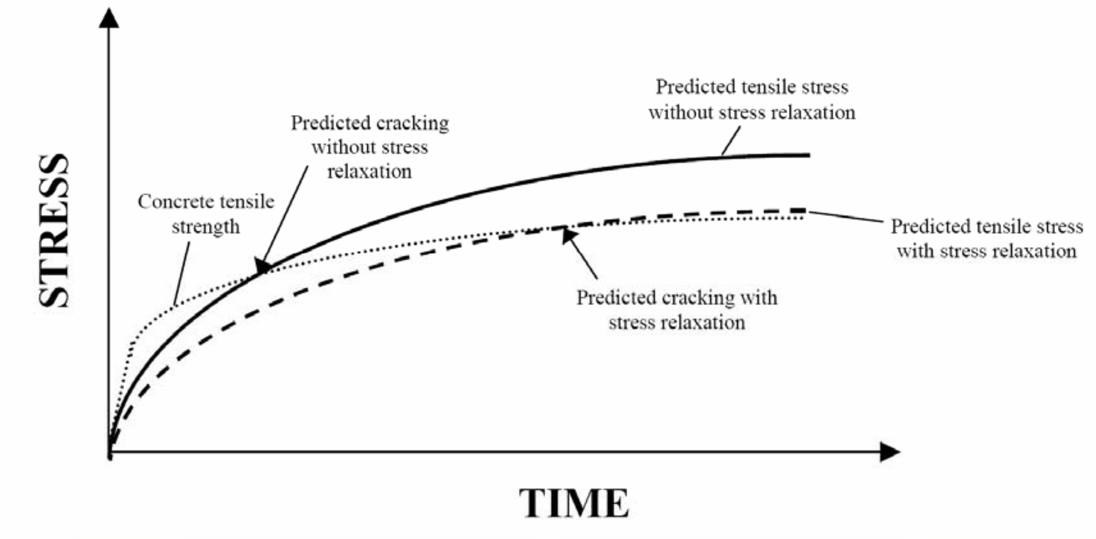

Figure 10: Time dependence of restrained shrinkage, creep, and tensile strength (Brown et al., 2001 after Mehta, 1993)

2.1.8.1 Mix Design

There has been research done on exactly what types of mixes creep more or less. In particular, concrete with higher water content creeps more (Krauss and Rogalla, 1996). Since higher water content also increases shrinkage, it is unclear whether this addition of water is actually beneficial. Increasing cement paste volumes increase the creep potential (Xi et al., 2003).

As the compressive strength of a concrete increases, creep decreases and tensile strength increases. However, the creep decreases at a much greater rate than the increase of the tensile strength. This helps to explain why higher strength concretes usually have worse crack performance than normal strength concretes (Xi et al., 2003).

2.1.8.2 Curing Conditions

Curing conditions significantly modify the creep behavior of concrete. Drying creep dominates basic creep (creep not depending on air drying) on bridge decks, which are usually drying from both sides. "Drying creep is typically 2 to 3 times basic creep when the air relative humidity is 70 to 50 percent, respectively (Krauss and Rogalla, 1996)."

2.1.8.3 Admixtures

Addition of retarders can increase the creep at early age, which can relieve more of the early age shrinkage and thermal issues. Slower curing mixes have higher creep (Krauss and Rogalla, 1996).

2.1.8.4 Plastic Settlement

Plastic settlement of concrete occurs while the concrete is still fresh. As water rises to the surface, the concrete subsides. If there is insufficient cover, cracking will occur over the top reinforcement as the concrete subsides on either side. Babaei (2005) considers this one of four primary causes of bridge deck cracking.

2.1.9 Geometry

The geometry of the design can influence bridge deck cracking, as it can influence stress concentrations and differential movements. This is a very complex subject, and thus difficult to make generalizations about, but a few things are known about how geometry influences bridge deck cracking.

2.1.9.1 Skew

Some respondents in the survey indicated that they thought skew increased cracking, probably because of stress concentrations. Krauss and Rogalla (1996) believe that skew does not significantly affect transverse cracking, but that it does cause slightly higher stresses near the corners. One researcher (Purvis, 1989) found bridges with a skew over 30 ° were more susceptible to transverse cracking.

2.1.9.2 Depth of Deck

The depth of deck influences the differential movements associated with solar radiation heating of the top surface and can also influence other temperature effects, as the inner core will retain heat longer. However, for actual concentration of stresses, the depth of deck has a minimal impact. Though research is lacking, the information that there is indicates that thinner decks lead to more cracking (Xi et al., 2003; French et al., 1999).

2.1.9.3 Cover

It is believed that the concrete cover does have an impact on deck cracking, but there is not a consensus on what that impact is. Shallow cover increases the likelihood of settlement cracking (Krauss and Rogalla, 1996; Cheng and Johnston, 1985). However, if the cover gets too deep, over about 3 inches, the steel reinforcement is less effective at distributing tensile stresses (Krauss and Rogalla, 1996). Some researchers found worse cracking with cover over 3 inches while others found no correlation. Top cover between 1.5 and 3 inches is recommended (Xi et al., 2003; Krauss and Rogalla, 1996; PCA, 1970).

2.1.10 Tensile Strength

The tensile strength of the concrete determines if the concrete will actually crack. Unfortunately, concrete is very weak in tension and the actual tensile strength is poorly understood, as it changes with time. The tensile strength of concrete is often estimated as 10% of the concrete's compressive strength (ACI Committee 318, 2002). The actual tensile strength is subject to considerable fluctuation from sample to sample, because the tensile strength is very sensitive to anything acting as a stress concentrator or crack initiator. Once the concrete starts cracking in tension, it fails almost instantly.

The concrete cracks when the stress is higher than the tensile strength at that time. If the stresses develop faster than the strength, the concrete will crack at early age. Figure 11 shows the tensile strength curve—when the stress reaches the tensile strength, the concrete will crack.

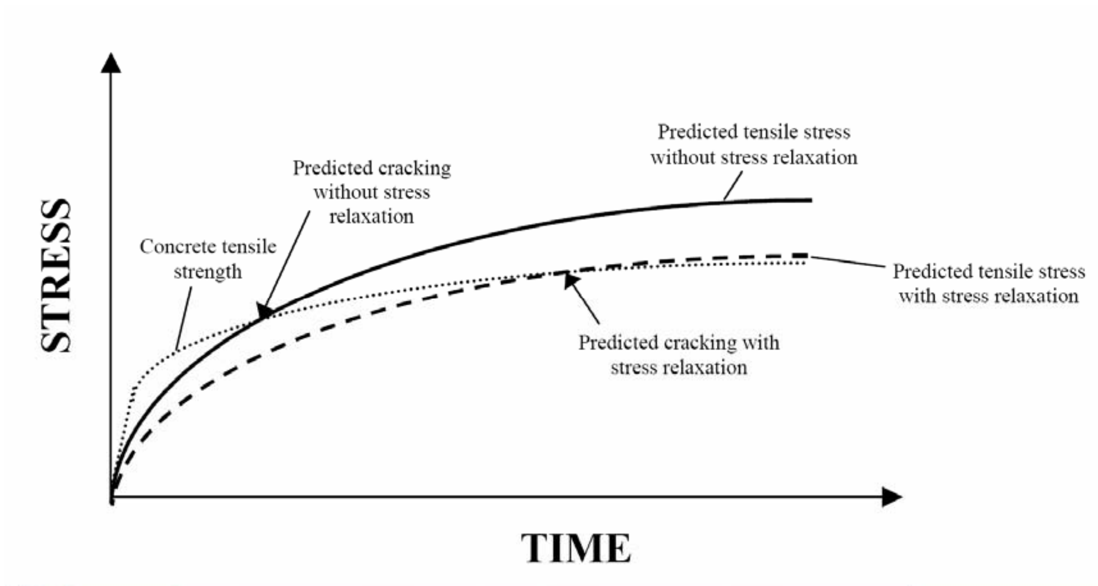

Figure 11: Time depende nce of res trained shrink age, stress relaxation (cree p), and tensile strength (Brown et al., 2001 after Mehta, 1993)

To further complicate matters, some evidence shows that the concrete cracks below its tensile strength. Table 1 shows some results obtained by Altoubat and Lange (2002) showing that the concrete was cracking at a restrained shrinkage stress below that of the direct tensile strength. Likely this would be due to the likelihood of flaws in larger samples causing cracking to propagate at a lower stress level.

Table 1: Restrained shrinkage stresses and age at cracking (Altoubat and Lange, 2002)

Concrete Mix	Stress (MPa)	Age (hrs)	Direct Tensile Strength (MPa)	Stress/Strength	Delay factor
HPC-0.32	1.759	69.5	2.325	0.757	NA
HSF-0.32	1.898	100.5	2.465	0.770	1.446
NC-0.4	2.130	144.7	2.649	0.804	NA
SF-0.4	2.221	174.8	2.790	0.796	1.208
NC-0.5	1.782	159.5	2.214	0.805	NA
SF-0.5	1.767	181.0	2.307	0.766	1.135
PP-0.5	1.887	134.5	2.083	0.906	0.843
HPC: High performance concrete, NC: Normal plain concrete, SF: Steel fiber, HSF: HPC with steel fiber; PP: Polypropylene fiber, Delay factor = FRC fracture time / PC fracture time					

There are two important factors: the rate of increase and the ultimate strength. If the tensile strength of the concrete rises at a fast enough rate, it can outpace stresses induced by shrinkage at early age, preventing cracking at early age. Long term, the ultimate tensile strength needs to be high enough to resist all stresses that come upon it. There are several factors that can increase tensile strength.

2.1.10.1 Fibers

Fibers can greatly help tensile strength at early age. However, polymer fibers have a modulus of elasticity lower than that of hardened concrete, and thus do not help long term. It has been shown that steel fibers increase ultimate tensile strength. The fibers are potentially very beneficial in increasing the rate of increase of the tensile strength, thus avoiding early age cracking (Kao, 2005).

2.1.10.2 Mix Design

A stronger concrete will have a higher tensile strength. Thus, lower water to cement ratios, higher cement contents, and other factors that are known to increase concrete compression strength will also increase the tensile strength. Unfortunately, these factors usually also increase shrinkage and thermal problems, so if trying to limit cracking, often it is not beneficial to increase the concrete's strength.

2.1.11 Corrosion

Corrosion is often a long term cracking problem. Much of the corrosion problems come from having existing cracks that allow ingress of water and salts. These cracks accelerate the corrosion problem, which increases the cracking problem.

2.1.11.1 Chloride Ion Penetration

Different types of concrete corrode at different rates, depending on the permeability of the concrete and the degree of passivation. Silica fume has been added to increase the density of the concrete, but many researchers indicate that silica fume increases sensitivity to curing procedures. If the concrete is cured properly, cracking can be avoided for the most part (Shing and Abu-Hejleh, 1999). Silica fume has a high heat of hydration, is sticky, and is expensive; these issues tend to negate the benefits in ion penetration (Xi et al., 2003).

2.1.11.2 Rebar Type

Epoxy-coated rebar has not been shown to reduce the corrosion problem in the field. In the lab, it performs well, but that is under ideal conditions. After handling in the field, the epoxy has shown both delamination and scratching. Epoxy-coated rebar recovered from failed structures often show delamination and corrosion problems. Epoxy rebar tends to localize the corrosion, increasing the rate of corrosion at those places. It has been shown that cracks tend to larger in bridge decks with epoxy-coated rebar (Krauss and Rogalla, 1996; Meyers, 1982; Linford and Reaveley, 2004).

Stainless-steel rebar does not corrode, but it is very expensive and has only been used by one Department of Transportation, Oregon's. Stainless-clad rebar seems to be a viable alternative, as it costs only some 50% more than standard rebar and shows significant resistance to corrosion.

2.1.12 Department of Transportation Opinions

Many papers have been published that include results of surveys on the causes of bridge deck cracking. In addition, many Departments of Transportation commissioned researchers to evaluate what the causes of bridge deck cracking were in their state. These causes may be mechanical, procedural, or a number of other things. A brief review of the surveys and opinions of the reports are presented here.

Krauss and Rogalla (1996) surveyed 52 agencies in the United States and Canada. Most of the respondents indicated that they considered early transverse cracking a problem; nearly all report extensive cracking on bridge decks. The agencies were requested to indicate what they thought to be the causes of bridge deck cracking. Table 2 gives the results of that question; the number in parentheses is the number of responses giving that cause.

Twenty agencies (out of fifty-two) consider improper curing to be a cause of cracking. Wind, thermal effects, and air temperature were each listed by seven agencies. These cannot be remediated easily, but correcting the curing problems should be a high priority. The most common materials problem cited was concrete shrinkage, with drying shrinkage specifically singled out. A few of the agencies also considered deflection design to be a reason for cracking in bridge decks.

Table 2: Causes of bridge deck cracking, agency survey (Krauss and Rogalla, 1996)

Construction	Materials	Design
improper curing (20) wind (7) thermal effects (7)	concrete shrinkage (17) [5 cited drying shrinkage specifically]	deflections (7) excessive cover (3) placement sequence (2)
air temperature (7) relative humidity (4) vibration (2) placement conditions/weather (2)	concrete mix design (7) plastic shrinkage (3) excessive cement (3) concrete temperature (3) use of retarders (2)	

In addition, Krauss and Rogalla ranked the causes of bridge deck cracking they evaluated from their own research and many other sources. Table 3 gives those findings. These findings are very similar to those discussed earlier throughout the analysis of the mechanical causes of cracking.

Table 3: Factors affecting bridge deck cracking (Krauss and Rogalla, 1996)

Factors	Effect			
	Major	Moderate	Minor	None
Design				
Restraint	✓			
Continuous/simple span		✓		
Deck thickness		✓		
Girder type		✓		
Girder size		✓		
Alignment of top and bottom reinforcement bars		✓		
Form type			✓	
Concrete cover			✓	
Girder spacing			✓	
Quantity of reinforcement			✓	
Reinforcement bar sizes			✓	
Dead-load deflections during casting			✓	
Stud spacing			✓	
Span length			✓	
Bar type—epoxy coated			✓	
Skew			✓	
Traffic volume				✓
Frequency of traffic-induced vibrations				✓
Materials				
Modulus of elasticity	✓			
Creep	✓			
Heat of hydration	✓			
Aggregate type	✓			
Cement content and type	✓			
Coefficient of thermal expansion		✓		
Paste volume—free shrinkage		✓		
Water-cement ratio		✓		
Shrinkage-compensating cement		✓		
Silica fume admixture		✓		
Early compressive strength			✓	
HRWRAs			✓	
Accelerating admixtures			✓	
Retarding admixtures			✓	
Aggregate size			✓	
Diffusivity			✓	
Poisson's ratio			✓	
Fly ash				✓
Air content				✓
Slump†				✓
Water content				✓
Construction				
Weather	✓			
Time of casting	✓			
Curing period and method		✓		
Finishing procedures		✓		
Vibration of fresh concrete			✓	
Pour length and sequence			✓	
Reinforcement ties				✓
Construction loads				✓
Traffic-induced vibrations				✓
Revolutions in concrete truck				✓

† within typical ranges

The Kansas Department of Transportation recommended that a silica fume overlay be used to decrease permeability. In addition, wet cure specifications were recommended. They used wet burlap for 7 days, and it cut cracking by 50%. Finally, they liked polymer overlays, but recommend a heavy grit blast (#6 or #7) (Xi et al., 2003).

According to the Utah Department of Transportation report (Linford and Reaveley, 2004), there are a number of factors causing cracking. Restrained shrinkage is listed as the most common cause. Issa (1999) suggests ten causes, listed in order of descending importance:

1. Inadequate concrete curing procedures which result in high evaporation rates and thus a high magnitude of shrinkage, especially in early age concrete.
2. The use of high slump concrete.
3. High water-to-cement ratios due to inadequate mixture proportions and retempering of concrete.
4. Insufficient top reinforcement cover.
5. Inadequate vibration of the concrete.
6. Deficient reinforcing details of the joint between a new and old deck.
7. Sequence of deck section placement.
8. Vibration and loads from machinery.
9. The weight of concrete forms.
10. Deflection of formwork.

The Utah Department of Transportation analysis of their bridges found that composite deck attachment to girders, bents, diaphragms, and abutments exacerbated the cracking problem, as it increased the restraint of the deck. Steel girders, as opposed to concrete girders, greatly increased the cracking problem; this is probably because of the differences in thermal expansion coefficients or the difference in thermal mass. Large concrete placements also increase cracking.

The Michigan Department of Transportation report (Aktan et al., 2003) included analysis of a database of inspections. They had several conclusions:

- More cracks were observed on the continuous bridges than the simple span bridges.
- Bridges with PCI (Precast Prestressed Concrete Institute) girders showed less longitudinal crack density than other girder types.
- More transverse and diagonal cracks were observed on bridges with adjacent box girders than other girder types.
- Map cracking was only observed on bridges with steel girders.

Xi et al. (2003) conducted an analysis of Colorado bridges for the Colorado Department of Transportation, and developed a list of recommendations as well. They recommended:

- Type II cement or Type I cement with increased fly ash.
- Cement content below 470 lb/yd^3 if possible.
- Water to cement ratio around 0.4.

- At least 20% Type F fly ash.
- Maximum 5% silica fume.
- May use ground granulated blast furnace slag to improve durability.
- Specify strength at 1, 3, 7, 28, and 56 days.
- Consider using permeability, drying shrinkage, and crack resistance tests as acceptance tests.
- Largest aggregate size possible and well graded aggregate to minimize cement paste volume.

In addition, they recommended a number of things regarding design factors, primarily aimed at minimizing restraint. For construction practice, it is recommended that the air temperature be between 45° and 80° F for batching, and generally to reduce evaporation however possible. They recommended 7 day continuous moist curing. (Shing and Abu-Hejleh,1999; Xi et al., 2003)

The Michigan report (Aktan et al., 2003) gives the responses of thirty-one Departments of Transportation in regards to the causes of bridge deck cracking. Each respondent was asked to give the three top causes of bridge deck cracking in their jurisdiction. Figure 12 gives the responses.

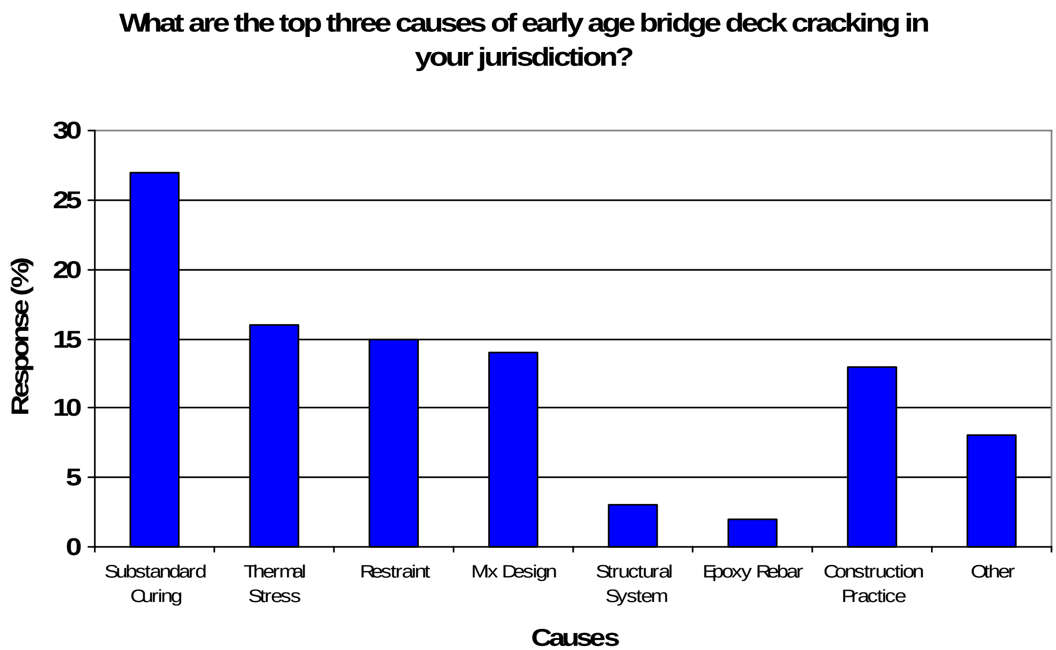

Figure 12: Frequency of top three causes of early-age bridge deck cracking (Aktan et al., 2003)

Research in the U. K. has indicated that their early age cracking problem is primarily due to restraint of early thermal movement, rather than restraint of shrinkage as previously thought (The Highways Agency, 1989). The researchers note that cracking has become more prevalent in recent years, as higher strength concretes have been implemented; higher strength concretes usually also produce more heat in the curing period. Thermal movement would be of little consequence if the member was unrestrained, but bridge decks are highly restrained by the beams on which they rest. In plain concrete, thermal cracking tends to yield a few wide cracks; minimal temperature reinforcement leads to more and smaller cracks.

Babaei (2005) reduced all the causes of bridge deck cracking to four central points: settlement of plastic concrete, thermal shrinkage of curing concrete, drying shrinkage of hardened concrete, and flexure. The causes for each of these mechanical processes are then identified and possible methods for reduction given.

Plastic settlement occurs as the concrete bleeds. Often, voids develop under transverse reinforcement bars where rising water collects, and a crack develops above, due to the restraint upon settlement at that location. Several factors promote this condition: shallow cover, a higher slump mix, and large reinforcement size. Babaei constructed a table showing the probability of cracking based on these conditions (Table 4).

Table 4: Probability of Plastic Shrinkage Cracking (Babaei, 2005)

	Probability of Cracking (percent)								
	2 in. slump			3 in. slump			4 in. slump		
Bar Size	#4	#5	#6	#4	#5	#6	#4	#5	#6
¾ in. cover	**80%**	**88%**	**93%**	**92%**	**99%**	**100%**	**100%**	**100%**	**100%**
1 in. cover	**60%**	**71%**	**78%**	**73%**	**83%**	**90%**	**85%**	**95%**	**100%**
1.5 in. cover	**19%**	**35%**	**46%**	**31%**	**48%**	**59%**	**44%**	**61%**	**72%**
2 in. cover	**0%**	**2%**	**14%**	**5%**	**13%**	**26%**	**5%**	**25%**	**39%**

Thermal shrinkage during curing is another major type of problem. The concrete cures at high temperature from the heat generated by hydration. It then cools, but is restrained from shrinking by the beams, causing stresses in the deck. Cracking thus occurs as the deck cools.

Babaei states that the difference between the deck and beam temperature contributes strain at the rate of about 5.5 microstrain/degree F. Creep cannot compensate, because the stresses are fully realized within a few days. A temperature differential of about 40 degrees F is enough to produce cracking without other factors; other factors such as drying shrinkage contribute to cracking with less temperature differential. It is best, therefore, to keep the differential to 22 degrees F or less. To do this, less cement, Type II cement, or retarders are recommended. In addition, precautions should be taken in cold weather.

Drying shrinkage cracking is the third type of problem addressed by Babaei. This occurs over long periods of time, on the order of a year. Assuming that creep is 50% of shrinkage, 400 microstrains of drying shrinkage would be needed to crack the concrete. An 8 to 9 inch thick deck can shrink up to about 550 microstrains, depending on the mix. The deck shrinkage is about 2.5x less than that of standard ASTM shrinkage prisms. Therefore, a reasonable parameter for maximum long term specimen shrinkage (assuming deck/beam thermal differential of 22F) would be about 700 microstrains. For 28 day shrinkage, that number would be about 400 microstrains.

There are several factors affecting drying shrinkage cracking mechanically. Aggregate mineralogy is one; porous, "soft" aggregate concrete can have shrinkage twice that of concrete with hard, non-porous aggregates. The type and source of cement also impacts shrinkage; it is best to use cement from a proven source, and type II if possible. If admixtures are used, it is important to test the mix beforehand in case unforeseen interactions occur. Finally, minimizing the water in the concrete is key.

The final primary cause of cracking in the opinion of Babaei is from flexure, particularly from unshored construction in continuous bridge decks. To minimize early cracking from this source, it is best to place the deck concrete in midspan first. This minimizes the movement in the area over the support after that section is placed. (Babaei, 2005)

It appears, then, that the causes of cracking are many and varied. Design, construction, and materials issues are all considered contributors. Many point to curing problems as a primary cause of cracking. A large proportion point to shrinkage problems associated with the mix design. A number of design issues seem to be neglected as well, though often designs are non-negotiable in most aspects. It seems that thermal problems are largely ignored. The number of causes is large, and a number of actions not common in construction could help reduce cracking.

2.1.13 Application in the Field

The Michigan Department of Transportation report (Aktan et al., 2003) gave an interesting commentary on the code and adherence to the code by the contractors. Construction monitoring of projects was conducted to see whether contractors adhered to the MDOT Standard Specifications for Construction. There were a number of areas that did not meet the standards:

- Freefall of the concrete was often more than 6 inches.
- Vibratory compaction was often not done within 15 minutes of placing, as concrete delivery delays sometimes exceeded 30 minutes.
- Vibrators were not used in a pattern, but rather randomly. Vibrators seemed to be used to move concrete into place.
- Curing was applied for 7 days, but proper precautions were not taken to ensure it was a wet cure operation (which was required).
- Curing compound was applied very late, rather than immediately after bleed water had left. Sometimes the entire deck was placed before curing compound was applied.
- Far more than the maximum of 10 feet of textured concrete were left exposed without curing compound.
- Burlap was not applied until the next day, and then not properly wetted. It was supposed to be placed as soon as the concrete surface could support it, not more than two hours after pouring.
- Proper procedures for keeping burlap wet were not followed; no soaker hoses were used.
- The expansion joint boundaries are problematic. Excess concrete overflows, loses its plasticity, and is scraped off and thrown in with the deck concrete near the joint. Concrete that falls off the joints should not be placed back on the deck. (Aktan et al., 2003)

Thus, it appears that even if the departments of transportation have appropriate specifications in place for curing and other construction issues, these specifications are not always followed. In design, deck cracking problems are generally ignored as a design parameter. Concrete mix designs are usually created to maximize strength and other parameters such as freeze-thaw resistance, but shrinkage and crack resistance are generally relegated to secondary consideration.

2.1.14 Summary/Conclusion

The United States has a vast bridge deck cracking problem, which has grown in recent years with the increasing use of high strength concrete and the commonplace usage of composite girder/deck designs. There are several key improvements that can help improve the cracking problem.

This literature review has discussed the mechanics of bridge deck cracking. Many causes of bridge deck cracking were identified, but not all are under the control of the engineer. Figure 13 attempts to illustrate the areas where the engineer has good control of the causes of cracking. Many aspects of the bridge design are controlled by the geometry and loads, so the engineer has only minimal control. Some areas, like the thermal movement, are environmental conditions. There are several key areas where the engineer has good control: plastic and drying shrinkage of the concrete deck, the restraint in the deck provided by fibers, and the rebar type used. With these, and making good choices where only moderate control is possible, cracking can be controlled.

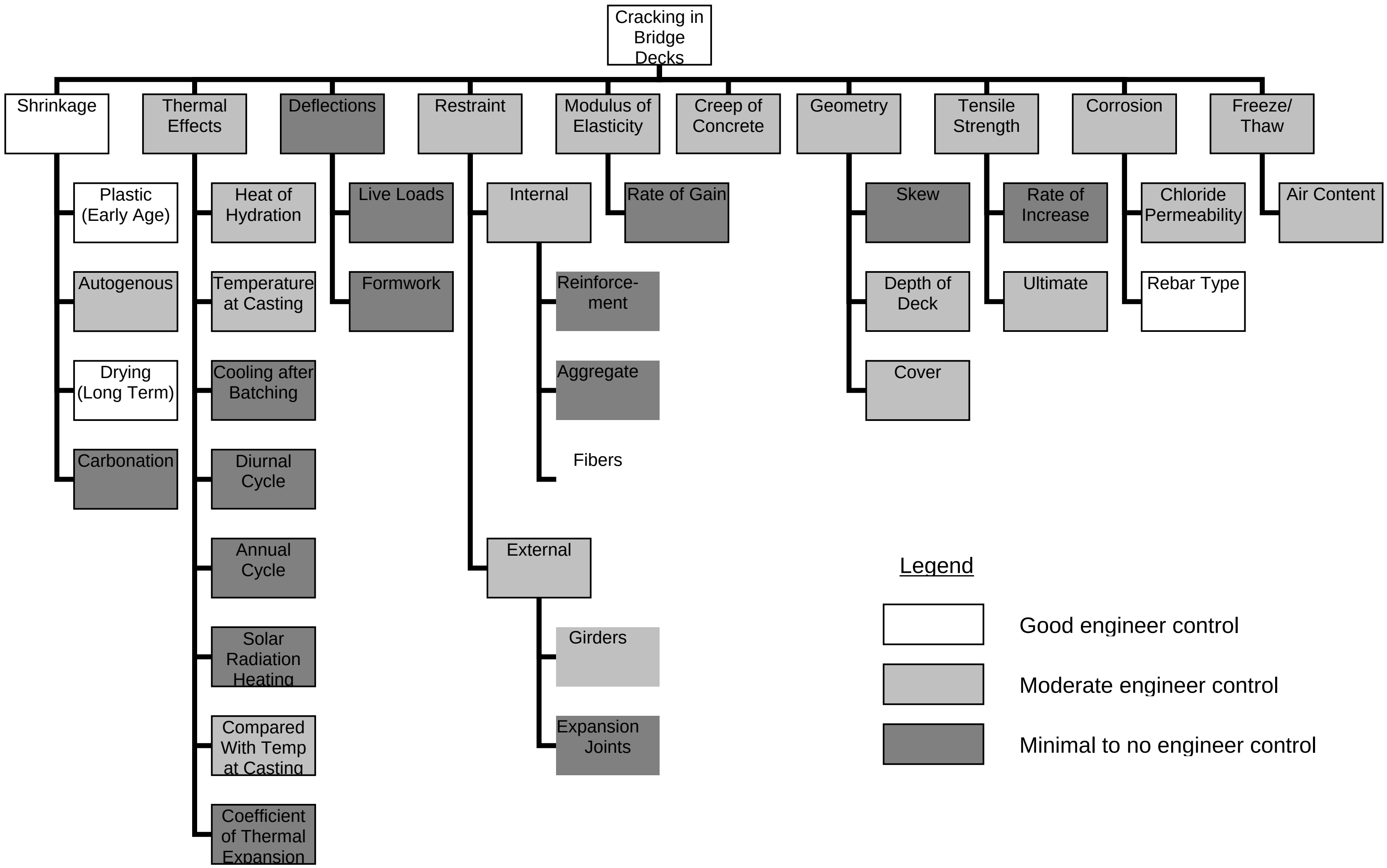

Figure 13: Factors affecting cracking in bridge decks: level of engineer control

There are several areas of the mechanics of bridge deck cracking that can be controlled. How is this control exerted by the engineer? There are several methods for reducing bridge deck cracking that have been identified by this literature review.

The most important method for reducing the cracking problem is to implement a true 7-day wet curing system for all bridge decks, including wet burlap with, perhaps, plastic sheeting over it. It is known that such a specification exists in many states, but often the implementation of the procedure is lax. The system needs to be put in place promptly and measures need to be taken to ensure the burlap remains wet for the full curing time. Using this method has already been very successful where it has been truly carried out. In general, good finishing and placement procedures need to be carried out at every site; methods for ensuring contractor compliance should be enacted.

The next way to reduce cracking is to revamp the concrete mix designs. To reduce the shrinkage and thermal problems, the concrete mixes need to have less cement content. A higher aggregate content is recommended, associated with a moderate increase in the water to cement ratio to about 0.4. Using larger aggregate, and a better gradation, can help maintain workability while reducing the cement content. It is also recommended that a small amount of fly ash be used, to act as a mild set retarder and to reduce the heat of hydration. Shrinkage should be looked at as a primary constraint in the selection of concrete mix designs. High strength of concrete is not as important or as difficult to achieve as getting a crack-free bridge deck.

Appropriate thermal controls should be adhered to, but the actual setting temperature is not overly important in the concrete's subsequent behavior. Greater problems are associated with the daily temperature swings and solar radiation, which cannot be controlled.

Epoxy-coated rebar usage should be re-evaluated; its benefits in the field seem to be negligible, and significant problems in cracking have been associated with its use.

Fibers need to be considered as a useful tool in both reducing cracking and the severity thereof, and in early-age tensile strength and shrinkage response. The next section of the literature review will consider polymer fiber properties in greater depth.

2.2 Fiber-Reinforced Concrete

Polymer fibers are one of the innovative approaches being taken to improve the behavior of concrete. Plain concrete is a brittle material, a poor characteristic for a structural building material. In addition, it has problems with cracking and shrinkage. All three of these characteristics may be improved with the addition of polymer fibers.

Fibers have substantial effects on most properties of concrete; these effects have been studied in numerous papers, which this literature review investigates. Polypropylene fibers, which this project studied, differ substantially from other types of fibers in their affect on concrete behavior. Since polypropylene fibers are made of a material with a comparatively low modulus of elasticity, they do not have much effect on the properties of the concrete until cracking. However, they do have a substantial impact on the concrete behavior during curing, while the concrete is still weak.

2.2.1 Fiber Material Properties

There are several properties that a good reinforcing fiber must have to be effective: tensile strength, ductility, high elastic modulus, elasticity, and Poisson's ratio. Several are key to the mechanical behavior of fiber-reinforced concrete.

To see significant improvements in tensile capacity of concrete, the fiber must be much stronger than the concrete matrix in tension, since the load bearing area is much less than the matrix. For ductility improvements, the fiber must be able to withstand strains much greater than the matrix. Fibers subject to creep have a reduced effectiveness.

The most important, though, is the elastic modulus. The proportion of the load carried by the fiber depends directly upon the comparative elastic modulus of the fiber and matrix. If the elastic modulus of the fibers is less than that of the concrete matrix, the fibers will contribute relatively little to the concrete behavior until after cracking. In addition, the composite strain after cracking will be higher. This is a primary problem afflicting polymer fibers: a relatively low elastic modulus. (Johnston, p. 25-26)

Zhang and Li (2001) did extensive theoretical work modeling the influence of fibers on drying shrinkage. There is not a simple linear relationship between the moduli ratio and the shrinkage; however, the moduli ratio does have a significant effect.

> Increasing the fiber modulus or reducing the matrix modulus can raise the efficacy of fibers with respect to the restrain on the matrix shrinkage. Based on this result, it can be concluded that high modulus fibers, such as steel and carbon fibers, are more effective than low modulus fibers, such as polypropylene and polyvinyl alcohol fibers, in reducing the matrix shrinkage under the same fiber content and fiber geometry. In addition, fibers in immature cementitious matrix are more effective on the restraint to the matrix shrinkage than that in the

matured matrix due to the difference in the matrix elastic modulus. (Zhang and Li, 2001)

2.2.2 Workability

The workability of fiber-reinforced concrete is a major issue. The primary factors deciding the level of workability are the paste volume fraction, the fiber dosage rate, and the fiber aspect ratio. Typically, fibers decrease slump, but this does not necessarily make fiber mixes harder to compact with vibration. Fibers do make mixes somewhat drier due to their high specific surface area.

Johnston (p. 11-13) gives the results of Pfeiffer and Soukatchoff, who did tests regarding the affect of paste volume fraction on workability. They assessed slump in terms of paste volume fraction and fiber content by volume. Their work was with steel fibers, but the results are likely to be qualitatively similar to what is seen in polymer fibers. Figure 14 gives these results.

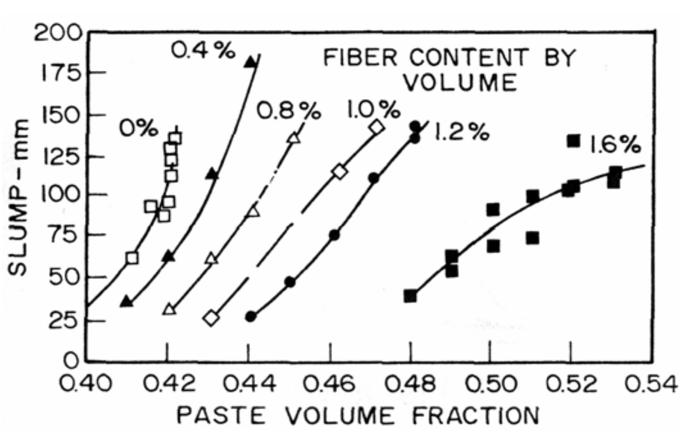

Figure 14: Effect of paste volume fr action on wo rkability of steel fiber- reinforced mortars with 30 mm fibers (Johnston, after Pfeiffer and Soukatchoff, 1994)

Johnston (p. 11) also gives the results of Edgington, Hannant, and Williams, who did tests correlating the steel fiber aspect ratio to workability. In their tests, they assessed vibration time required for placement compared to fiber aspect ratio and volume. For each aspect ratio, there was a distinct limit beyond which an increase in fiber content caused a dramatic decrease in workability.

Balaguru and Khajuria (1996) obtained similar results on the slumps of mixes with fibers; their work with polymeric fibers showed slumps decreasing with increasing fiber dosage levels. With plain concrete they had a slump of 8.9 inches; at the highest dosage rate, about 4 lb/yd^3, they had a slump of only 1.6 inches. However, the decreased slump did not result in a similar increase in the difficulty of vibratory compaction.

Kao (2005) worked with polymer fibers, and found a strong reduction in the slump of a concrete with the addition of the fibers. This trend depended somewhat on the type of fibers; the smaller the fiber, the more rapidly the slump decreased.

2.2.3 Early Age Shrinkage

Polymer fibers decrease early age unrestrained shrinkage, according to Ramseyer (1999), but the magnitude and effectiveness of shrinkage reduction is poorly understood. Filho and Sanjuan (1999) did work on early age shrinkage and polypropylene fibers. Their findings indicate a reduction of about 20% (from 2700 to 2000 microstrain) in early age shrinkage with a 0.2% polypropylene fiber mix.

However, Altoubat and Lange (2002) found a slight increase in shrinkage with the addition of fibers. This result seems to be based upon a test normalized at 12 hours, so the earliest shrinkage behavior is omitted; this could cause the discrepancy.

Kao (2005) investigated the early age shrinkage properties of polymer fiber-reinforced concrete extensively. Kao found that the early age shrinkage (at less than 24 hours) was greatly reduced by the addition of fibers. Each fiber described a curve: the shrinkage decreased with increasing dosage of fibers up to a point, and then increased as more fibers were added. The optimum dosage varied, but with all fibers, reduction of at least 50% of the early age shrinkage was realized.

Theoretically, the fibers should have a more significant effect on shrinkage at early age, due to their relatively higher modulus of elasticity at that point. Zhang and Li (2001) calculated this in their work on the modulus of elasticity, mentioned earlier.

2.2.4 Long Term Shrinkage

Whether polymer fibers have a significant effect on shrinkage after final set is a controversial issue. Steel fibers do seem to decrease shrinkage. Zhang and Li experimentally verified their mechanical work using a number of steel fiber mixes. These were normalized at one day. They all showed significant decreases in shrinkage in the steel fiber mixes, in keeping with the calculated predictions (Zhang and Li, 2001). There are a number of studies indicating that steel decreases long-term shrinkage. Polymer, however, is another matter.

In a study of high-performance cements, the addition of a polyethylene fiber had absolutely no effect on the free shrinkage behavior (Lim et al., 1999). Altoubat and Lange (2002) found that the polypropylene fiber actually increased the shrinkage somewhat; they theorized that this was because the fiber prevented microcracking at the surface from relaxing the stress.

Kao (2005) analyzed the long term shrinkage behavior of a variety of fibers and dosage rates. He found a slight decrease in the unrestrained shrinkage with the addition of fibers, but the dosage rate did not matter much.

2.2.5 Compression Strength

The compression strength of concrete has been shown to be only slightly affected by the addition of fibers, except at very early age, under 24 hours. This is due to the fact that polymer fibers have a lower modulus of elasticity than does concrete once the concrete cures. Thus the fibers do not take load until the concrete cracks. However, at early age, the concrete has a lower modulus of elasticity, and the fibers take load.

Ramseyer (1999) found that in high-early strength concrete, 3 lb/yd^3 of Stealth fibers increased strength at early age (under 24 hours), but long-term effects were not consistently observed.

Balaguru and Khajuria (1996) tested both normal and lightweight concrete with polymeric fibers up to about 4 lb/yd^3. They found that the addition of fibers did not change the compressive strengths appreciably long term; the variation was within the experimental variation expected in concrete.

Aulia (2002) in testing a number of aggregates and mixes with polypropylene fibers found that "the use of 0.2 vol.-% polypropylene fibers alone resulted in the low influence on both the compressive strength and modulus of elasticity of concrete…." Essentially, there was no difference between the compressive strength with and without fibers.

Soroushian et al. (1992) found an interesting trend. With the addition of more fibers, the compressive strength significantly decreased. The plain concrete had a strength of about 6700 psi, while the average strength with fibers decreased with higher dosage rates to about 5200 psi at a 0.1% by volume dosage. It must be noted that when Soroushian et al. added fibers they also added a small amount of superplasticizer.

Kao (2005) also found a slight decrease in compression strength at 28 days. However, at 1 day, the strength of the fiber-reinforced concrete was usually equal to or higher than the plain concrete control.

2.2.6 Tensile Strength

One would think that adding fibers to concrete would increase the tensile strength of the concrete since the tensile strength of concrete is so low. However, the modulus of elasticity of the polymeric fibers is less than that of the concrete matrix, so the fibers do not take much load until cracking. Once cracking occurs, sometimes the tensile strength of the fibers bridging the crack is higher than that of the concrete, causing the ultimate tensile strength to be reached after cracking, when the fibers alone provide the strength. However, this does not actually increase the cracking strength of the mix. Ductility is obviously greatly increased.

Balaburu and Khajuria (1996) also tested the splitting tensile strength of lightweight concrete with polymer fibers. The strengths were not appreciably different at 28 days; they were slightly higher at 7 days. However, the difference was not statistically significant. A major difference was that after failure, the fiber cylinders maintained their coherence, while the plain concrete cylinders fractured into two pieces.

Kao (2005) found moderate increases in tensile strength at early age with the addition of fibers, but long term there was no significant benefit. This agrees with what could be expected based upon the moduli of elasticity of the fibers and of the concrete.

2.2.7 Flexure

In keeping with the effects commonly found for polymer fibers on compressive and tensile strength, the bending strength is not substantially affected by the addition of fibers. This is again primarily due to the low modulus of elasticity of the fibers. However, after cracking, the fibers come into play, and permit a greatly increased ultimate strain, though the load carrying capacity is decreased.

Balaguru and Khajuria (1996) tested the modulus of rupture fiber-reinforced samples, and found that the strength did not change appreciably.

Soroushian et al. (1992) studied the flexural strength of fiber-reinforced mixes. They found a moderate increase in the flexural strength with the addition of fibers, increasing with higher dosage rates of fibers. The plain concrete mixes had a strength of about 620 psi, while the highest dosage of fibers yielded a strength of about 740 psi in flexure.

Li (2002) noted a moderate improvement in bending strength with the addition of fibers, but stated that the major difference was in the behavior after reaching the ultimate load. Instead of brittle failure, the fiber mix showed somewhat ductile behavior, with ultimate deflection four times that of the plain concrete.

2.2.8 Modulus of Elasticity

Since the polymer fibers have a lower modulus of elasticity than the concrete matrix itself, it would be reasonable to assume that the addition of fibers has little effect upon the overall modulus. This assumption has been experimentally confirmed: Aulia (2002) found no significant variation in the modulus of elasticity of the concrete with the addition of fibers.

2.2.9 Failure Types

Fibers greatly enhance the ductility of concrete; failures normally brittle are now ductile, due to the fibers' crack bridging capability. It has long been known that fibers cause failures to exhibit completely different behavior. Instead of a complete and sudden fracture of the specimen, the specimen behaves much more ductily, with numerous small cracks developing before the specimen refuses to take more load. Aulia (2002) notes that the crack bridging and material interlock created by the fibers led to stable fracture processes, and hence higher fracture energy. A discussion of the fracture mechanics in fiber-reinforced materials may be found in Gordon (p. 189). Aulia's stress-strain curves showed more inelastic deformation before the ultimate load was reached, as microcracking developed.

Ramseyer (1999) noted that there was a lack of brittle fractures in fiber-reinforced concrete; instead, the specimens tended to fail under load, redistribute the load, and then accept more load. The failures tended to be very ductile.

2.2.10 Fibers as Crack Inhibitors

Fibers are commonly used to reduce cracking, particularly in slabs for structures. The cracking behavior of polymer fiber-reinforced concrete has been studied extensively. In particular, the time to crack and post-crack behavior have been analyzed.

It is a question whether the perceived advantages of fibers in crack inhibition translates to results in the field. For example, Brooks (2000) investigated a pair of bridges in Oregon, with one bridge using polypropylene fibers, and the other not. In this case, the bridge deck with fibers actually exhibited worse cracking than the plain concrete deck.

2.2.10.1 Crack Width and Time to Cracking

Aulia notes that "due to their high tensile strength and pull-out strength, the polypropylene fibers even could reduce the early plastic shrinkage cracking by enhancing the tensile capacity of fresh concrete to resist the tensile stresses caused by the typical volume changes…. All cracking stresses are sustained by the fibers" (Aulia, 2002).

Lim et al. (1999) studied the crack width development of concrete with and without fibers. In this case, the polyethylene fibers were in a very high shrinkage high performance mix. Utilizing a restrained shrinkage system, the mix without fibers cracked within 24 hours and had an 1100 micrometers crack width on the first day. This crack reached 11,000 micrometers at 20 days. The fiber mixes, however, did not have the same behavior. Instead of developing only one crack, they developed around 20 cracks each, with the maximum size being only about 150 micrometers after 50 days. This shows the tremendous effect of fibers on cracking.

2.2.10.2 Impact Resistance

Fibers have been shown conclusively to increase impact resistance greatly. Both time to first crack and time to failure are greatly increased; higher dosage rates lead to higher values for both (Soroushian et al., 1992). A large amount of energy is absorbed in debonding, stretching, and pulling out of the fibers after the concrete has cracked. Even before visual cracking, there seems to be a small increase in the impact toughness. (Hannant, p. 94-95)

Balaguru and Khajuria (1996) tested lightweight concrete at 28 days for impact resistance. Plain concrete cracked within the first 4 blows, while samples with fiber took somewhat longer. The greatest contribution, however, occurred after the first crack. The plain concrete totally failed within 5 blows, while the fiber concrete did not fail until after at least 9 blows, and usually more.

2.2.11 Fiber-Reinforced Concrete: Conclusion

Polymer fibers are good for several applications, but not others. Due to their relatively low modulus of elasticity, they have the most significant effect at early age and after cracking.

At early age, fibers decrease shrinkage significantly, and decrease cracking as well. Generally, at early age, all strength parameters are improved. However, after curing, the fibers no longer have an impact on compressive strength, and flexure and tensile tests show only slight improvements. Long term shrinkage similarly shows no major benefit.

After cracking, the fibers are again beneficial. Ductility is substantially increased, as failures are no longer brittle. Crack widths are greatly decreased, and impact resistance greatly increased.

Polypropylene and polyethylene fibers, then, are useful when early age properties need to be improved, or when ductility is important.

2.3 Literature Review: Conclusion

The review of bridge deck cracking reveals that many of the causes of the cracking are associated with movement of curing concrete. Polymer fibers have been shown to greatly reduce the movement of plastic concrete. Thermal movement, early age shrinkage, and early age settlement all could be improved substantially by the addition of polymer fibers.

Thermal movement occurs in fresh concrete, where the expansion due to heat is locked into the matrix when the concrete cures. As it cools, stresses are imparted to the matrix. However, if fibers were present, the initial expansion due to the heat of hydration would be greatly limited by the network of fibers.

Early age shrinkage is likewise restrained by the polymer fibers, as at that time the modulus of elasticity of the fibers is greatly in excess of the concrete matrix. Research has shown that early-age shrinkage is reduced by fibers.

Early age settlement would likewise be reduced. The addition of fibers always reduces the slump, preventing movement of the fresh concrete. As was seen in Table 4 (Babaei, 2005), reducing the slump greatly reduces the incidence of early age settlement cracking.

In addition, the behavior of the bridge decks after cracking could be greatly improved by adding fibers. Crack widths could be greatly reduced.

It appears, then, that adding polymer fibers to bridge deck mixes would be beneficial in a number of areas; many of the worst problems of concrete bridge decks could be significantly helped by the addition of polymer fibers.

Chapter 3: Research Scope

The research that was conducted focused on the shrinkage properties of fiber-reinforced concrete, both long term and at early age. The matrix tested is an extension of that tested by Kao (2005), with four new fibers and several higher dosage rates.

The primary objective of this research was to evaluate the fibers' usefulness in controlling bridge-deck cracking. To study this, tests were selected that focused on the shrinkage behavior of the concrete. The primary tests included unrestrained shrinkage, compression strength, splitting tensile strength, and a new test (first used by Ramseyer, 1999, modified by Kao, 2005), unrestrained shrinkage from time zero.

The mixes were all based on the Oklahoma Department of Transportation (ODOT) Type AA typical with fly ash mix. The only modification to the mixes was the addition of the fiber, and the removal of a corresponding volume of sand to compensate.

The fiber dosage rates were set at high levels, compared to those typically used for microfibers. It was hoped that the limits of the fibers' usefulness would be reached and the point at which the improvement of the mix diminished located for each fiber. The matrix used consisted of one, three and five pounds per cubic yard dosage rates, as those levels had given good results in previous research (Kao, 2005). The eight pound per cubic yard dosage was removed from the matrix, as the same research indicated that dosage was too high for microfiber mixes, as workability became a major issue, and shrinkage increased over the five pound dosage rate. For the macrofiber mixes, much higher dosage rates were possible without loss of workability, so ten and fifteen pounds per cubic yard dosages were tested as well, to evaluate the limits of the fiber usefulness.

3.1 Tests

Each batch of the matrix had the same set of tests run on it. The fresh concrete tests performed were the slump test, air content test, temperature, and unit weight. The tests that were run included compressive strength, tensile strength, unrestrained shrinkage, and unrestrained shrinkage from time zero.

3.1.1 Fresh Concrete Tests

Several environmental conditions were measured at the time of batching, in addition to several fresh concrete tests being run. The air temperature and humidity were tested with a combined thermometer/hygrometer device. The concrete temperature was measured with a probe thermometer.

The unit weight and air content of the mixes were measured with a pressurized air pot. The pot was weighed, filled with concrete, and weighed again. Using this data, and the fact that the pot was 0.25 cubic feet in volume, the unit weight was measured. The air content was measured according to ASTM C231. Figure 15 shows the air content pot apparatus.


Figure 15: Air content pressurized air pot apparatus

The slump test was carried out according to ASTM C143. Figure 16 shows the slump cone apparatus in use, before finishing.


Figure 16: Slump test apparatus

3.1.2 Compression Strength

The compressive strength of the concrete was obtained using the procedures in ASTM C39. Generally, twenty-five cylinders of concrete were cast in 4x8" plastic cylinder molds. These were greased with diesel prior to batching to facilitate the samples' removal. The molds were removed at about one day after batching, and the first samples broken. Three cylinders were broken at each testing time, unless there were not enough samples or one of the samples failed as a result of an obvious defect, in which case the result was thrown out. The cylinders were tested in a Forney compression testing machine; neoprene caps set in metal plates were used to provide an even loading surface. The load was applied at a rate between 16,000 and 38,000 pounds per minute. These tests were run at 1, 7, 14, and 28 days. Figure 17 shows a compression test setup.

Figure 17: Compression test with Forney compression testing machine

3.1.3 Tensile Strength

Tensile strength of the concrete was found using the splitting tensile test, ASTM C496. Half of the cylinders batched were used for this test, three at each testing time. These tests were also run at 1, 7, 14, and 28 days. The Forney machine was again used, but the loading apparatus was changed. One-inch-wide strips of a thin fiberboard material were cut to provide a yielding bearing surface for the cylinders. One of these strips was placed on a steel plate on the bottom loading platen, and taped down to prevent movement. The cylinder was then laid down on the strip. Another plate with a strip of the wood was placed on top of the cylinder, with the strip resting along the cylinder and the steel plate spreading the load from the upper loading platen to the strip and cylinder. The load was applied at a rate between 5,000 and 10,000 pounds per minute until the cylinder split in half. Figure 18 shows a splitting tensile test at completion.

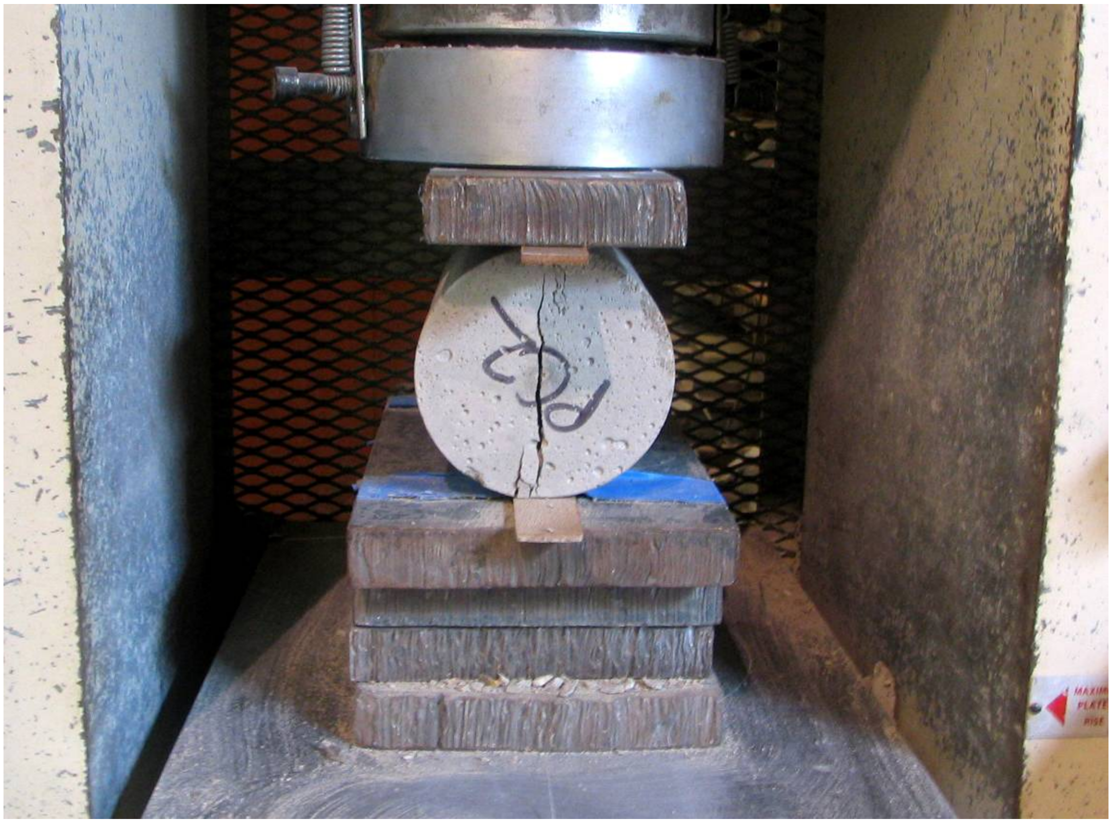

Figure 18: Splitting tensile test

3.1.4 Unrestrained Shrinkage

This shrinkage test was performed according to ASTM C490. Molds 3”x3”x10” were prepared by coating them lightly with diesel, and set screws were placed in the ends. Concrete was cast in the molds, and allowed to cure for twenty-four hours. The molds were then removed, leaving concrete prisms with studs at each end, 10” apart. These were measured at 1, 3, 7, 14, and 28 days. The one day reading was considered the zero value, and the shrinkage of the prisms compared from there. The system is accurate down to $10x10^{-6}$ strain; it measures to 10^{-5} inches on a 10 inch prism. Figure 19 shows the unrestrained shrinkage testing apparatus.

Figure 19: Unrestrained shrinkage test

3.1.5 Unrestrained Shrinkage from Time Zero

This test does not have an applicable ASTM standard, as it was developed at Fears Lab, with the initial design found in Chris Ramseyer's master's thesis (Ramseyer, 1999). Additional modifications were made by Jen Teck Kao (Kao, 2005). Further adjustments were made to the design for this project.

The apparatus tests a prism of concrete 3x3x10 inches, to permit direct comparison with results from the standard (ASTM) unrestrained shrinkage test. The concrete is restrained on one end by being cast around a bolt head, but is free to move on the other end. That end is cast around another bolt, but this bolt is anchored in an unrestrained sliding Teflon plate. The movement of this plate is then measured by a micrometer. See Appendix 1 for a full design of the device. Figure 20 shows the unrestrained shrinkage from time zero test in progress, after the side molds have been removed at 1 day.

Figure 20: Unrestrained shrinkage from time zero test in progress

The procedure for this test is thus:

1. Side plates are bolted onto the mold.

2. The mold is greased heavily with axle grease.
3. A thin sheet of plastic is placed over the grease, ensuring that the concrete cast inside will be completely free of restraint.
4. Bolts are screwed into the two ends of the mold, exactly ten inches apart, so that the unrestrained length of the concrete will be ten inches.
5. The concrete is cast in the mold.
6. The micrometer is then set up, bearing on the end Teflon plate.

Figure 21 shows time zero molds prepared for filling with concrete. The micrometer's needle will go through a hole in the foam block to bear on the white Teflon block.

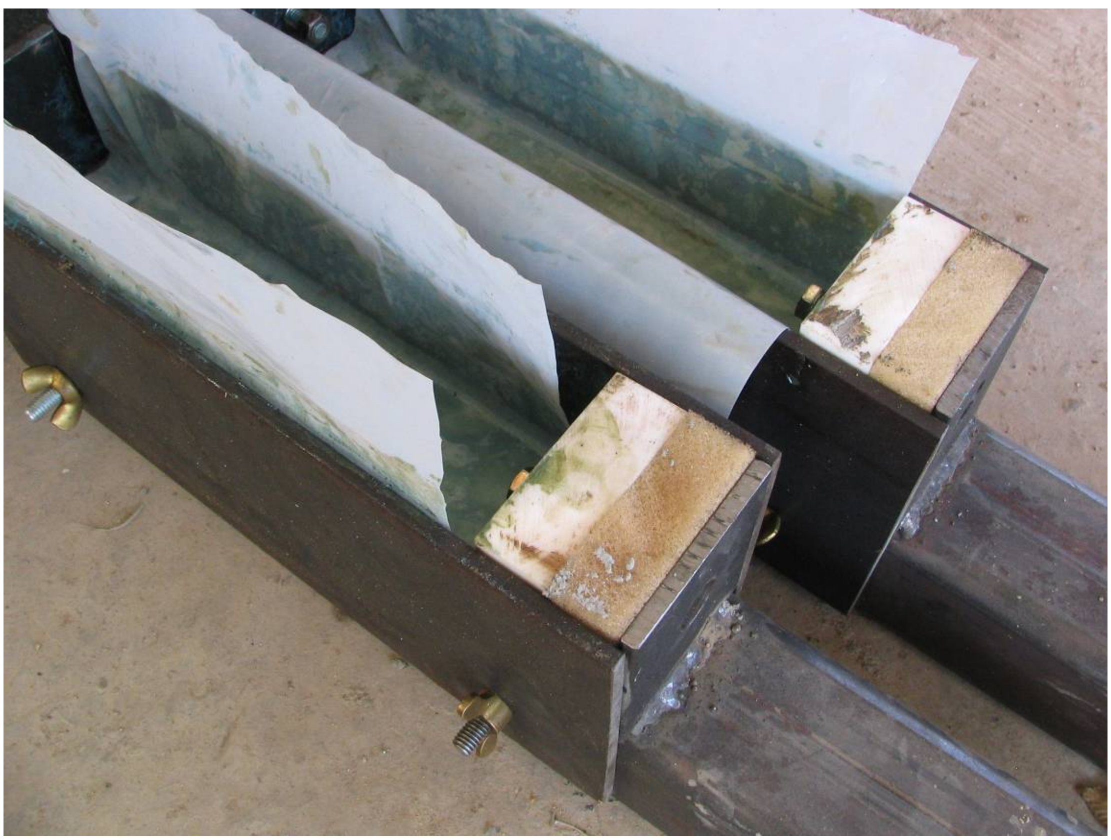

Figure 21: Time zero molds prepared for filling

The first reading is taken immediately, and then readings are taken every hour for the first six hours. The next reading is taken at one day, and additional readings later as desired, to compare with the readings of the standard unrestrained shrinkage test. The side molds are removed at twenty-four hours, to simulate the conditions in the unrestrained shrinkage test. Like the ASTM unrestrained shrinkage test, the system is accurate to $10x10^{-6}$ strain.

Readings were taken at time zero; 1, 2, 3, 4, 5, 6, and 24 hours; and 3 and 7 days. Several tests were run out to 28 days to provide data for comparison with the unrestrained shrinkage test.

This test yields excellent data on shrinkage from the batching time, quantifying the movement in concrete at early age. Several tests were run, comparing the shrinkage values of this time zero test with those of the normal ASTM unrestrained shrinkage test. There was a strong correlation, but the fact that there was only one time zero mold per batch may have made that data more inconsistent. Only one time zero mold was used per batch because the number of micrometers necessary to run many tests at once would have been quite expensive.

An objective of this research was to analyze this test, and to obtain data on whether it correlated with the ASTM unrestrained shrinkage test. This test provides shrinkage information at early age—the ASTM unrestrained shrinkage test (ASTM C490) ignores the first 24 hours. There were several possible issues with this test: the micrometer is vulnerable to being bumped, throwing off the results, and the level of restraint provided by the base and sides is unknown. It was hoped that the tests run here would help determine how viable this test is for more widespread use.

3.2 Matrix

The matrix tested had two variables: type of fiber and dosage rate of fiber. The matrix had four types of fiber, three to five dosage rates depending on the fiber, and one plain concrete control mix, for a total of nineteen mixes. Several batches were tested twice due to bad results or testing conditions.

The matrix was developed to investigate a number of different polymer fibers at several different dosage levels (Table 5). The objective was to find the limits of practical dosage levels for each fiber, both from a workability standpoint and from a performance standpoint. The microfibers had lower maximum dosage levels before the mixes became unworkable. The macrofibers maximum dosage levels were more determined simply by the fact that there was no benefit seen for higher dosage levels; the dosage could have been taken to a higher level, but the mix would likely not have shown improvements in any useful metrics.

Table 5: Primary matrix

Fiber	Dosage Rates (lb/yd^3)
Fibermesh Stealth	1, 3, and 5
Grace Microfiber	1, 3, and 5
Strux 90/40	1, 3, 5, 10, and 15
High Performance Polymer (HPP)	1, 3, 5, 10, and 15
Plain Concrete	No fiber

The testing regimen for the primary matrix was chosen to provide a good survey of the fibers' impact on the concrete properties, with a focus on shrinkage and early-age performance. Table 6 presents the testing regimen. The five control tests were all fresh concrete tests, chosen to make sure that the mixes were similar enough in batching conditions for comparison and to serve as a way to identify mixes that had anomalous behavior. The primary tests were the actual objectives of the testing.

Table 6: Primary matrix testing regimen

Control Tests and Readings	ASTM Standard
Air Content	C-231
Slump	C-142
Unit Weight	C-138
Concrete Temperature	---
Air Temperature and Humidity	---
Primary Tests	
Unrestrained Shrinkage from Time Zero	---
Unrestrained Shrinkage (ASTM)	C-490
Compression Strength	C-39
Splitting Tensile Strength	C-496

3.3 Fibers

The four types of fiber used in the primary matrix were Strux 90/40, Stealth, Grace Microfiber, and HPP. Each of these had distinct properties; the Strux and HPP were macrofibers, and tended to impede the finishing process. However, due to their fairly low surface area per pound, they did not significantly dry out the mix. The microfibers, Grace and Stealth, were much easier to finish, but did decrease the free moisture in the mix significantly.

All of the fibers used are synthetic polymers--either polypropylene, polyethylene, or a blend. Therefore, the fibers all have a modulus of elasticity below that of cured concrete, limiting the fibers' effect to before final set and after cracking. However, these are the two most problematic areas in concrete: shrinkage cracking and associated problems, and lack of ductility after cracking.

3.3.1 Stealth

Fibermesh Stealth is manufactured by SI Concrete Systems; it has since been replaced by Stealth e3 which was renamed Fibermesh 150. Stealth is a microfiber; the fibers range from ¼" to ¾", but are very small diameter. They are made out of polypropylene, with a modulus of elasticity of 5×10^5 psi. The recommended minimum dosage is 0.75 lb/yd; no upper limit is recommended by the manufacturer. The mixes tested have dosages significantly above this level.

Figure 22: Stealth Microfibers

3.3.2 Grace Microfiber

Grace Microfiber is a product of Grace Construction Products. As the name implies, the fiber is very small; there are over 50 million fibers per pound. The fibers are 20mm long and created of polypropylene, with a modulus of elasticity of $5x10^5$ psi. Grace recommends a dose between 0.5 and 1 pounds per cubic yard. Again, this fiber was tested at dosages well beyond this level. This fiber was specifically created to prevent cracking within the first 24 hours.

Figure 23: Grace Microfibers

3.3.3 Strux 90/40

Strux is a coarse fiber produced by Grace Construction Products. It is primarily intended to provide crack control. The fibers are created of a synthetic polypropylene/polyethylene blend. The fibers themselves are about 1.5 inches long, have an aspect ratio of 90, and a modulus of elasticity of 1.378×10^6 psi, according to the manufacturer. Grace recommends a dosage between 3.0 and 11.8 lbs/yd^3, so the dosage rates used in this research fully bracket that range.

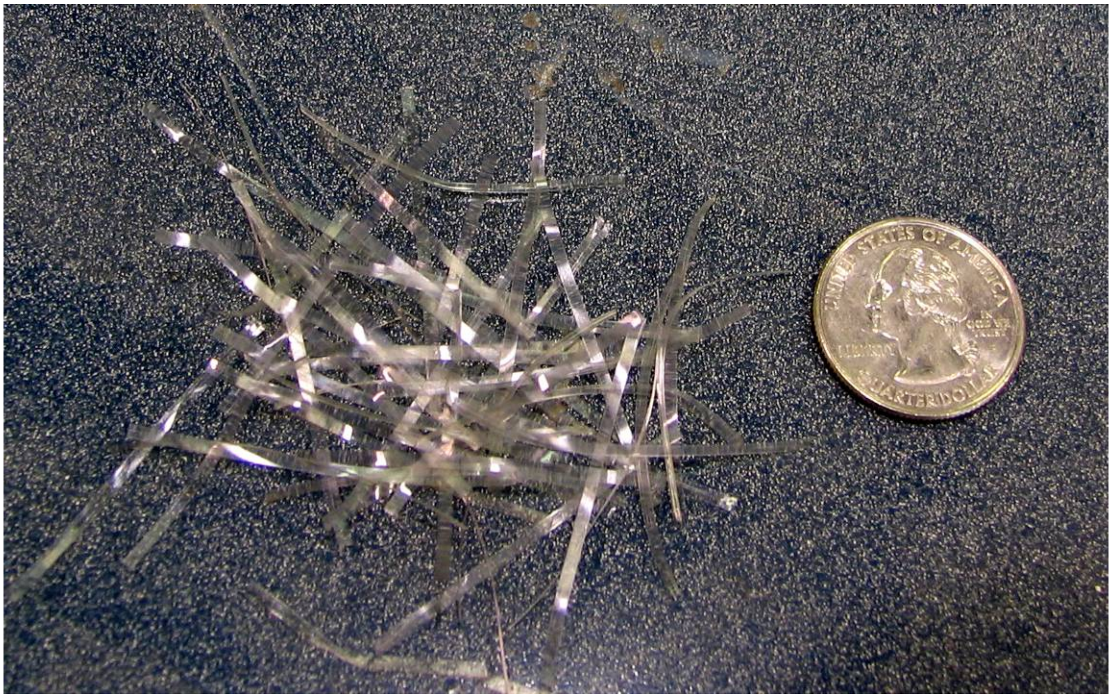

Figure 24: Strux 90/40 Fibers

3.3.4 High Performance Polymer (HPP)

This is a large and stiff fiber produced by SI Concrete Systems. It has since been replaced by the Enduro 600. HPP is 2 inches long, and is a macroscopic fiber, created out of polypropylene, with a modulus of elasticity of $5x10^5$ psi. The fiber is considerably thicker than others tested, about 1/20" by 1/30". The fiber is formed in a sinusoidal wave pattern, to prevent pull-out. The manufacturers recommend a dosage between 8 and 15 pounds per cubic yard; thus the high dosage rates for this project's tests of macrofibers.

Figure 25: HPP (High Performance Polymer) fibers

3.4 Base Mix

The mix design was an ODOT Type AA typical with Fly Ash. This is the standard mix for bridge decks at this time. The mix was modified in two ways: the air-entraining agent was removed, and the ADVA high range water reducer was doubled. Since the air entraining increases workability and was removed, the additional ADVA was required to keep the mix workable, because fibers tend to decrease workability.

Table 7 gives the mix proportions used for this mix, the ODOT Type AA mix. The Portland cement used was a Holcim type I/II from Midlothian, Texas; for additional data on this cement, please see Appendix 4. The fly ash used for the primary matrix was from the Tecumseh, Kansas power plant, and is known as Ash Grove fly ash. The fine aggregate was a Dover river sand, and the coarse aggregate was a #67 crushed limestone aggregate from Richards Spur. The high range water reducer was ADVAcast 500.

Table 7: Base mix

	Mix Proportions	
Total Volume of Mix	1	yd^3
Cement	526.0	lb
Fly Ash	132.0	lb
Coarse Aggregate, #67	1772.6	lb
Fine Aggregate, Dover Sand	1392.5	lb
Water	268.5	lb
ADVA (HRWR)	40.0	oz
Fiber	0.0	lb

3.5 Typical Batching Procedure

The following discussion outlines the batching procedure used throughout this research project. On some batches, the procedure may have been somewhat different, as circumstances dictated, but wherever possible, this procedure was followed.

3.5.1 Pre-batching preparation

The mix was designed on a spreadsheet following the Goldbeck and Gray method, based upon the basic mix proportions outlined above. When fiber was added, an identical volume of fine aggregate was removed from the mix. An appropriate batch size was selected, and the amount of each material needed for the batch was calculated.

The day before the concrete was batched, appropriate amounts of coarse and fine aggregate were collected from piles outside the lab (Figure 26). The aggregate was weighed out into five gallon buckets; fifty pounds were stored in each bucket. The buckets were sealed and kept inside the lab until they were needed at the batch time. A representative sample of the aggregates was collected from the excess when weighing out the buckets. These two samples, one from the fine and one from the coarse aggregates, were weighed and heated in an oven overnight at a temperature of about 300° Fahrenheit, and then weighed again. The moisture content thus obtained was input into the batch spreadsheet to adjust the amounts of water and aggregate in the batch to compensate for the moisture of the aggregate.

Figure 26: Coarse aggregate pile

3.5.2 Batching Procedure

The cement and fly ash were stored in sealed barrels inside the lab; at the batching time an appropriate amount was weighed out. All materials were weighed out in 5 gallon buckets and carried out to the mixer. The aggregate, fiber, and a portion of the water were added to the mixer first, and mixed for less than a minute. The cement, fly ash, remaining water, and high range water reducer were then added. The mix was mixed for 3 minutes, let rest for 3 minutes, and then mixed for 2 more minutes before dumping into a wheelbarrow. Figure 27 shows the batching area.

Figure 27: Batching area

The concrete temperature, air temperature, and humidity were measured. While some of the researchers started filling the cylinders and shrinkage molds, others did the slump test, unit weight, and air content tests. The finished cylinders and shrinkage molds were taken into an environmental chamber, shown in Figure 28. The environmental chamber was kept at 73.4°±2° F and 50%±2% humidity. The time zero mold was set up and the initial value read when the mold was taken into the environmental chamber. Molds were removed at 1 day, and the unrestrained shrinkage tests zeroed at that point.

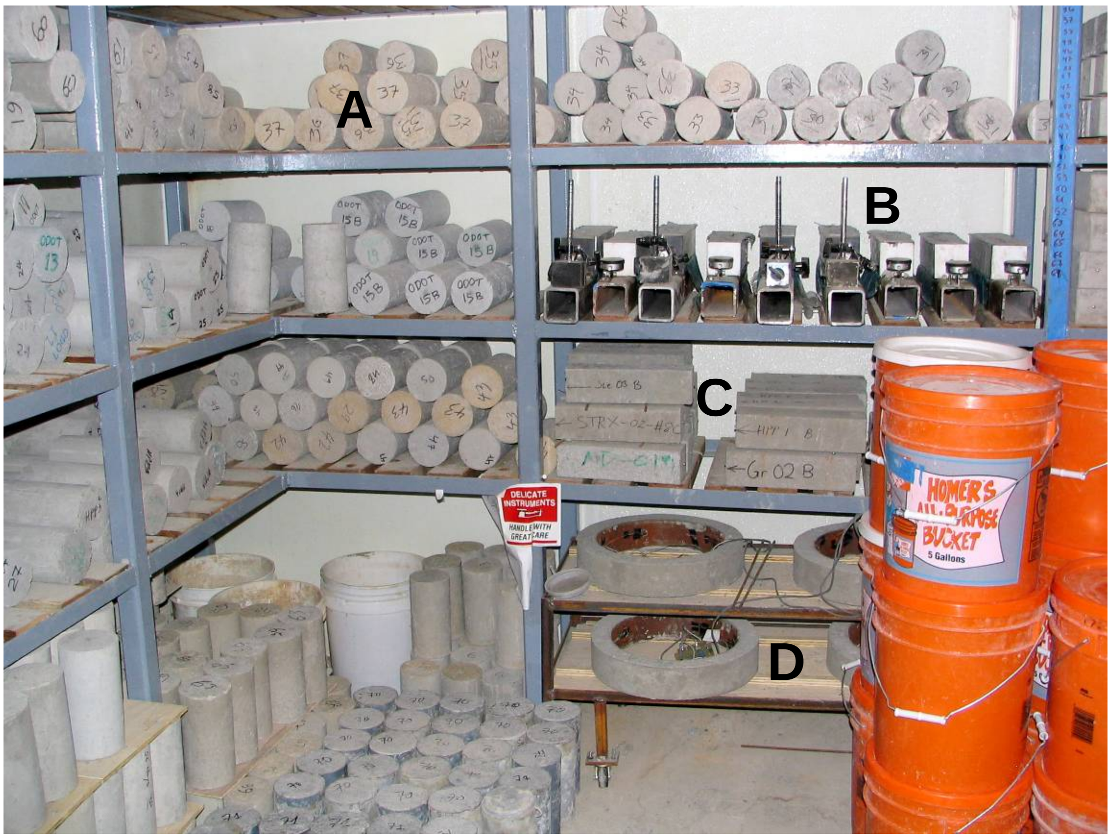

Figure 28: Environmental chamber and samples: A – 4x8" cylinders, B – unrestrained shrinkage from time zero samples, C – ASTM unrestrained shrinkage samples, D – restrained ring tests (not used in this research)

Chapter 4: Results

There were four fibers in the testing matrix, with several dosage levels for each (Table 8). These were selected based on the results of the preliminary matrix and of Jen Teck Kao's research (Kao, 2005), of which this was an extension. Manufacturer recommendations were also taken into account. The microfiber dosages selected were one, three, and five pounds per cubic yard. These were chosen based upon Kao's research, which indicated that higher dosage levels were not useful for microfibers. However, higher macrofiber dosages were included in the matrix, ten and fifteen pounds per cubic yard. These higher dosage rates were selected based upon the manufacturer recommendations, and upon the impact that the macrofibers had upon the concrete—macrofibers do not dry the mix out like microfibers, so higher dosage rates are possible. Several tests were conducted on each batch: compression, splitting tensile, unrestrained length change, and length change from time zero.

Table 8: Primary matrix batches

Fiber	Dosage Rates (lb/yd^3)
Fibermesh Stealth	1, 3, and 5
Grace Microfiber	1, 3, and 5
Strux 90/40	1, 3, 5, 10, and 15
HPP	1, 3, 5, 10, and 15
Plain Concrete	No fiber

4.1 Fresh Concrete Tests and Conditions

The air temperature, air humidity, fine aggregate moisture, and coarse aggregate moisture were measured to provide information about the batching conditions. It is well documented that the air temperature can influence concrete behavior. The air humidity is more important if the mix is cured outdoors, which was not the case here. The moisture contents of the aggregates were adjusted for in the mix proportions, but since the moisture measurements are not always completely accurate, very high and very low moisture contents are often associated with anomalous results. Table 9 gives the batching conditions.

Table 9: Primary matrix batching conditions

Fiber	Dosage (lb/yd^3)	Air Temp	Air Humidity	Fine Agg. Moisture	Coarse Agg. Moisture
Stealth	1	75	71%	2.20%	0.47%
Stealth	3	80	55%	3.96%	0.89%
Stealth	5	78	64%	3.96%	0.89%
Grace Microfiber	1	85	58%	2.26%	0.22%
Grace Microfiber	3	88	54%	1.46%	0.34%
Grace Microfiber	5	92	48%	0.91%	0.44%
Strux 90/40	1	91	44%	1.43%	0.16%
Strux 90/40	3	89	45%	1.43%	0.16%
Strux 90/40	5	70	83%	2.20%	0.18%
Strux 90/40	10	83	45%	1.70%	0.25%
Strux 90/40	15	87	43%	1.70%	0.25%
HPP	1	80	56%	1.77%	0.00%
HPP	3	72	88%	1.77%	0.00%
HPP	5	78	76%	2.26%	0.22%
HPP	10	70	55%	1.83%	0.17%
HPP	15	76	50%	1.83%	0.17%
Plain Concrete #2	0	88	56%	1.44%	0.17%
Plain Concrete #3	0	54.5	43%	1.73%	0.21%

All of the batches were completed during the summer and fall, usually during the morning hours. The air temperatures varied between 70° and 92° Fahrenheit, which is on the high end of permissible temperatures for batching. There was one exception: plain concrete #3 was batched in much cooler conditions. However, the concrete materials were kept indoors until batching—this tended to normalize the actual concrete temperatures. The actual conditions of all of the batches were fairly similar (Table 10). The air humidity varied considerably, but since the concrete was cured in an environmental chamber, the humidity would not have much impact on the mix conditions. The fine and coarse aggregate moisture contents were found by heating a sample in an oven overnight before the batching. The moisture was compensated for in the actual batches by subtracting an appropriate amount of water. However, it has been noted that very high moisture contents adversely affected the properties of the concrete. For this reason, two mixes were batched again, after poor results. For one, it appears that the oven had been turned off before drying had occurred in the samples. This led to a mix that was far wetter than it should have been.

Table 10: Primary matrix fresh concrete properties

Fiber	Dosage (lb/yd^3)	Slump (in)	Air Content (%)	Unit Weight (pcf)	Concrete Temp
Stealth	1	2	2.3%	152.00	80
Stealth	3	0.5	2.7%	151.24	86
Stealth	5	0.25	2.2%	151.16	82
Grace Microfiber	1	1.5	2.2%	151.12	92
Grace Microfiber	3	0.25	2.5%	151.24	86
Grace Microfiber	5	0	--	151.72	88
Strux 90/40	1	4	2.5%	151.72	90
Strux 90/40	3	3.25	2.8%	150.48	90
Strux 90/40	5	1.25	2.7%	151.56	78
Strux 90/40	10	0.25	2.4%	150.80	86
Strux 90/40	15	0	--	150.40	84
HPP	1	3.5	3.1%	149.52	89
HPP	3	2.75	2.4%	151.00	81
HPP	5	1	2.3%	152.08	90
HPP	10	2	2.4%	150.16	79
HPP	15	0.75	2.3%	150.76	78
Plain Concrete #2	0	4.5	3.4%	149.24	92
Plain Concrete #3	0	3.25	2.7%	150.12	77

The fresh concrete properties of the primary matrix reflect the batching conditions. Again, the concrete temperature was rather higher than desirable. However, since all mixes showed similar temperatures, ranging from 77° to 92° Fahrenheit, the high temperatures do not hinder comparative analysis. The unit weight measurements also showed considerable scatter. The plain concrete #2 test, in particular, is somewhat abnormal, as an identically proportioned batch in Jen Teck Kao's research showed an air content of 2.2% and a unit weight of 152.4 pcf. This, plus some other anomalous results by plain concrete #2, was the reason that plain concrete #3 was batched. Air content also showed scatter.

4.2 Compression Tests

The compression strength of concrete with and without fibers is very similar; however, the ductility of the failure is vastly increased with fibers. Instead of shattering at failure, at the higher fiber contents, the cylinders simply crack, and refuse to take more load. They do not fail entirely. See figures 29 and 30 for a comparison of plain concrete and fiber-reinforced concrete failures.

Figure 29: Plain concrete compression failure: brittle

Figure 30: Fiber-reinforced concrete compression failure: ductile (Strux 90/40 10lb dosage)

Table 11 gives the average compression strengths of the cylinders for each batch at each testing time. They are all averages of three tests, save a few that have only two tests because one of the cylinders was flawed, or a test went wrong. Samples from all of the batches in the matrix were compression tested at 1, 7, 14, and 28 days. At each time, three samples were broken, and the strengths averaged. In all cases, the load at failure of the cylinder was noted, and that load was divided by the area of a standard cylinder to obtain the compression strength in psi. A few of the batches did not have enough cylinders to average three values; those batches are noted in the table.

Table 11: Primary matrix compression test results

Batch		Compression Strength (psi)			
Fiber	Dosage (lb/yd^3)	1 day	7 days	14 days	28 days
Stealth	1	2676	5301	6514	6163
Stealth	3	2819	5916	6315	6547
Stealth	5	2907	5585	***	6249
Grace Microfiber	1	3015	5186	6447	7007
Grace Microfiber	3	3176	5689	6055	6241
Grace Microfiber	5	2863	5409	5958	6207
Strux 90/40	1	2710	5180	5909	6226
Strux 90/40	3	2775	5390	5429	5941
Strux 90/40	5	2530	5130	5335	5716
Strux 90/40	10	2711	5002*	5511*	6031
Strux 90/40	15	2488*	5223**	5475**	5718*
HPP	1	3012	5409	6105	6128
HPP	3	3139	5273	6003	6301
HPP	5	3270	5897	6649	6326
HPP	10	2527	5216	5678	5827
HPP	15	2255	5079	5345	5840
Plain Concrete #2	0	2908	5305	5932	6301
Plain Concrete #3	0	2458	4940	5640	6046
*Average of 2 cylinders tested **One cylinder tested ***Data missing					

The batches Grace Microfiber 1 lb and HPP 5 lb were the ones that were redone at a later date; they were slightly drier than expected (refer to the fresh concrete information). It is noted that the fibers seem to provide a slight improvement in strength in some dosage levels; for the microfibers, the strength decreases with increasing dose; for the macrofibers, the strength is more consistent throughout.

4.3 Splitting Tensile Tests

The primary matrix also was tested with the splitting tensile test to obtain the indirect tensile strength of the concrete. The cylinders were tested at 1, 7, 14, and 28 days, with three cylinders being broken at each testing time. On several occasions, less than three cylinders were broken; these are noted in the table below (Table 12). This test seems more likely to produce scatter than the compression tests; and in a few instances, the strengths showed behavior that is likely inaccurate, such as the strengths peaking at 7 days for the Grace Microfiber 5 lb mix. Such peculiarities are likely simply a product of the uncertainty inherent in testing a small sample. One behavior of the fiber mixes should be noted: quite often, the cylinders would continue to take load after they had split, as the fibers bridged the gap. This ductility was also shown in their post failure behavior—the fibers continued to hold the samples together even after splitting. Figure 31 shows a splitting tensile test failure with a high fiber dosage. Notice the fibers bridging the crack. Table 12 shows the data collected from the tensile tests.

Figure 31: Fiber-reinforced concrete splitting tensile failure: ductile

Table 12: Primary matrix splitting tensile strength

Batch		Splitting Tensile Strength (psi)			
Fiber	Dosage (lb/yd^3)	1 day	7 days	14 days	28 days
Stealth	1	368	724	682	720
Stealth	3	469	591	786	808
Stealth	5	451	625	***	772
Grace Microfiber	1	474	715	795	824
Grace Microfiber	3	483	608	764	667
Grace Microfiber	5	423	779	759	753
Strux 90/40	1	368	653	598	672
Strux 90/40	3	352	650	684	709
Strux 90/40	5	384	657	629	735
Strux 90/40	10	508	757*	777*	884
Strux 90/40	15	461*	802**	811**	771*
HPP	1	361	693	733	829
HPP	3	474*	666	783	766
HPP	5	494	639	684	775
HPP	10	393	686	861	842
HPP	15	369	690	746	860
Plain Concrete #2	0	365	670	717	694*
Plain Concrete #3	0	419	640	735	797
*Average of 2 cylinders tested **One cylinder tested ***Data missing					

This test seems to yield somewhat more erratic results; nevertheless, trends were evident. Again, the cylinders were tested in groups of three at 1, 7, 14, and 28 days.

4.4 Unrestrained Shrinkage

Shrinkage evaluation is the primary objective of this research. Unrestrained shrinkage is the standard way of measuring this, though it starts at 24 hours, well after final set. The samples tested were removed from their molds and zeroed at 24 hours. There were three samples for each batch; they were read at 1, 3, 7, 14, and 28 days. Some of the tests were read at 75 days as well.

There are two plain concrete mixes shown in Table 13, plain concrete #2 and #3. They have the same mix proportions, but for some reason they differ widely in their unrestrained shrinkage behavior. A considerable difference in their air contents was noted; this is perhaps related to this trend. Plain concrete #2 had a very high air content. It is unclear which best represents the actual behavior; this topic will be discussed at length in a later section. It is noted, however, that the long term trends of both are similar, and that the fiber-reinforced mixes tend to have a trend toward less shrinkage at greater ages.

Table 13: ASTM unrestrained shrinkage test results (normalized at 1 day)

Fiber	Dosage (lb/yd^3)	3 days	7 days	14 days	28 days	75 days
Stealth	1	77	167	250	230	313
Stealth	3	65	140	235	275	345
Stealth	5	100	165	***	290	345
Grace Microfiber	1	15	100	145	250	300
Grace Microfiber	3	53	133	170	233	297
Grace Microfiber	5	57	160	183	247	310
Strux 90/40	1	***	215	230	260	295
Strux 90/40	3	113	213	353	257	313
Strux 90/40	5	80	***	223	260	***
Strux 90/40	10	70	127	190	250	***
Strux 90/40	15	40	85	150	210	***
HPP	1	60	113	163	217	280
HPP	3	67	113	147	207	263
HPP	5	60	107	143	223	287
HPP	10	73	117	193	250	***
HPP	15	37	93	170	203	***
Plain Concrete #2	0	33	93	153	240	320
Plain Concrete #3		57	133	213	283	***
*** Data Missing						

4.5 Unrestrained Shrinkage from Time Zero

The unrestrained shrinkage from time zero test is an innovative test used to obtain free shrinkage from the batching time. One test was run on each batch, so the potential for scatter was not accounted for. However, since the values for the free shrinkage vary so widely with different mixes, the scatter does not affect the usefulness of the data in comparing the mixes. The time zero test for plain concrete #3 failed, so the results for that batch are not included here.

The shrinkage gauge was read at the initial casting time, 1, 2, 3, 4, 5 and 6 hours after batching, and 1 day, 3 days, and 7 days after casting. On some of the batches the readings were continued out to 14 and 28 days. Table 14 gives the shrinkage values at 6 hours, 24 hours, and 7 days as found by this test.

Table 14: Unrestrained shrinkage from time zero tests results (normalized at time 0)

Batch		Shrinkage (microstrain)		
Fiber	Dosage (lb/yd^3)	6 hours	24 hours	7 days
Stealth	1	2460	2480	2620
Stealth	3	1720	1730	1900
Stealth	5	840	810	960
Grace Microfiber	1	1470	1450	1590
Grace Microfiber	3	1380	1350	1550
Grace Microfiber	5	1500	1500	1620
Strux 90/40	1	2500	2490	2630
Strux 90/40	3	1680	1650	1650
Strux 90/40	5	1280	1280	1400
Strux 90/40	10	430	450	530
Strux 90/40	15	640	640	770
HPP	1	1790	1790	1850
HPP	3	960	960	1110
HPP	5	1390	1450	***
HPP	10	1110	1110	1220
HPP	15	1190	1190	1290
Plain Concrete #2	0	1840	1820	1930
(Plain Concrete #3's test failed) ***Data missing				

The shrinkage from time zero test gives insight into a period of concrete shrinkage that is rarely investigated. Since the ASTM unrestrained shrinkage test is usually normalized at 1 day, the early age shrinkage is missed. The data above shows that a very large portion of the free shrinkage of concrete is ignored with that test. In fact, most of the concrete's free shrinkage occurs before 6 hours. From 6 to 24 hours the concrete often actually expands (like the Strux 90/40 1 and 3 lb batches), and then proceeds after 24 hours on the familiar shrinkage curves found by the ASTM unrestrained shrinkage test.

The early age unrestrained shrinkage is strongly affected by the fiber dosage rates. This impact will be discussed in depth in the discussion section.

4.6 Fiber-Reinforced Concrete: Summary of Results

The general indication from these tests is that fibers have a significant effect on early age compression strength. At 28 days, neither the compression nor splitting tensile tests show much change with the addition of fibers. The unrestrained length change tests indicate that fibers decrease the 28 day shrinkage values somewhat. Fibers greatly reduce early age shrinkage, according to the unrestrained shrinkage from time zero test, but in this case the effect depends very strongly on the dosage rate of the fibers. Plain concrete has about 2000 microstrains of early age shrinkage; that is decreased by about 60% in some of the fiber mixes.

4.6.1 Stealth Summary

Stealth is a microfiber, yielding slumps from 0.25 inches to 2 inches. The compression strength showed some improvement over plain concrete; the one and three pound dosage mixes were the highest. On tensile strength, the results were rather erratic, some points higher and some lower than the control. The unrestrained shrinkage results indicated that the Stealth fibers increased shrinkage from 1 to 7 days, but after that they reduced the shrinkage considerably. The time zero test results indicate an increase in early age shrinkage with the one pound dosage rate, but a huge decrease, from 2000 to about 800 microstrains, with the five pound dosage mix.

4.6.2 Grace Microfiber Summary

The Grace Microfiber mixes had slumps between 0 and 1.5 inches. The compressive strength was again moderately increased by the fibers; the one pound per cubic yard mix was the one that showed the greatest increase, about 1000 psi at 28 days. The splitting tensile tests erratic behavior; the best results increased strength by about 50 psi. The unrestrained shrinkage specimens showed slight improvement over the plain concrete; the trend at 28 days was toward less shrinkage than the plain concrete control. The unrestrained shrinkage from time zero tests indicated that all three dosages cut the early age shrinkage by about 500 microstrains; there was not much difference between the dosages.

4.6.3 Strux 90/40 Summary

Strux 90/40, a macrofiber, yielded slumps from 1.75 to 4 inches, depending on dosage. The compression strength was slightly lower for the three and five pound dosage mixes than for the control at 28 days. Good compression strength results were found at 24 hours, however. The tensile strength was not substantially affected, though there seems to be a slight progressive increase in strength with increasing dosage; the five pound mix was about 80 psi stronger than the control at 28 days. The unrestrained shrinkage showed higher shrinkage from 1 to 7 days, and then lower thereafter than the control. The time zero shrinkage test showed higher early age shrinkage than the control for the one pound dosage, slightly lower than the control for the three pound mix, and considerably lower than the control for the five pound mix. The ten and fifteen pound mixes showed outstanding early age shrinkage reduction.

4.6.4 HPP Summary

The HPP macrofiber mixes had slumps from 1 to 3.5 inches. The five pound per cubic yard mix showed 300 psi gains in compressive strength at 28 days, while the other batches showed minimal differences from the control. At 24 hours, however, the compression strengths were increased by as much as 750 psi. The splitting tensile capacity of the batches was not strongly impacted by the addition of the fibers. The unrestrained shrinkage tests showed considerable improvement for all five dosage rates. The length change from time zero tests showed the least early age shrinkage from the three pound per cubic yard mix, with the five pound dosage giving some reduction, and the one pound mix mirroring the plain concrete control mixes. The ten and fifteen pound mixes showed early age shrinkage results similar to the three pound mix.

Chapter 5: Discussion

This study is focused on the behavior of the four types of polymer fibers, particularly their impact on shrinkage. However, a full range of aspects of the fiber-reinforced concrete was investigated to see whether the fibers impacted them. Workability, a concern with fiber-reinforced concrete, was considered in detail. Shrinkage, both plastic and long term, was investigated in depth, and compression and tensile strength were also considered.

The objectives were two-fold: to characterize the mixes, and to determine which dosage rates of what fibers were best. To do this, appropriate plain concrete mixes were batched as controls. Each fiber was analyzed separately to determine its optimum dosage rates, and to see its strengths and weaknesses. Finally, a discussion of the differences between macro and micro fibers is presented.

5.1 Workability

There is a strong correlation between fiber dosage and slump; workability is strongly affected by the fibers. Two of the fibers were macrofibers, the Strux and the HPP. Those fibers did not dry up the mix nearly as much as the microfibers. They did, however, affect the finishing more. Nevertheless, due to their lower surface area per weight, the macrofibers were easier to consolidate, and there was not as noticeable a difference between the low and high dosage rates.

All of the batches were batched at temperatures higher than typically considered permissible, but since all were fairly similar, the results are valid in comparison with one another. The moisture contents of the aggregates have some effect on the mixes, because with increasing moisture content, the error in obtaining the moisture content goes up, and thus there is more random error in the actual water in the mix. This has affected all of the characteristics of the concrete; two batches with very high moisture contents had to be redone entirely.

5.1.1 Slump

All of the fibers strongly impacted the slump of the mix. As the fiber dosage rate increased, the slump decreased. Figure 32 below illustrates the findings.

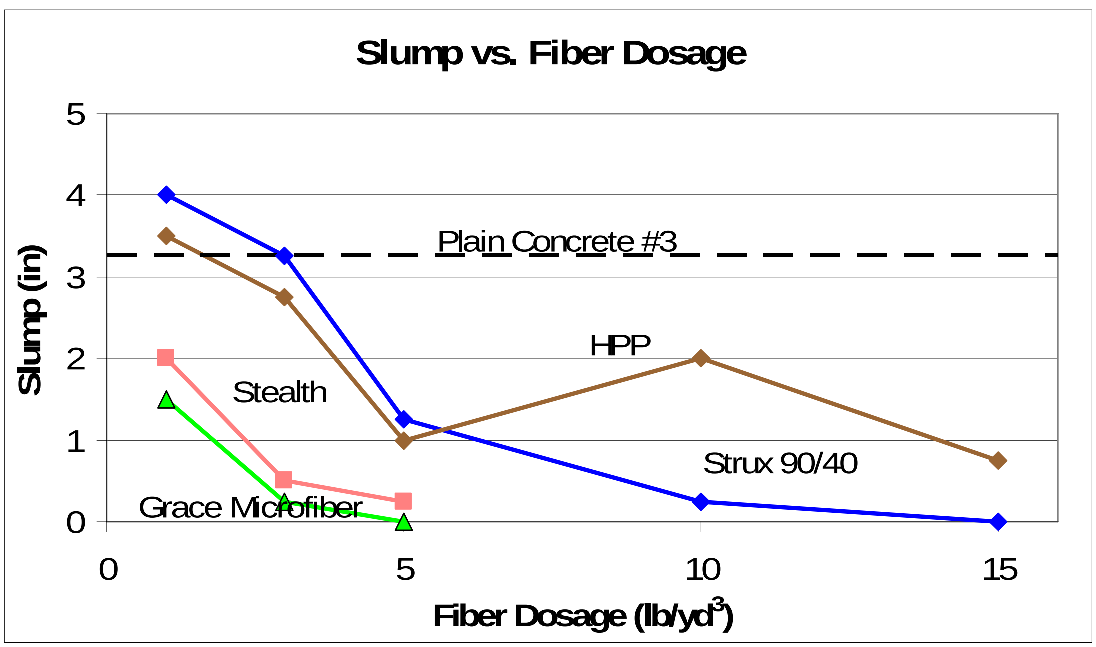

Figure 32: Slump versus fiber dosage

There are two classes of fibers represented here, as mentioned earlier. The microfibers (Stealth and Grace Microfiber) both bind the mix together and dry the mix out due to their high surface area to volume ratios. Thus the slumps of the microfiber mixes start at the 1 lb dosage well below plain concrete, and quickly drop until they approach zero.

The macrofibers, on the other hand, do not dry the mix out. The higher dosage mixes appeared fully as moist as the low dosage mixes. The only mechanism for reducing slump, then, was mechanically holding the mixture together. Thus, there had to be far more fibers before the mixes showed significantly less slump. The Strux fiber was smaller than the HPP, and approached zero more quickly. The HPP 15 lb mix, though full of fibers, still showed ¾" slump. It is doubted whether the HPP fibers would ever reduce the slump to zero, even at dosages higher than those tested here. Figure 33 shows a mix with a high dosage rate of HPP fibers. Though coarse, the mix is still moist enough to finish.

Figure 33: Concrete mixture with high dosage of HPP fibers

5.1.2 Finishing

The finishing characteristics are a much more qualitative measure. The evaluation here is based upon the experiences of the researcher. Again, as in all the workability measurements, the macrofibers and microfibers behaved very differently. The microfibers dried the mix out, but did not greatly hinder the finishing. There were some of the microfibers sticking through the surface of the concrete, but these were not much of a problem, as they are so soft and fine. The macrofibers, on the other hand, were quite difficult to finish on the cylinders. The HPP, in particular, are so stiff that they tended to stick out in all directions and required considerable work to level. The Strux 90/40 fibers, similar to small ribbons, were also difficult to finish as they also stuck out. Figures 34 and 35 show the finishing characteristics of HPP and Strux 90/40 fibers at high dosage levels.

Figure 34: Concrete finish on HPP high dosage mix at time of casting

Figure 35: Concrete finish on Strux 90/40 high dosage mix after unmolding

5.2 Fresh Concrete Characteristics

The fresh concrete properties of the mixes did not show a strong influence from the addition of fibers. Theoretically, the unit weight of the mixes should have decreased somewhat with increasing fiber dosage, as the fibers replaced an equal volume of sand. However, as shown in Figure 36, there was no clear trend in the unit weights of the mixes. The experimental scatter of the mixes and of the test itself obscured any trend. Since all of the fibers had the same unit weight, the theoretical curve shown is the same for all of them.

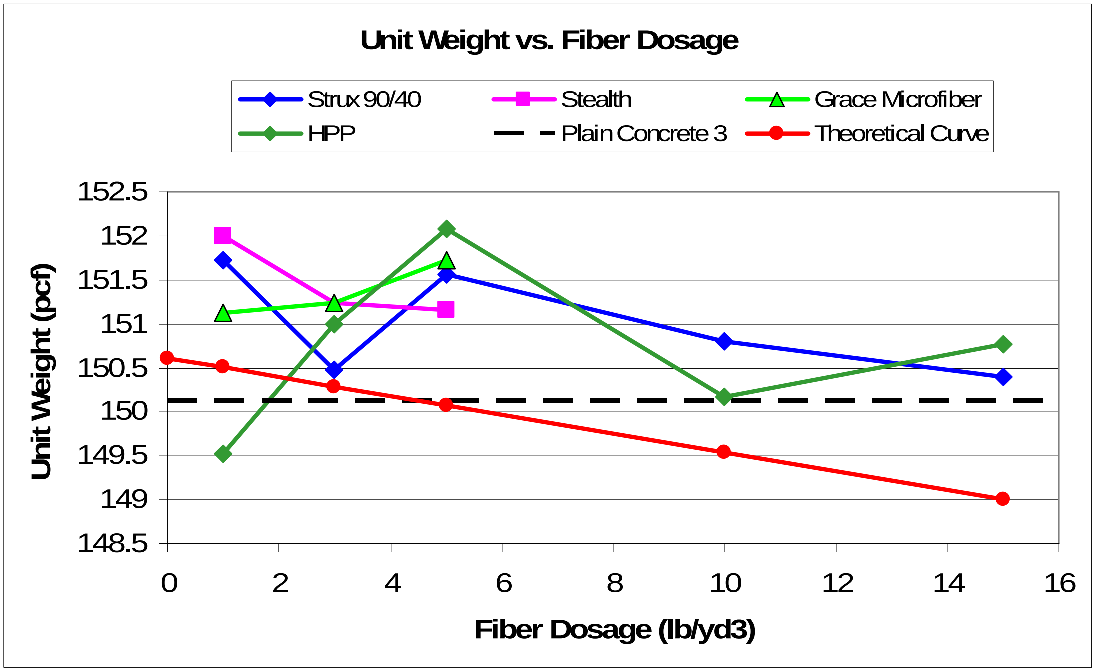

Figure 36: Unit weight versus fiber dosage

Similarly, the air content of the mixes did not show a clear trend (Figure 37). On several batches the air content was not measured, either due to a lack of material or problems with the testing apparatus. Even so, the scatter of the results is such that one may conclude that air content is not impacted strongly by the addition of fibers.

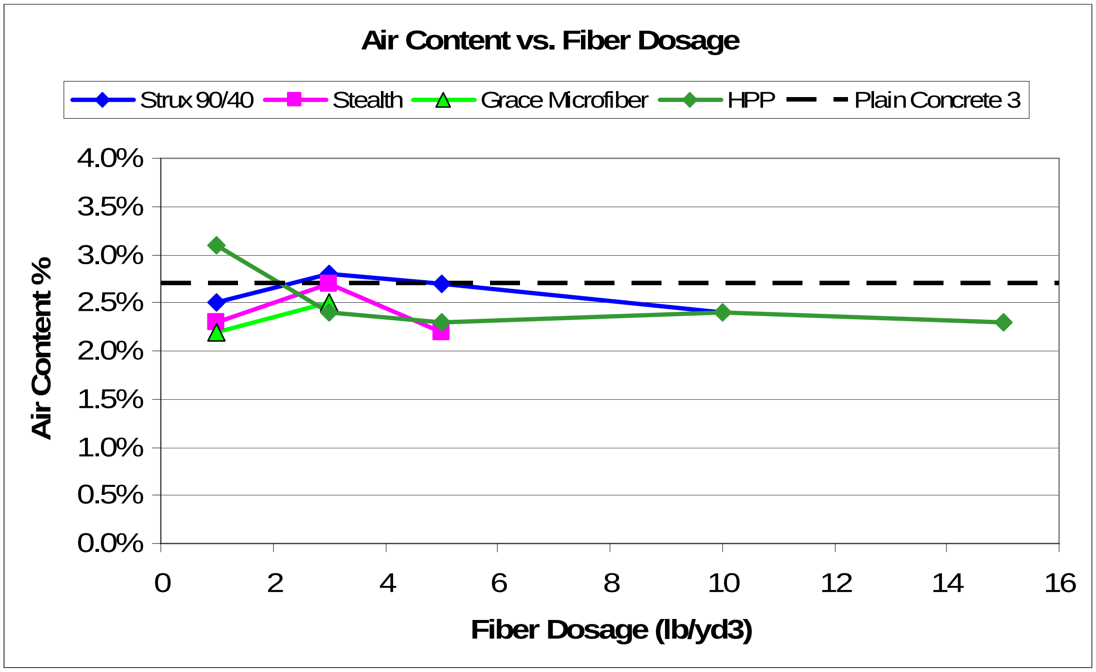

Figure 37: Air content versus fiber dosage

5.3 Shrinkage

Shrinkage was the principle topic of interest in this study, as it relates most directly to the bridge deck cracking problem. The two tests used measure strictly unrestrained shrinkage, so the concrete's response to restraint is not evaluated. Nevertheless, the unrestrained shrinkage data obtained gives strong indications of how adding fibers to bridge deck concrete will impact the cracking problem.

There are several topics within the shrinkage area that will be considered. First, the unrestrained shrinkage from time zero test itself will be discussed, including how consistent, how useful, and how accurate the test is. Next, an evaluation of the long and short term shrinkage, and how they relate, will be undertaken. Finally, the fibers themselves will be discussed in depth.

5.3.1 The Unrestrained Shrinkage from Time Zero Test

The unrestrained shrinkage from time zero test is a new test. It was first used, in a much different form, by Ramseyer (1999). Subsequently, the test was greatly modified by Kao (2005). The test was further refined for this project; the present design was discussed in the research scope section. Here, one sample was used for each batch, primarily due to the difficulty in setting up the test and to limited quantities available.

How can this test be validated? The repeatability of the test has not been strongly tested. On one batch, there were two samples cast, one using the latest mold design, and one using Kao's design. The results were compared (Figure 38).

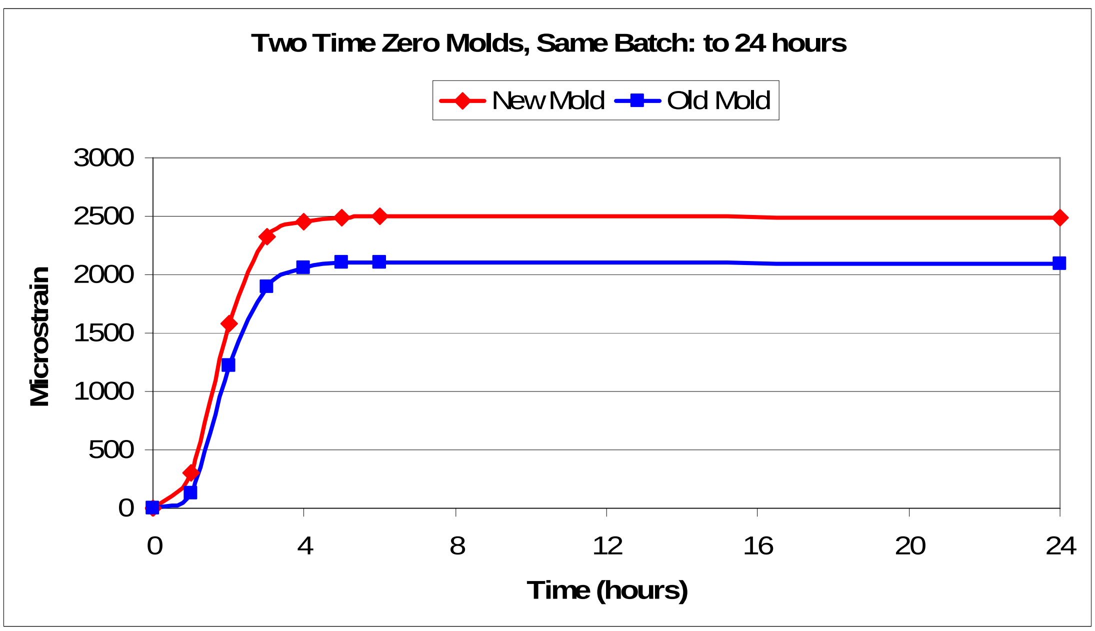

Figure 38: Time Zero Mold Comparisons to 24 hours (Strux 90/40 1lb)

The new design exhibited considerably more shrinkage than the old. There are several possible explanations for this. First, the mold design was changed to promote an even more free movement of the concrete specimen and the attached Teflon plate. The other possibility is that the test is simply very sensitive to slight variations in conditions in the early age high shrinkage period. It is noted that the shrinkage curves mirror each other very closely after the first 4 hours. Figure 39 shows the curves, starting at 4 hours—here, the results were zeroed at 4 hours.

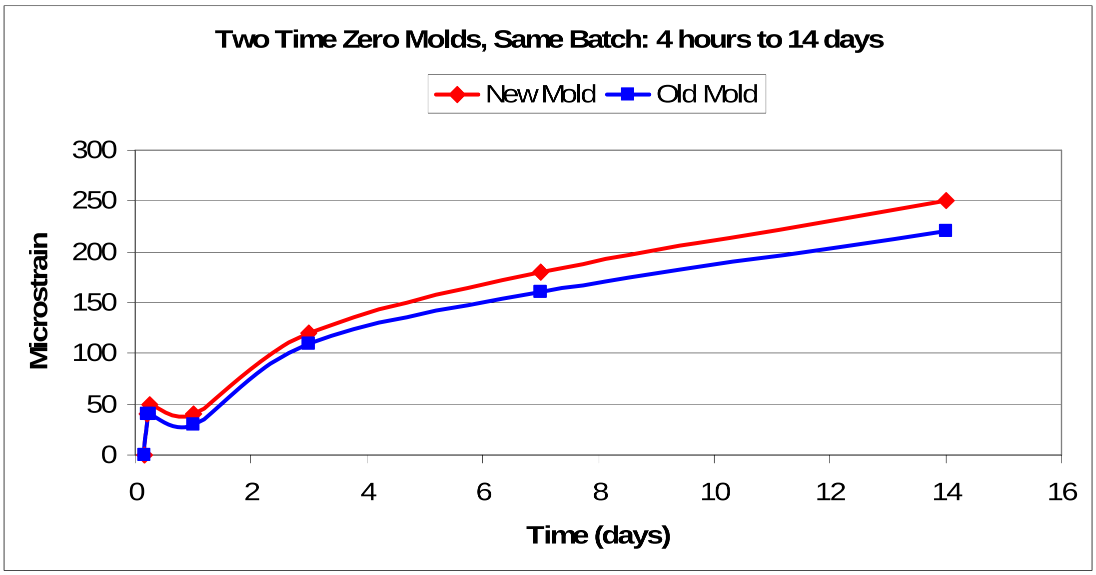

Figure 39: Comparison of Time Zero Molds from 4 hours (Strux 90/40 1lb)

What does this mean? The variation in shrinkage between the two molds occurred before the concrete had finally set. At this point, the concrete was much more sensitive to the level of restraint in the molds. In addition, the concrete would also be very sensitive to curing conditions, particularly evaporation rate. However, these two molds were cured side-by-side in an environmental chamber. Therefore, it is thought that the primary reason for the difference between the two molds is the reduced level of restraint in the latest time zero mold and perhaps some experimental scatter. Since the difference between mixes is usually very large with the time zero test, it was not thought necessary to use multiple time zero tests to correct for experimental scatter, so the results must be considered as approximate, and useful primarily for qualitative comparison rather than quantitative analysis.

An obvious method of validation for this test is to run it out to 28 days and compare its results with those obtained by the standard ASTM unrestrained shrinkage test. This was done on seven different mixes. The values of the time zero tests at 24 hours were taken as the zero point for the time zero shrinkage, and that value subtracted from the results in order for the comparison to take place. Figure 40 gives the average results of the seven; the fairly good agreement indicates that there is no strong systematic error.

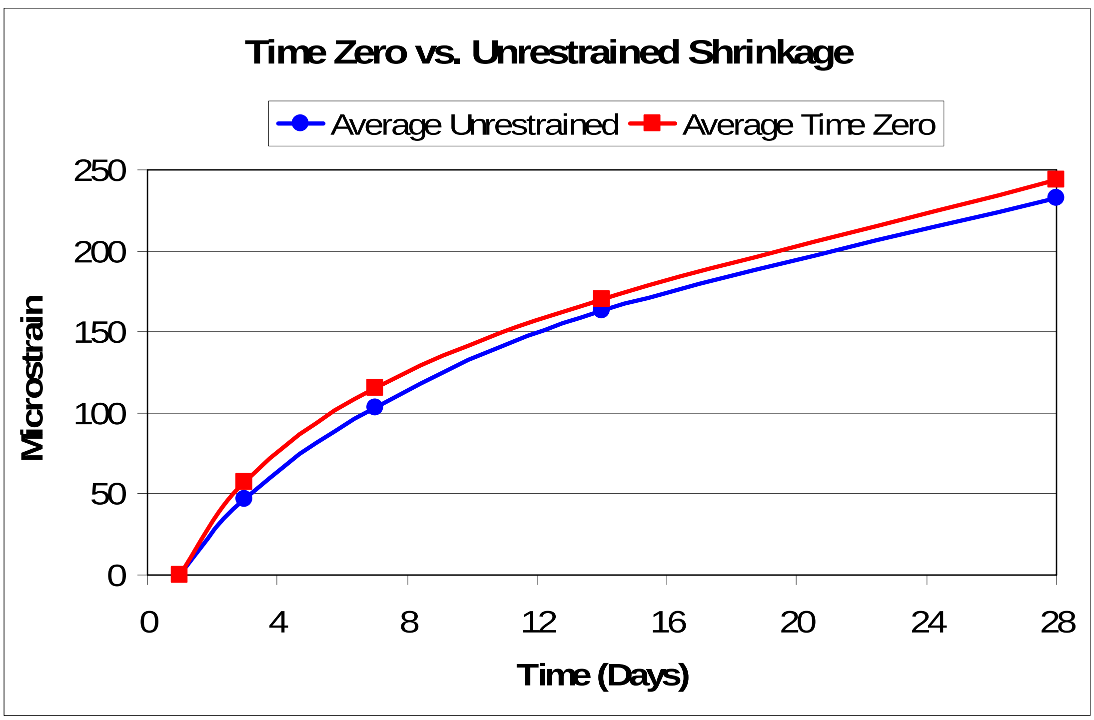

Figure 40: Average time zero versus average ASTM unrestrained shrinkage

As may be expected, however, there is some variation in individual mixes due to the small sample size. Figures 41, 42, and 43 show the comparisons of the ASTM unrestrained shrinkage test results to the unrestrained shrinkage from time zero results. In all cases, there is some difference between them. It must be noted that the ASTM unrestrained shrinkage is more sensitive to operator bias, while the time zero test is more sensitive to variations in the environment and setup. There were three ASTM unrestrained shrinkage samples used for each test, and the numbers shown are the average. On the other hand, there was only one time zero mold for each batch.

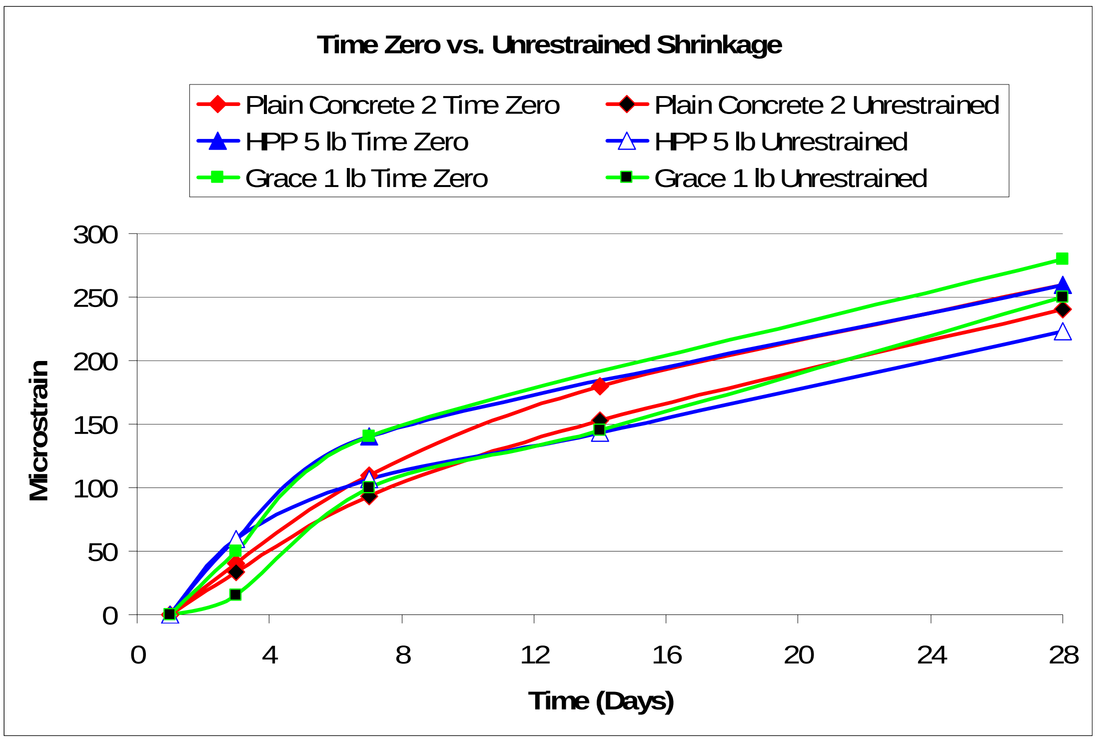

Figure 41: Time Zero versus ASTM Unrestrained Shrinkage

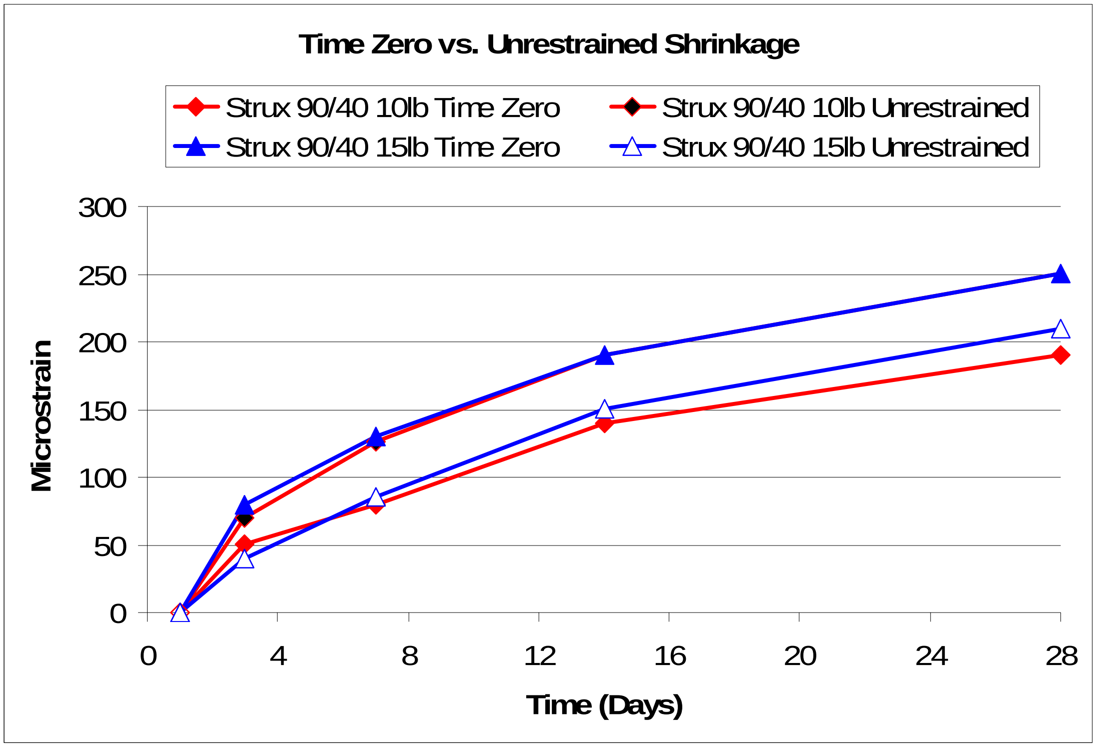

Figure 42: Time Zero versus ASTM Unrestrained Shrinkage (Strux 90/40 high dosage rates)

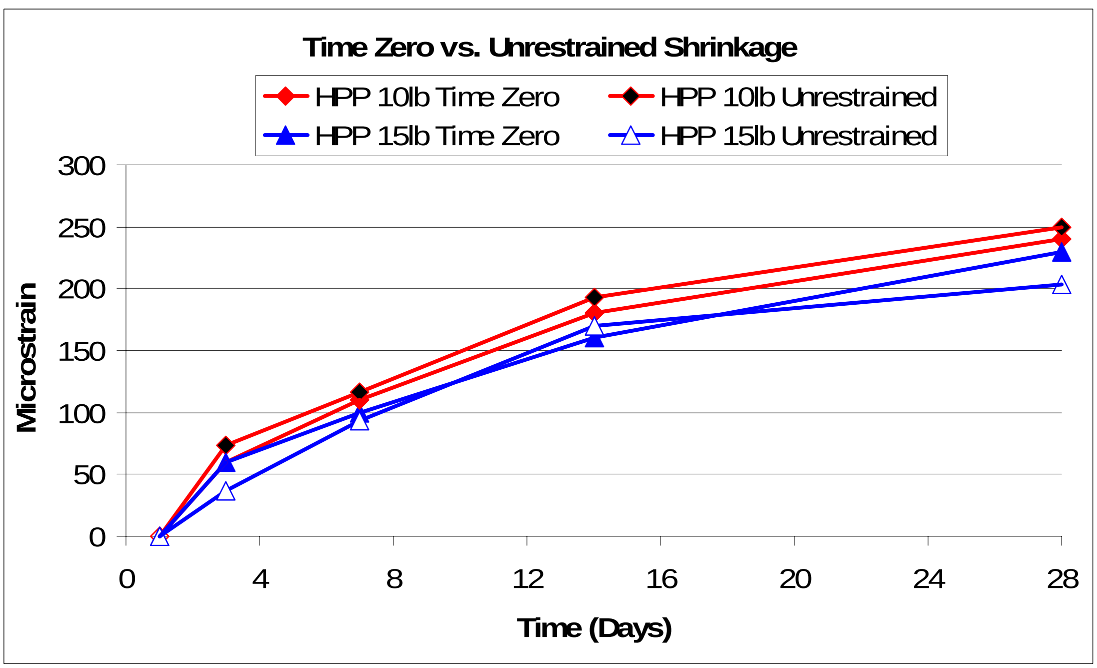

Figure 43: Time Zero versus ASTM Unrestrained Shrinkage (HPP high dosage rates)

What do the numbers generated by the unrestrained shrinkage from time zero test mean? Are they good for anything? This test provides a valuable insight into the plastic early age shrinkage of a concrete. As discussed earlier in the literature review, many bridge decks crack at early age, and much of the problem is associated with plastic shrinkage. The plastic shrinkage, as shown in this test, can have magnitudes nearly 10 times the size of the drying (long-term) shrinkage for the same mix. If only a small fraction of this shrinkage is converted to residual stress in the concrete, the concrete is well on its way to cracking. Most of the shrinkage is compensated for by creep, as the concrete has not reached final set. In addition, the modulus of elasticity of the concrete is low, so the stress developed is relatively low for a given shrinkage value. Nevertheless, because there is so much shrinkage, and because the concrete (in the case of a bridge deck) is often restrained by a rigid substrate of some sort, early age cracking is a distinct possibility. With this test, the plastic shrinkage can be measured, and this test can provide a valuable qualitative measure for comparing mixes. Because the translation to stress is unknown and varies, quantitative analysis of the residual stress developed cannot be undertaken. However, if a mix has one quarter the plastic shrinkage of another, it is valid to conclude that that mix is far less likely to crack at early age than the other. Therefore, this test will provide good, useful information on the early-age cracking tendencies of the concretes tested here.

A further discussion of the results of the time zero test is warranted. The magnitude of the shrinkage seen in this test is far in excess of that seen in other tests. A typical test will reach 1500 to 2000 microstrain within 6 hours, while an ASTM shrinkage test will reach some 300 to 500 microstrain at 28 days. The very high shrinkage values are primarily a product of the plastic shrinkage of the mix; the mechanisms of plastic shrinkage were detailed in the literature review. Figure 44 from Holt (2001) shows how the plastic shrinkage magnitudes are affected by curing conditions.

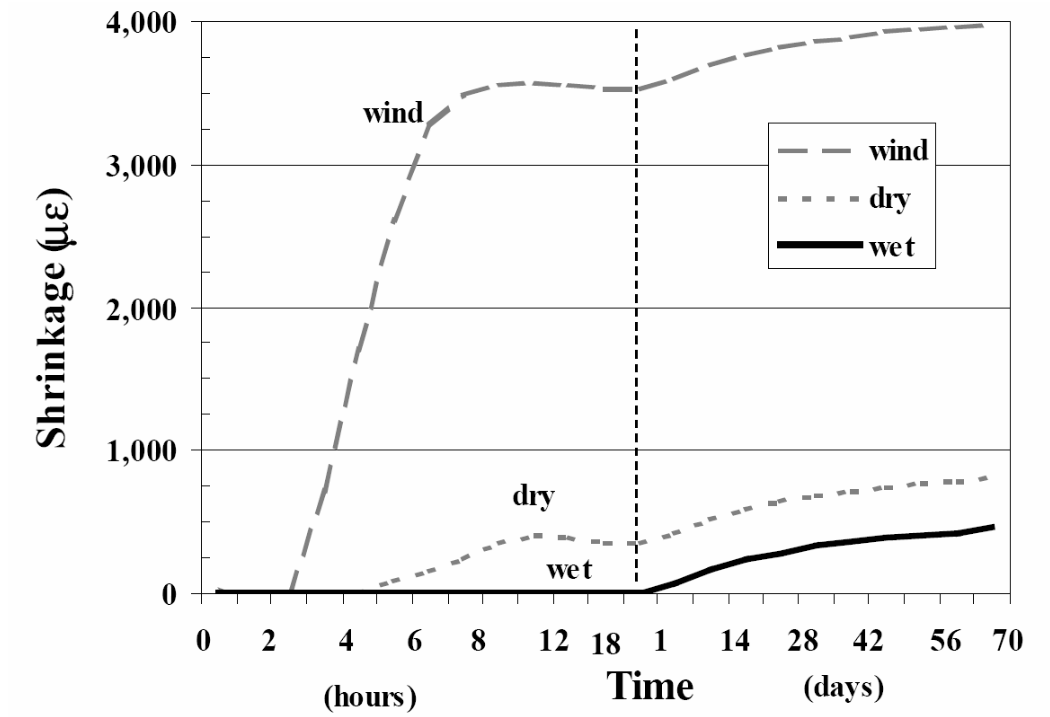

Figure 44: Accumulation of early age and long term shrinkage, with various curing environments during the first day. Wind = 2 m/s (4.5mph), dry = 40% RH, wet = 100% RH. (Holt, 2001)

Primarily, the shrinkage is caused by evaporation, causing the free water surface to drop inside the concrete. The menisci of the surface exert a suction of sorts on the particles surrounding them, causing shrinkage. Because of this mechanism, the plastic shrinkage is very sensitive to the curing conditions, particularly wind, humidity, and temperature. This makes comparison of mixes not cured in identical conditions almost impossible. All of the batches in this research project were cured in an environmental chamber at 72° F and 50% humidity. The environmental chamber where the samples were cured is rather breezy from the air conditioner, dehumidifier, and other equipment. This probably contributed to the large magnitude of the plastic shrinkage readings. It does not, however, hinder comparison between mixes cured in identical conditions as these were.

5.3.2 Shrinkage from Time Zero

Shrinkage from time zero, as just discussed, provides a good insight into the plastic shrinkage behavior of the fiber-reinforced concrete batches tested here. First, the two microfibers will be discussed, with their behavior at early age, and then the two macrofibers.

5.3.2.1 Shrinkage from Time Zero: Stealth

The Stealth microfiber is a very small and fine fiber, hardly visible in the concrete. It provides a drying impact on the mix, as well as a mechanical internal restraint. The fibers, particularly at the higher dosage rates, are ubiquitous through the mix—every portion of the mixture is held to every other by many tiny fibers. This holding together of the mix accounts for the dramatic reduction of plastic shrinkage seen in these fibers at high dosage rates. Figure 45 shows the results to 24 hours for the three Stealth mixes. It is unknown why the Stealth 1 lb dosage showed an increased plastic shrinkage. It appears that a dosage rate of at least 3 lb per cubic yard is needed to realize significant reductions in plastic shrinkage with this fiber. The 5 lb per cubic yard mix yielded one the lowest shrinkage from time zero results of any mix tested in this research. Unfortunately, the mix was also very dry and hard to work with due to the high water demand of the microfibers. The trend of reducing early age shrinkage by increasing fiber dosage is very strong; however, this benefit has to be weighed against workability issues associated with the large surface area of the fibers. The slump on the 5 lb mix was only 0.25 inches, and the workability was fully as bad as that low slump indicates. The Stealth fibers, due to their huge number, form a web through the mix, and at the high dosage levels, nearly a mat, making consolidation very difficult. It is clear that plastic shrinkage can be reduced substantially with high dosage rates of the Stealth fiber, but other factors must be considered in determining an optimum dosage rate; that will be the subject of a later section.

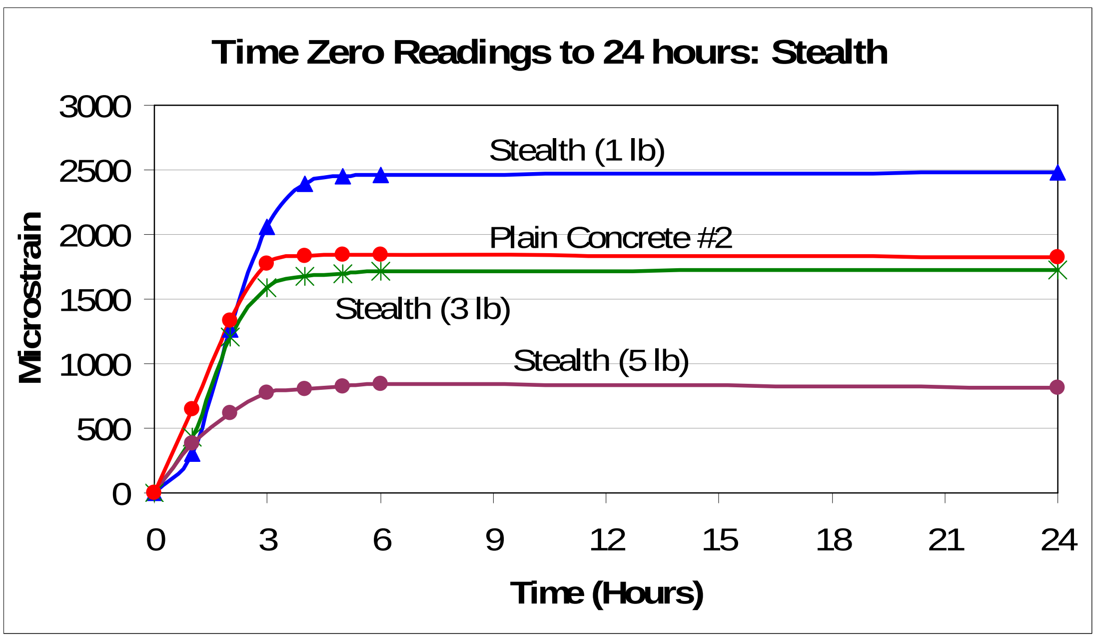

Figure 45: Time zero shrinkage results: Stealth

5.3.2.2 Shrinkage from Time Zero: Grace Microfiber

Grace microfiber is similar to the Stealth fiber, though manufactured by a different company. Both are very fine and small. It would be expected that the early age shrinkage results would be very similar, but this was not the case. Figure 46 shows the time zero shrinkage results for the Grace fibers. One thing may have caused the odd results: the 1 and 5 lb dosage rate mixes were tested with old time zero molds. This may have somewhat decreased the apparent shrinkage for those mixes. With this accounted for, it appears that the Grace Microfiber 3 lb per cubic yard dosage rate was the best at reducing plastic shrinkage. A further analysis of what dosage rate is best for this fiber is undertaken later.

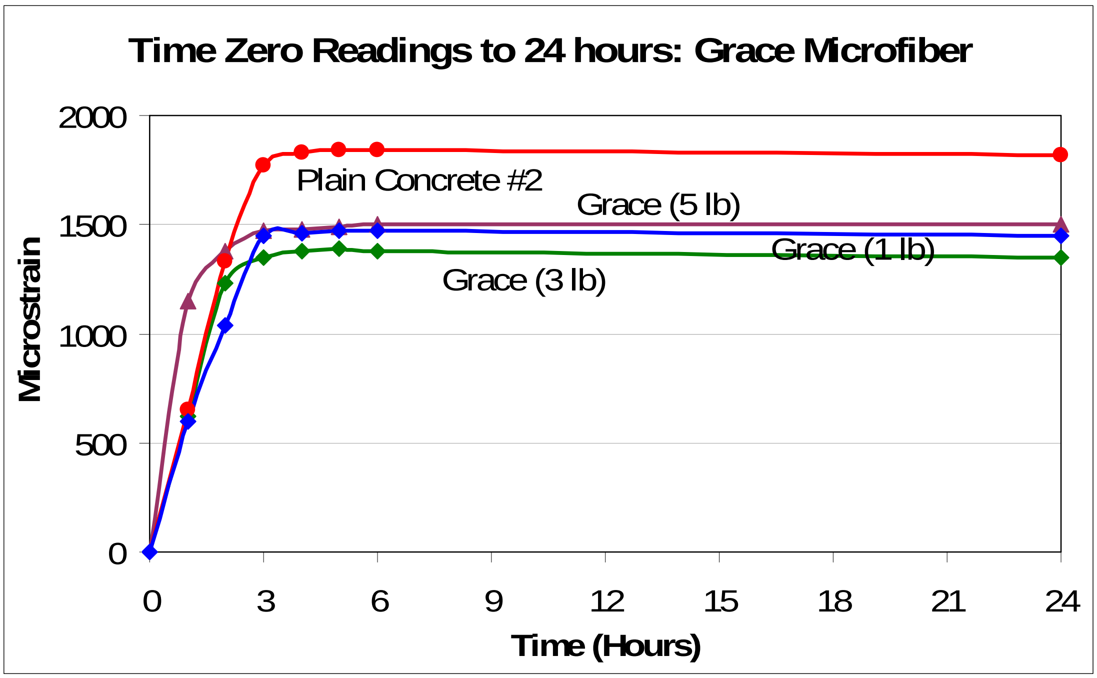

Figure 46: Time zero shrinkage results: Grace Microfiber

5.3.2.3 Shrinkage from Time Zero: Strux 90/40

The Strux 90/40 fiber is the smaller of the two macrofibers tested. The 1, 3, and 5 lb mixes behaved similarly to the Stealth fibers: 1 lb per cubic yard dosage significantly increased the plastic shrinkage, while the 3 lb dosage rate was similar to the plain concrete control mix. The increase in plastic shrinkage at 1 lb dosage may be because the flat fibers (their aspect ratio is 90) act as slip planes in the matrix; whatever the reason, this phenomenon disappeared at higher dosage rates. At the higher dosage rates, the results got considerably better; the 10 lb dosage rate yielded the lowest time zero shrinkage result of any mix in this research. It is interesting to note that the 15 lb dosage rate had a higher plastic shrinkage than the 10 lb; it is likely that the 10 lb dosage is close to the optimum dosage for reducing plastic shrinkage with this fiber. Since these are macrofibers, they did not significantly dry the mix out, so very high dosages, like those undertaken here, were quite feasible. Figure 47 gives the results.

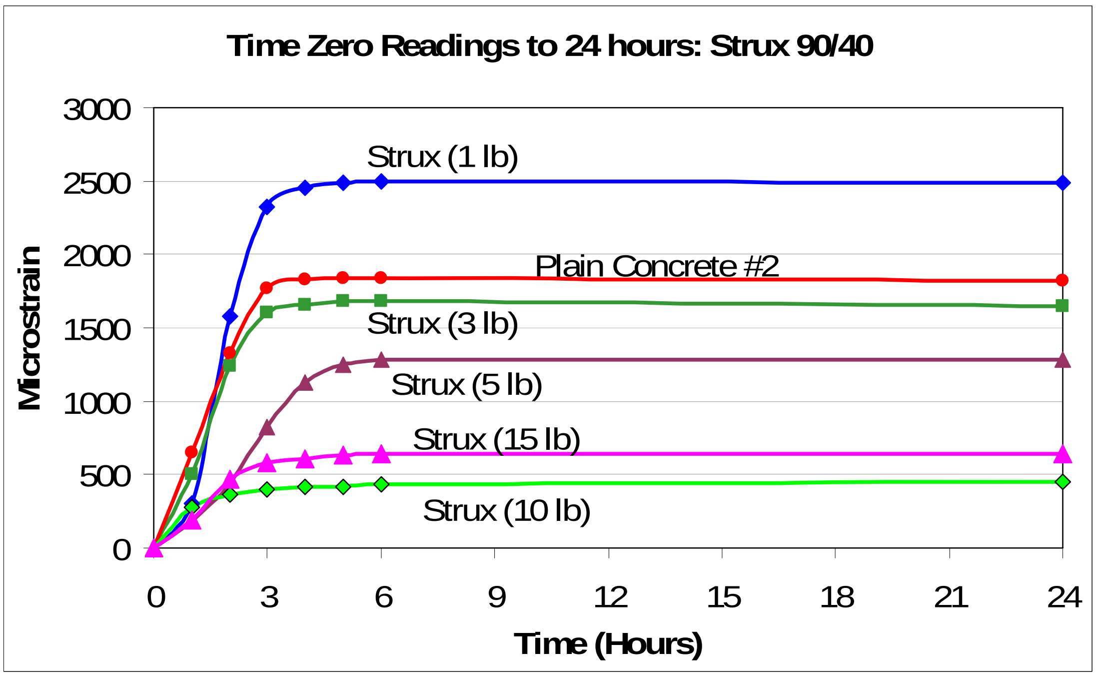

Figure 47: Time zero shrinkage results: Strux 90/40

5.3.2.4 Shrinkage from Time Zero: HPP

The high performance polymer fiber was by far the largest and stiffest fiber tested. Low dosage rates of this fiber did not impact the behavior of the concrete very much, as there were simply too few fibers to do much. Like Strux, HPP reached a point where the addition of more fibers increased plastic shrinkage, rather than reducing it. It is uncertain, however, what dosage is the optimum, as there was no consistent trend. The HPP 3 lb mix readings may be an anomaly, but there was no other indication of odd behavior with that mix. Further analysis of the HPP fibers, and a determination of the optimum dosage for this mix, will be undertaken later on. Figure 48 gives the plastic shrinkage results.

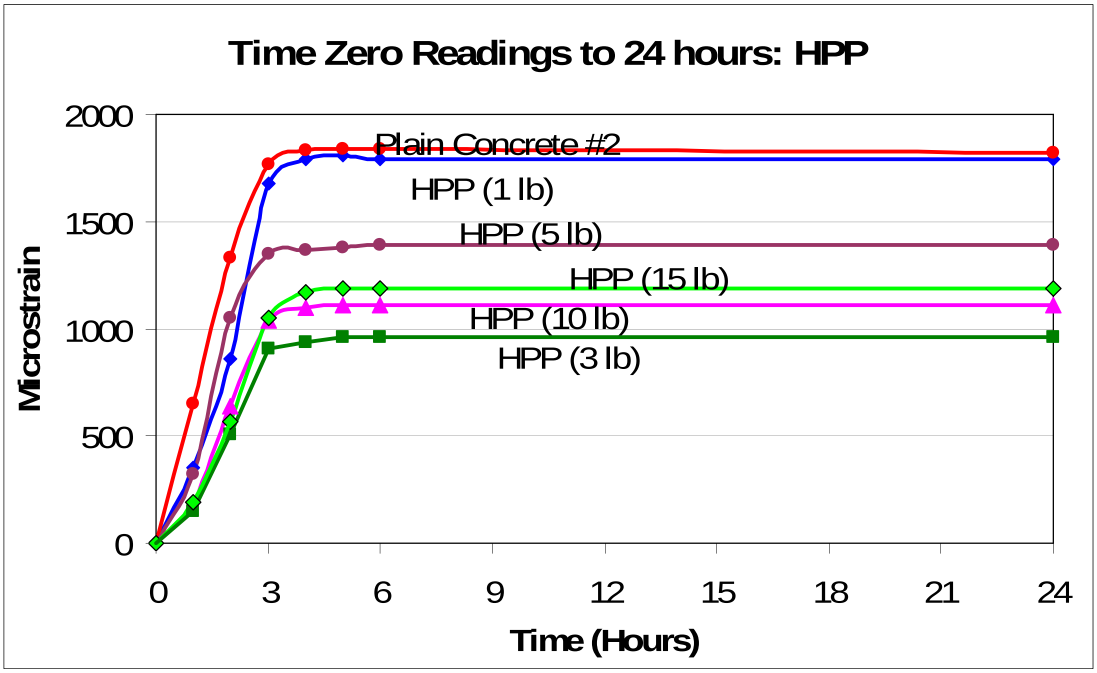

Figure 48: Time Zero shrinkage results: HPP

5.3.3 ASTM Unrestrained Shrinkage

The ASTM unrestrained shrinkage test is the industry standard test for determining shrinkage. It is normalized at 24 hours, so the plastic phase of the shrinkage has already been completed, and the shrinkage measured is drying and autogenous. The results of this test are important in evaluating long term shrinkage problems, but not early age cracking. The shrinkage at 28 days, shown for all the different batches in Figure 49, appears to show a significant decrease with the addition of fibers. However, the scatter in the results must be considered. The bars in the figure show the data range for each point. When the scatter is considered, it is apparent that there is little statistical difference in the results. At 1 lb, only HPP shows a statistically significant decrease. At the highest dosages, there seems to be perhaps a 20% reduction in long term shrinkage, but at most dosage levels the difference is negligible. Apparently, the fibers, with their low modulus of elasticity, do not do much to the shrinkage once the concrete's modulus of elasticity is significantly higher than the fibers'. Since this test only considers shrinkage after the concrete has hardened, the fibers probably should not impact the results much. A similar chart with 95% confidence intervals may be found in Appendix 3.

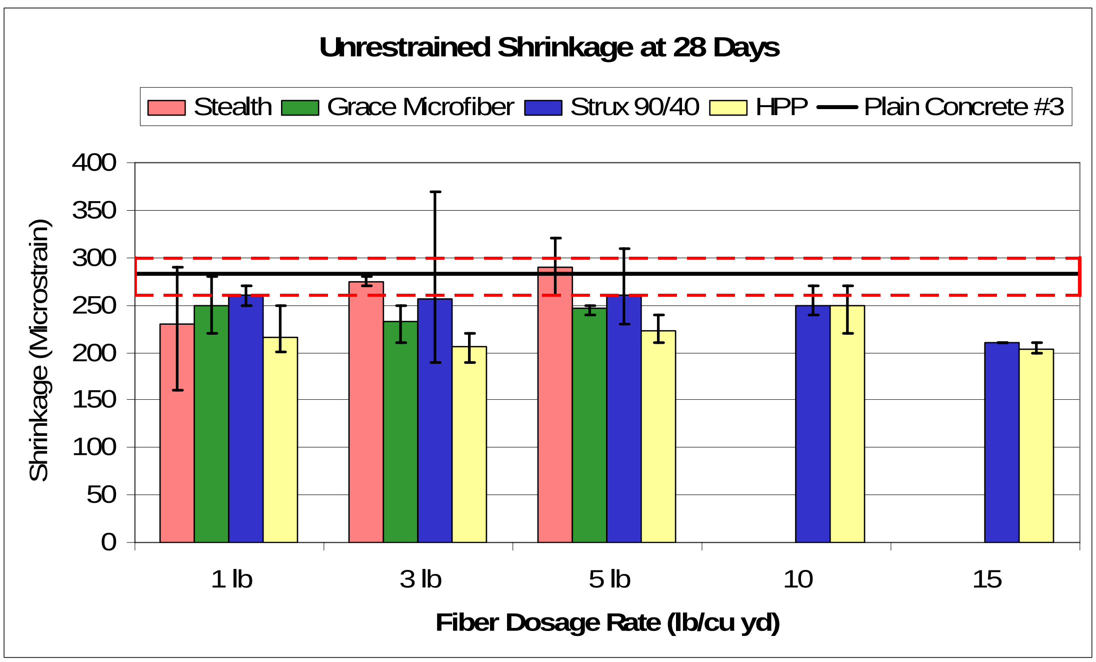

Figure 49: Unrestrained shrinkage at 28 days (bars show data range)

The only fiber that showed much truly significant unrestrained shrinkage benefits at all was the HPP fiber. Figure 50 shows the full curves for all 5 batches. Four of the five batches showed statistically significant reduction with the addition of fibers; the 15 lb dosage showed the best results. Interestingly, there was not a clear trend with increasing dosage rates—rather, adding any amount of fiber had about the same effect. Appendix 3 has full curves for all of the batches.

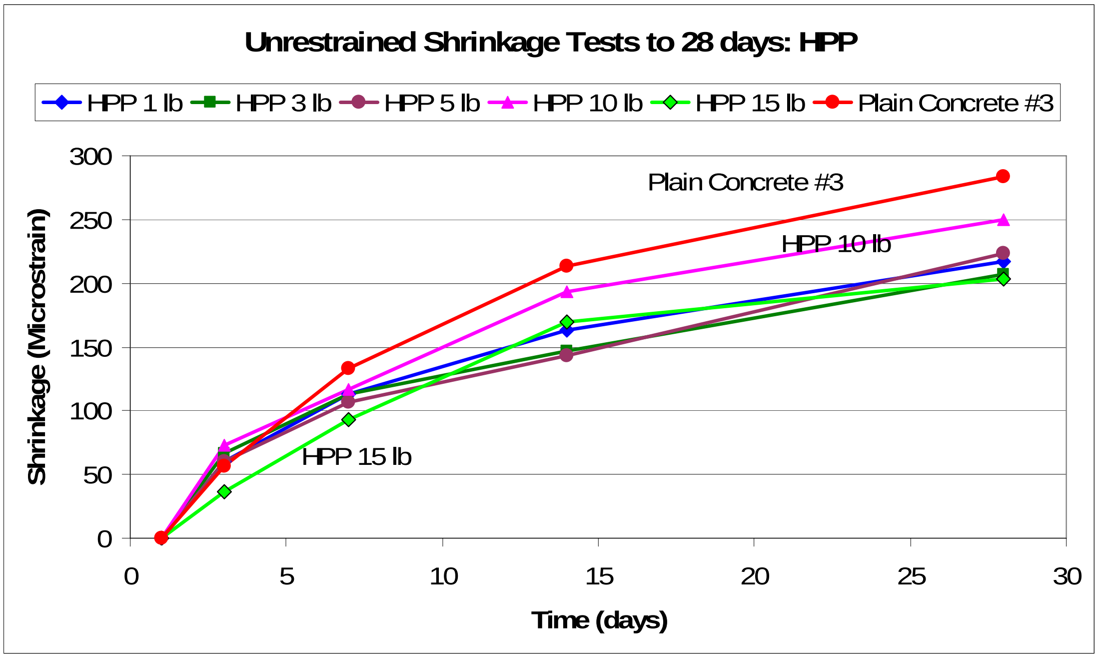

Figure 50: Unrestrained shrinkage curves: HPP

5.4 Plain Concrete Control Mixes

Two plain concrete control mixes were batched. To help verify the results, two plain concrete control mixes batched by Jen Teck Kao (2005) are referred to here as well. The procedures and the mix used were as nearly the same as possible—this work was an extension of the work by Kao, and the author worked on that project as well. The only difference between Kao's mix and the ones presented here was the origin of the fly ash. Kao used Red Rock fly ash, the fly ash used here was from Ash Grove, which can have a large impact on the results. Kao's primary control mix and secondary control mix from his two matrixes are presented here. Kao's secondary control mix is called PC#1, as it was batched at the beginning of this research project, though part of Kao's research.

5.4.1 Plain Concrete: Fresh Concrete Properties

The objective of this section is to identify the control values of this mix without fibers. The primary variables in this evaluation were the environmental variables, like air temperature, humidity, and aggregate water content. Table 15 lists the fresh concrete properties and conditions.

Table 15: Plain concrete fresh concrete properties and batch conditions

	PC JTK	PC #1	PC #2	PC #3
Slump (in)	6	3.5	4.5	3.25
Air Content (%)	2.2%	2.2%	3.4%	2.7%
Unit Weight (pcf)	150.96	152.4	149.24	150.12
Concrete Temperature	82	84	92	77.2
F. A. Moisture	4.21%	2.80%	1.44%	1.73%
C. A. Moisture	0.63%	0.39%	0.17%	0.21%
Air Temperature	77	86	88	54.5
Air Humidity	50%	58%	56%	43%

Initially, PC #2 was going to be the control mix for this research, but upon further evaluation the very high entrapped air content was noted, and the corresponding low unit weight. This may have had some impact on the shrinkage and strength results, so PC #3 was batched to replace it if necessary. PC #3 was batched in cooler conditions than any other batch in the matrix, so measures were taken to ensure that the concrete temperature was not far lower than the temperatures common in the matrix.

5.4.2 Plain Concrete: Shrinkage from Time Zero

The shrinkage from time zero test was the most important test to get a good baseline for, as it was a new test and relatively untested. In addition, the fact that only one sample was used for each batch increased the chances for errant results. Figure 51 gives the shrinkage from time zero of the four plain concrete batches. There is good correlation except for the PC #3 mix. Something went wrong with the testing apparatus, so the results were not used. Therefore, the PC #2 mix was chosen as the benchmark shrinkage from time zero for the rest of the matrix. Batches PC #1 and PC Jen Teck Kao used the old time zero testing apparatus, while PC #2 used the new version. PC Jen Teck Kao also used a different fly ash. It seems that PC #2 has slightly lower shrinkage than expected, but since it is the only option, it was the one chosen as benchmark.

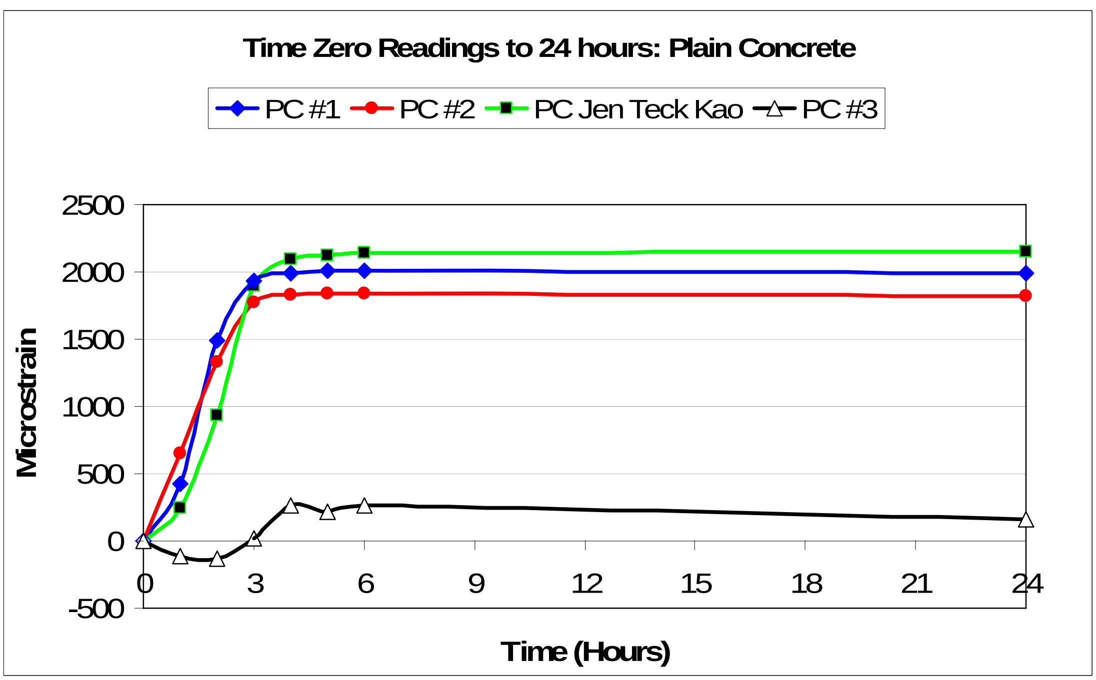

Figure 51: Plain concrete shrinkage from time zero

5.4.3 Plain Concrete: ASTM Unrestrained Shrinkage

The four plain concrete ASTM unrestrained shrinkage measurements were compared as well. Figure 52 gives the results for the mixes. The primary reason the PC #3 batch was done was because of the abnormally low shrinkage result from the PC #2 mix. The plain concrete #2 mix had a very high air content, and very high statistical variation from sample to sample. The range bars on the figure indicate the 95% confidence interval for each data point. PC #3 and PC Jen Teck Kao showed very tight results; the samples agreed well. PC #1 had only one sample tested, so the scatter was not known. PC #3 showed a curve that agreed better with the mixes of Jen Teck Kao, it had appropriate air content, and very little scatter of the data. For these reasons, PC #3 was chosen as the baseline mix for unrestrained shrinkage.

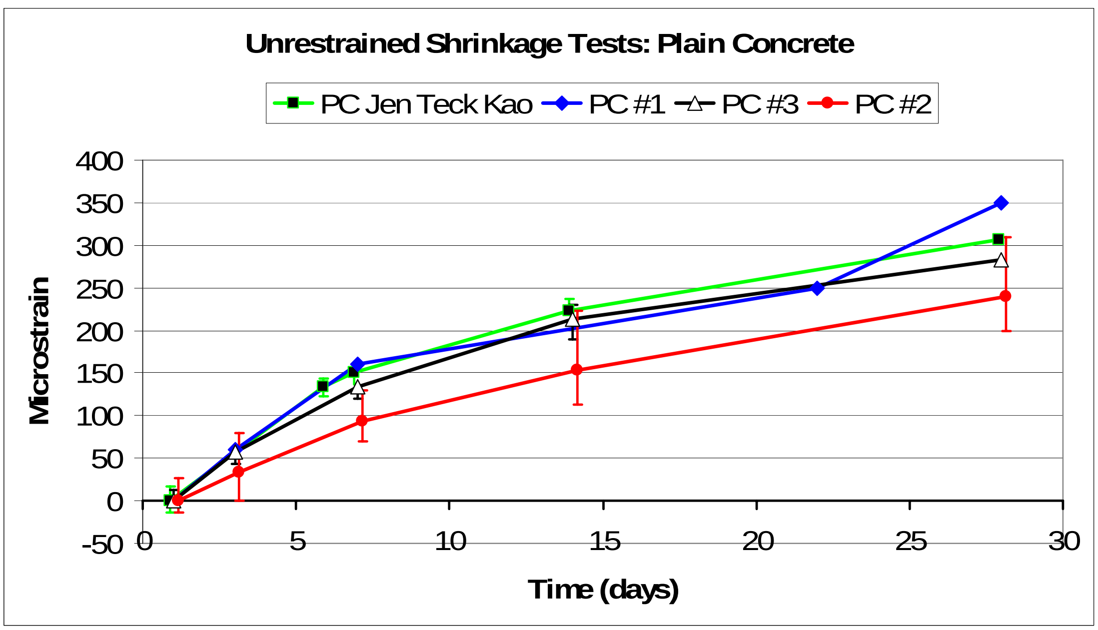

Figure 52: Plain concrete ASTM unrestrained shrinkage (bars show data range)

5.4.4 Plain Concrete: Compression Strength

There were three plain concrete mixes tested for compression strength; the PC #1 mix was not tested. There was good correlation between all three of the mixes, and the scatter was minimal. Figure 53 gives the results out to 28 days. The bars in the figure give the data range at each point. There were three cylinders tested at each point, except for the last on PC #3, where four cylinders were tested. The mixes all approached a value just over 6000 psi. Since PC #3 has shown the best and most consistent results elsewhere, PC #3 was used as the baseline mix for compression strength analysis as well.

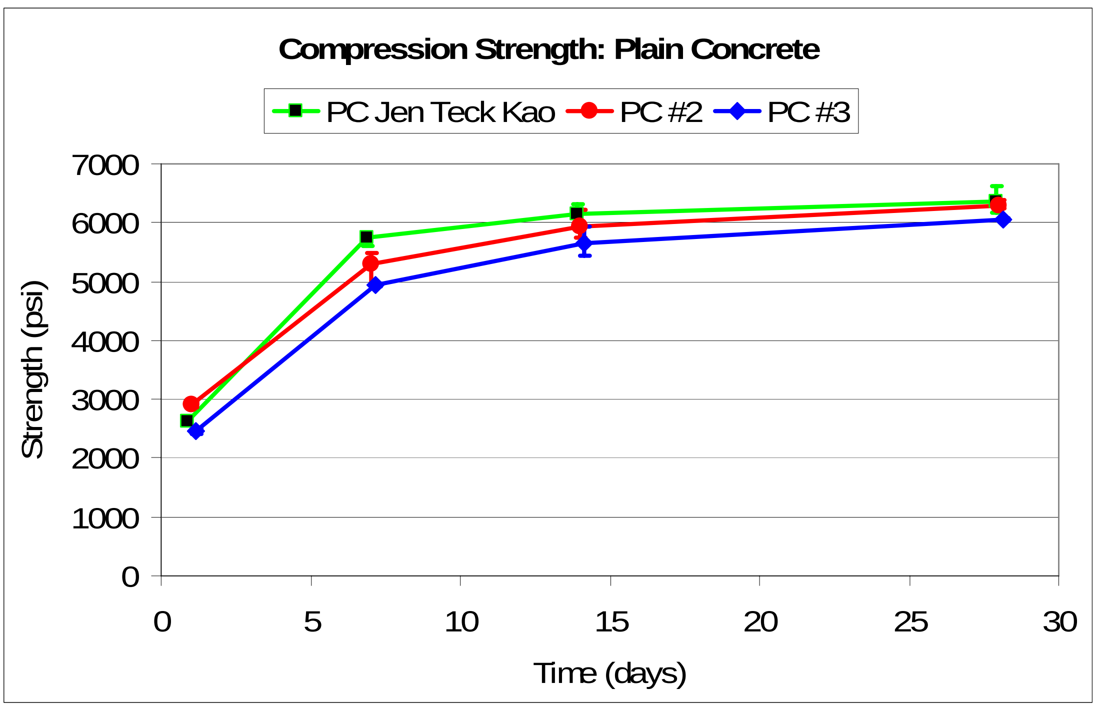

Figure 53: Plain concrete compression strength (bars show data range)

5.4.5 Plain Concrete: Splitting Tensile Strength

The splitting tensile strengths of the plain concrete mixes produced rather odd results (Figure 54). The results given by Jen Teck Kao seem to be problematic, compared to those found by this research project. The new plain concrete mixes (PC #2 and PC #3) correlate fairly well with each other, though the splitting tensile test usually has considerably more scatter than the compression test. In addition, all of the batches in the whole matrix (all of the batches with fibers) had 28 day splitting tensile strengths between 650 and 850 psi. Therefore, the results from Kao were discounted, and PC #3 used as the baseline. The results found in this test were another reason that the PC #3 was batched. The PC #2 results dropped from 14 to 28 days, and the scatter was very high at 28 days. The PC #3 mix behaved as expected, gaining a small amount of strength from 14 to 28 days. The PC #3 mix had less scatter and followed the expected trends better than the PC #2 mix for all tests except the time zero test, and had a more appropriate air content and unit weight. The PC #3 batch was used at the baseline for all tests except the time zero test, which failed for an unknown reason. In that case, PC #2 was used as the control mix.

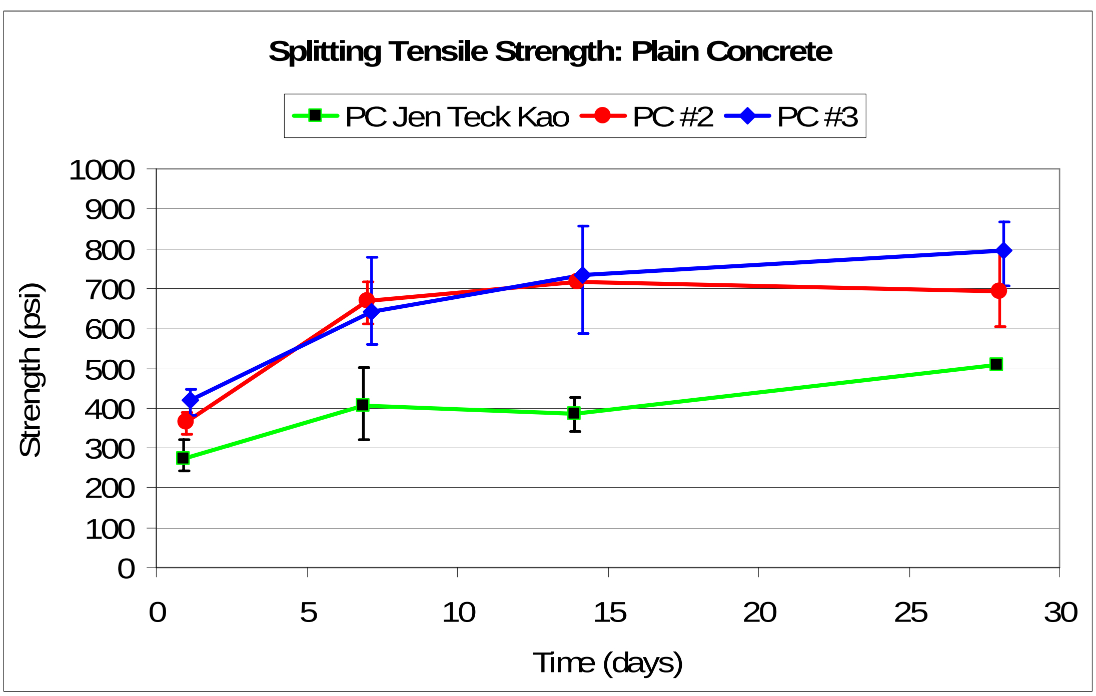

Figure 54: Plain concrete splitting tensile strength (error bars show data range)

5.5 Fiber Evaluation

One of the objectives of the research is to analyze each of the fibers and identify the optimum dosage for that fiber. First, a general overview of the fibers is given. Selected charts are presented here and additional charts can be found in Appendix 3.

5.5.1 General Survey of Fibers

When looking at the overall results, there are certain trends that are obvious: for example, the plastic shrinkage is greatly reduced by moderate to high dosages of fibers. Also, it is obvious that there is an optimum dosage point above which additional fiber is detrimental rather than beneficial. Long term shrinkage, compression strength, and splitting tensile strength, on the other hand, do not exhibit such obvious trends. The four primary tests are surveyed here.

5.5.1.1 Unrestrained Shrinkage from Time Zero

The shrinkage at 24 hours is the most important data point found by this test, as that connects to the long term shrinkage ASTM test. Therefore, the magnitude of the shrinkage of each batch at 24 hours is plotted in Figure 55.

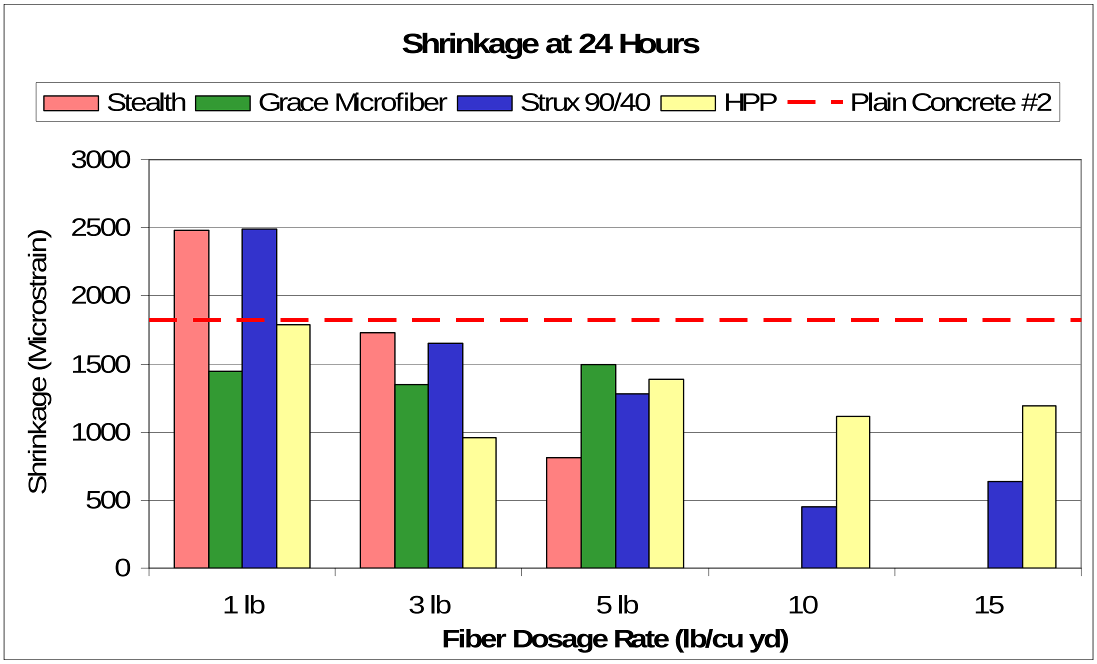

Figure 55: Unrestrained shrinkage from time zero: 24 hour readings

Obviously the plastic shrinkage, which is what is measured at 24 hours, is considerably reduced by fibers. The only fiber to exhibit uncertain results is Grace Microfiber, and that was discussed earlier (the 1 and 5 lb mixes had old molds, and thus potentially not as much of the shrinkage was measured). All other mixes show good reduction in early age shrinkage.

5.5.1.2 ASTM Unrestrained Shrinkage

The shrinkage results at 28 days from the ASTM unrestrained shrinkage test show the drying and autogenous shrinkage. Since the samples were zeroed at 24 hours, the plastic shrinkage is not shown. Twenty-eight days was chosen as the comparison point, as not all of the mixes were measured at longer term. Figure 56 gives the ASTM shrinkage results at 28 days.

There does not seem to be much of a clear trend anywhere with the ASTM unrestrained shrinkage test. Generally, the addition of fibers reduced the shrinkage by up to 10%, but for most batches the change was not statistically significant. Stealth fibers seem to increase the shrinkage somewhat, but all of the mixes are fairly close to the benchmark. It appears that much of the variation could be attributed to experimental scatter more readily than to a trend of importance. Whatever the case, the differences between the mixes here are small, and therefore this test is not considered important in selecting the optimum dosage rate for each fiber.

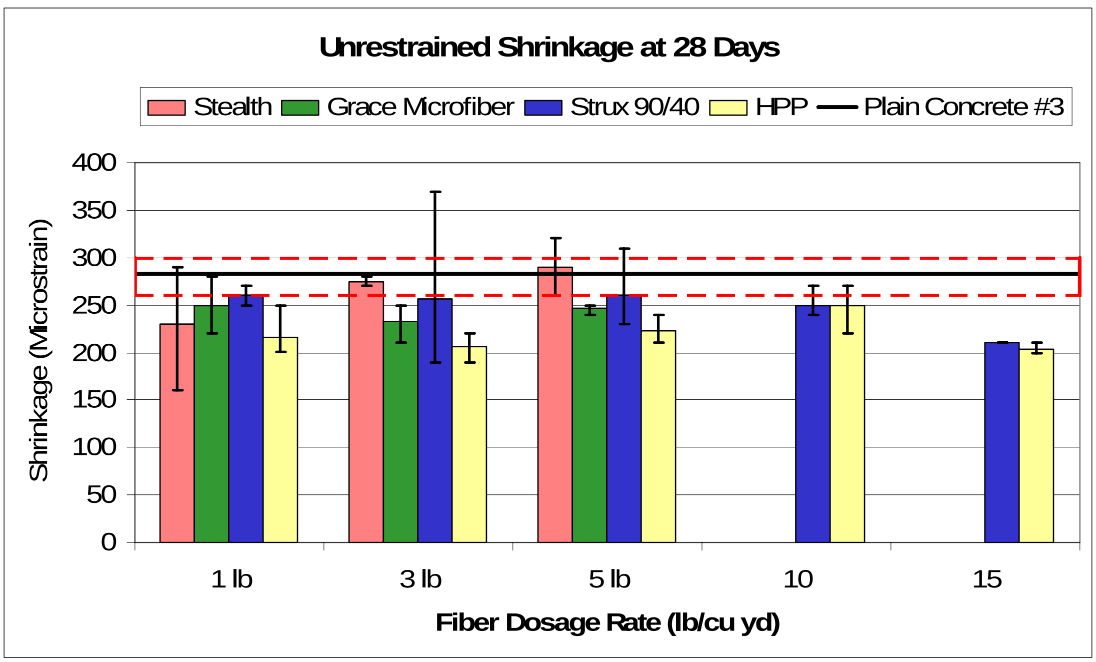

Figure 56: ASTM unrestrained shrinkage at 28 days (bars show data range)

5.5.1.3 Compression Strength

It has long been debated whether fibers modify the compression strength. As discussed in the literature review, it seems that most researchers feel the impact is slight, though there may be some effect at early age, before the concrete attains much strength. The main reason polymer fibers would not do much to compression strength is their low modulus of elasticity—they don't carry much load until the concrete cracks. What they add, then, is ductility upon failure. This was noted in the testing of the cylinders in compression—after failure the cylinders did not disintegrate, but rather held together. Ductility and potentially early age strength are the beneficial impacts of fibers on the compression strength of concrete. Was the same early age benefit seen in this project? Figure 57 gives the results at 24 hours.

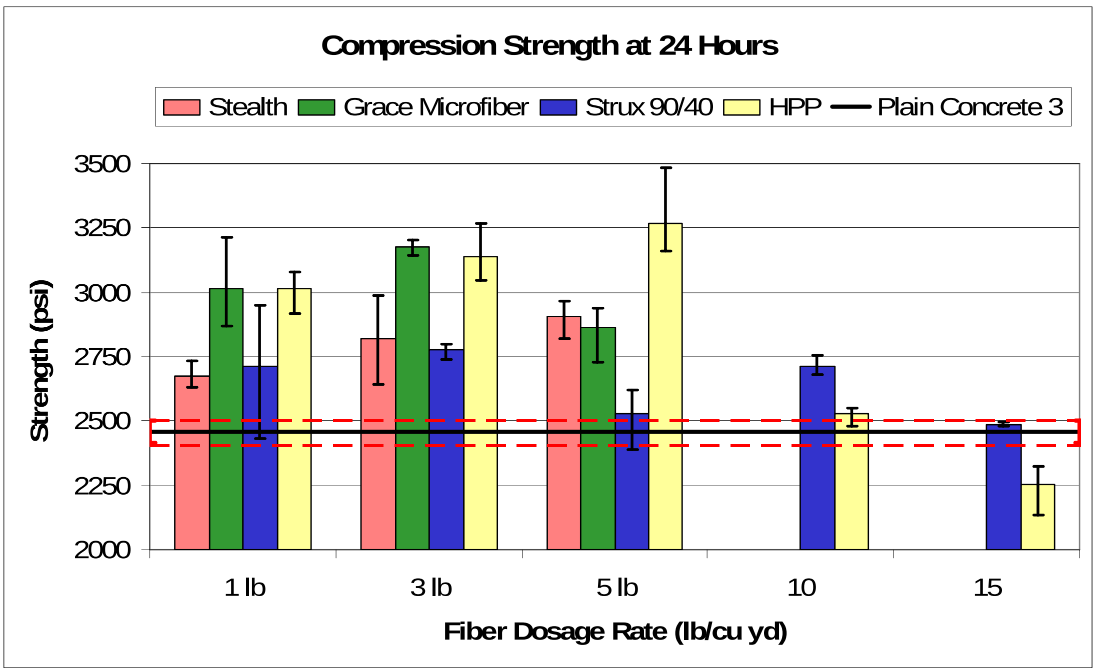

Figure 57: Compression strength at 24 hours (bars show data range)

As the figure shows, there was a significant increase in compression strength at 24 hours with the addition of fibers. Nearly all of the fiber dosage rates had an increase in strength that was statistically significant; some increased by as much as 750 psi. This test is very significant, and therefore will be used in determining the optimum dosage of each fiber.

What about 28 day strength? According to the literature, polymer fibers do not usually increase strength long term. This trend was continued in this research. Instead of increasing strength with increasing fiber dosage, there was a small detrimental effect. This is to be expected, as the fibers have a low modulus of elasticity, and thus behave more like air voids, carrying no load, than any type of reinforcement. Figure 58 gives the results of the compression testing at 24 hours.

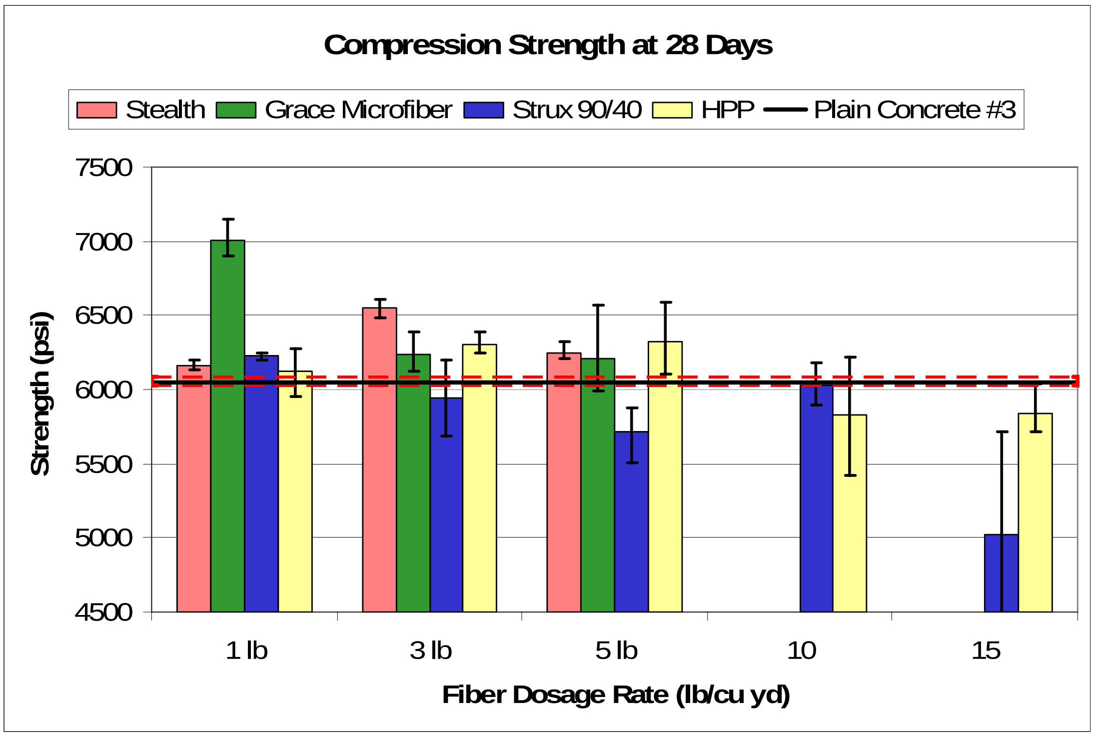

Figure 58: Compression strength at 28 days (bars show data range)

There is no significant unexpected trend here, though the microfibers do seem to behave differently than the macrofibers. The microfibers do not seem to decrease compression strength at 28 days, though the macrofibers show significant decline in strength at high dosage rates. Statistically, the macrofibers had high scatter, and thus little statistically significant change was found. The microfibers had less scatter, and several batches showed statistically significant increases in strength. Appendix 3 has similar figures with 95% confidence intervals shown for each batch.

5.5.1.4 Splitting Tensile Strength

The final test of importance in characterizing the mixes is the splitting tensile test. It has long been debated whether polymer fibers increase splitting tensile strength. The tensile strengths usually approximately mirror the compression test results, but in this research they often did not. The splitting tensile strengths at 24 hours do not show nearly as significant benefits with the addition of fibers as do the compression strengths. In fact, most fibers decreased the strength with a 1 lb dosage rate. Figure 59 gives the results at 24 hours.

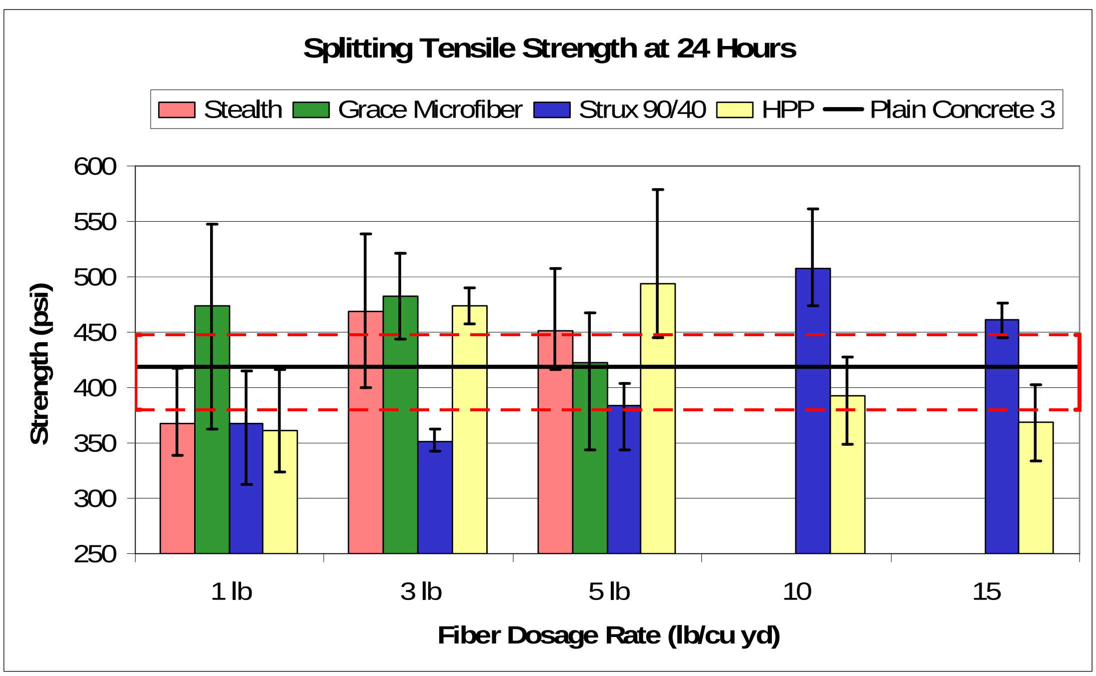

Figure 59: Splitting tensile strength at 24 hours (bars show data range)

All of the fibers exhibit some sort of curve with increasing dosage rates. Again, the microfibers did not decrease strength nearly as much as the macrofibers at most dosage rates. The best dosage rates for each fiber are fairly easy to spot in this test, as the curves are all formed without any apparent outliers.

A major issue with the splitting tensile test is the wide scatter commonly found. Because of this, very few batches ever showed statistically significant differences from the plain concrete control mix. It appears that using three samples is not enough to obtain solid results for the tensile strength. Nevertheless, trends are evident here, and will be considered in identifying the optimum dosage rate, though the statistical analysis indicates that the confidence in such findings is lower than might be hoped.

The tensile strengths at 28 days also exhibit substantial scatter, limiting the conclusions that may be drawn. It is fairly clear that fibers do not produce any significant increase in tensile strength at 28 days. Figure 60 gives the results at 28 days. There are several odd results here. Grace microfiber produced a curve exactly opposite what was expected to be found—it is not certain what the results mean. In addition, the HPP curve is approximately the reverse of the one at 24 hours, with the lowest readings at 3 and 5 lb per cubic yard. The other two fibers, on the other hand, produced curves similar to those seen at 24 hours. What these results mean is hard to say. Due to the very high variation, it is impossible to make a conclusion with much confidence. Appendix 3 has charts showing the 95% confidence intervals for these tests.

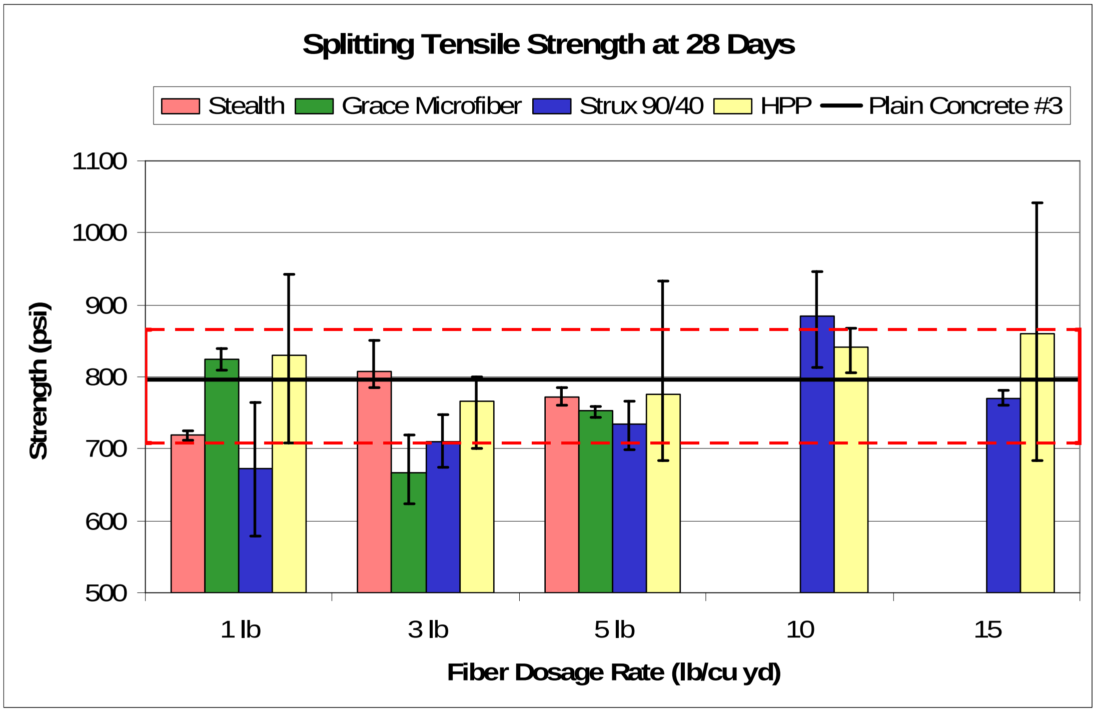

Figure 60: Splitting tensile strength at 28 days (bars show data range)

5.5.2 Stealth Optimum Dosage

The Stealth microfiber exhibited what may be considered "classic" microfiber behavior. The mix was dried out by the addition of the fibers, due to their very high surface area to volume ratio. This caused the slump to drop drastically with the addition of fibers (Figure 61). In addition, the shrinkage at 24 hours dropped significantly with increasing dosage. The workability significantly declined with the increase in fibers- the 5 lb dosage rate was very difficult to work with and hard to consolidate. This consistent trend downwards in shrinkage and workability is one of the expected trends. The other is with the strengths. Figure 62 gives the strengths at 24 hours. Both the compression and splitting tensile strengths exhibit a strong increase with the increasing dosage of fiber. By the 3 lb dosage rate, both compression and tensile strength are above the plain concrete values.

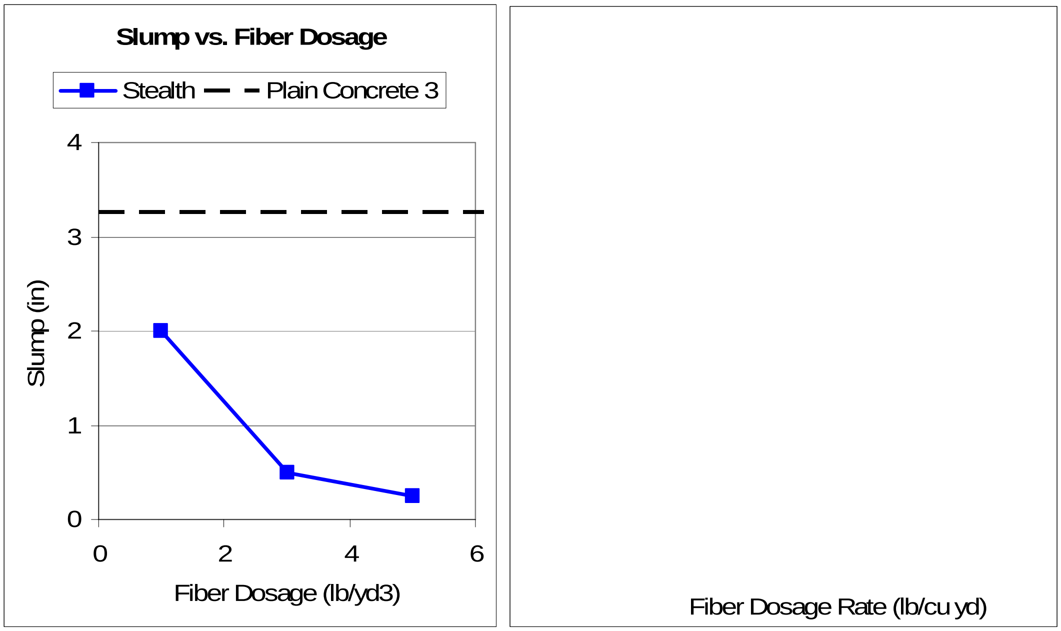

Figure 61: Stealth slump versus fiber dosage and shrinkage at 24 hours

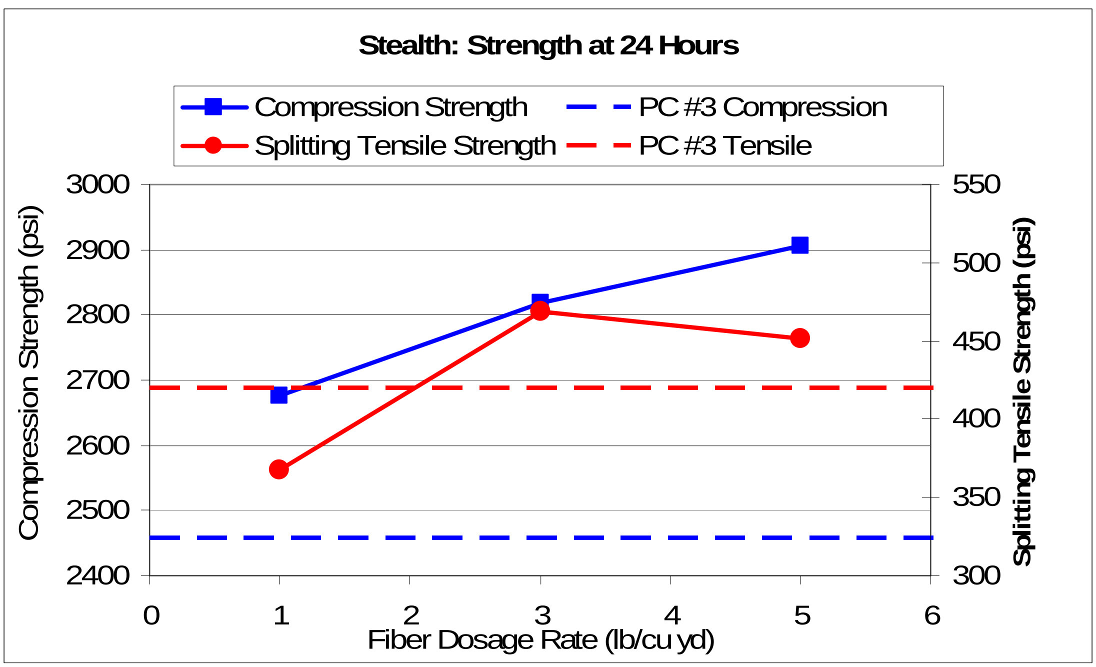

Figure 62: Stealth strength at 24 hours

The strength at 28 days shows a slightly different trend, and the optimum mix starts to become apparent (Figure 63). The 3 lb dosage rate is about 300 psi above the plain concrete in compression strength.

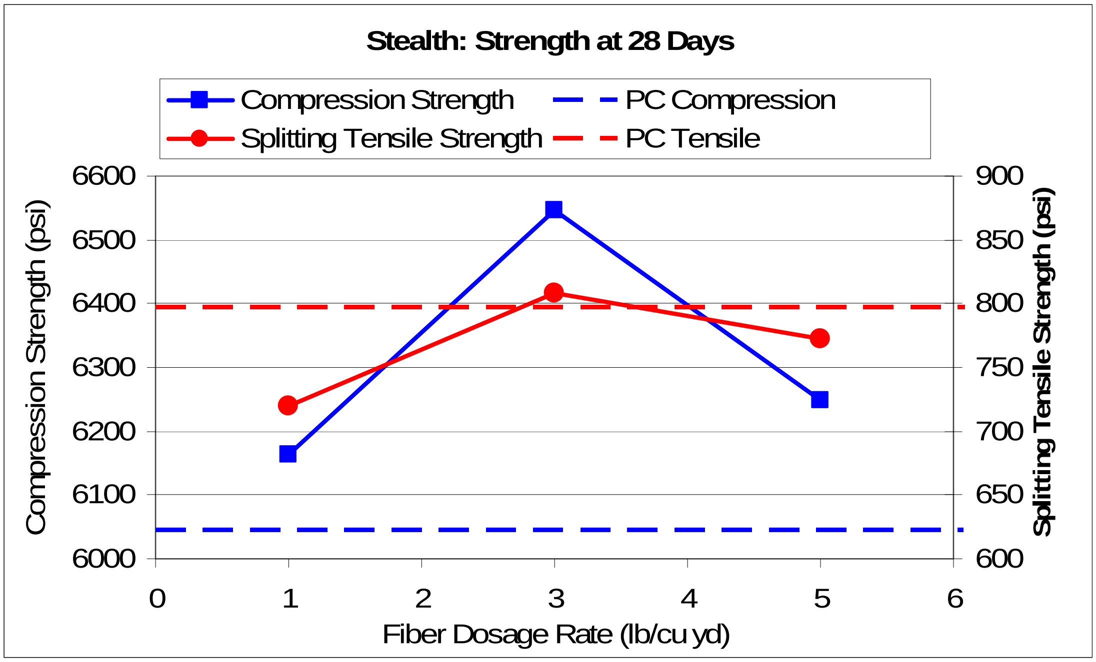

Figure 63: Stealth strength at 28 days

The one problem with Stealth is the loss of workability at about the same point as the benefits of the fiber start to be realized. For use in most situations, additional water or a water-reducing admixture would be required to compensate for the fiber's drying effect. This could negate the positive effects of Stealth. However, there are significant benefits from the addition of about 3 lb per cubic yard of Stealth fibers. Both early age and long term strength are increased, and plastic shrinkage is slightly reduced. The 3 lb per cubic yard dosage rate of Stealth fibers is close to the optimum dosage of Stealth fibers for the purposes mentioned here, but even so, the benefits are only moderate.

5.5.3 Grace Microfiber Optimum Dosage

The Grace microfiber showed less obvious results in this study: the trends were not clear, making an optimum dosage hard to determine. Again, the Grace fibers were microfibers, so increasing dosage of the fibers dried the mix out. In addition, the vast numbers of fibers formed a network in the concrete, contributing to the low slump (Figure 64). Workability was adversely affected by the addition of these fibers. Plastic shrinkage, on the other hand, was positively impacted by the Grace Microfibers. The 1 and 5 lb mixes were tested with the old unrestrained shrinkage from time zero molds, so the reading could be an underestimate of the shrinkage, but the 3 lb mix was tested with a new mold. This indicates that the 3 lb per cubic yard dosage had the lowest plastic shrinkage by a significant amount.

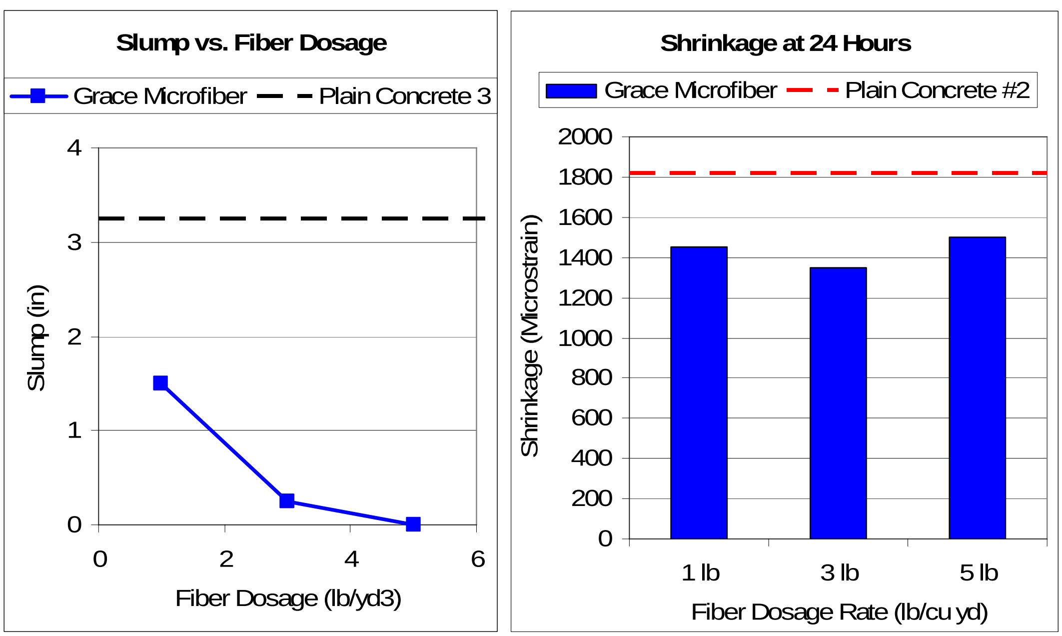

Figure 64: Grace Microfiber slump versus fiber dosage and shrinkage at 24 hours

The Grace Microfiber's strength at 24 hours continued the trend of good results with the 3 lb dosage. Figure 65 shows the compressive and tensile strength at 24 hours versus the results of the plain concrete control mix. The plain concrete results were all well below the Grace Microfiber results. The 3 lb mix, in particular, performed exceptionally: its compressive strength was over 700 psi higher than the control mix, and its tensile strength about 50 psi higher. The 1 lb mix also performed well.

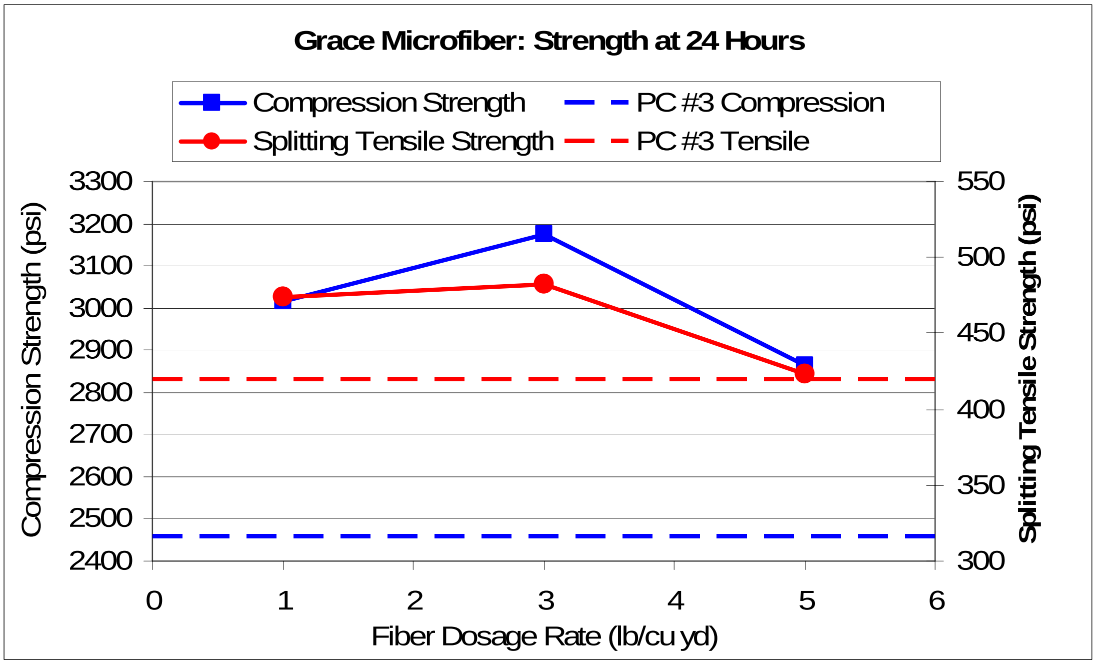

Figure 65: Grace Microfiber 24 hour strengths

The 28 day strength results (Figure 66) show a different trend entirely. The Grace Microfiber 1 lb mix had excellent results, but the 3 lb mix actually had slightly reduced strength. This is not altogether unexpected; as mentioned before, polymer fibers theoretically should not increase strength long term, based upon the relative modulus of elasticity. The results from the 1 lb mix are quite impressive, however, indicating that there might be some impact with microfibers at 28 days.

What is the optimum dosage of Grace Microfiber? For strength, 1 lb dosage is the most beneficial—solid improvement at 24 hours and very good improvement at 28 days. The best fiber dosage at reducing shrinkage is the 3 lb per cubic yard dosage rate. The best overall dosage rate for Grace Microfibers is probably 3 lb per cubic yard. The Grace Microfiber seems to have a significant beneficial impact at the appropriate dosage rates.

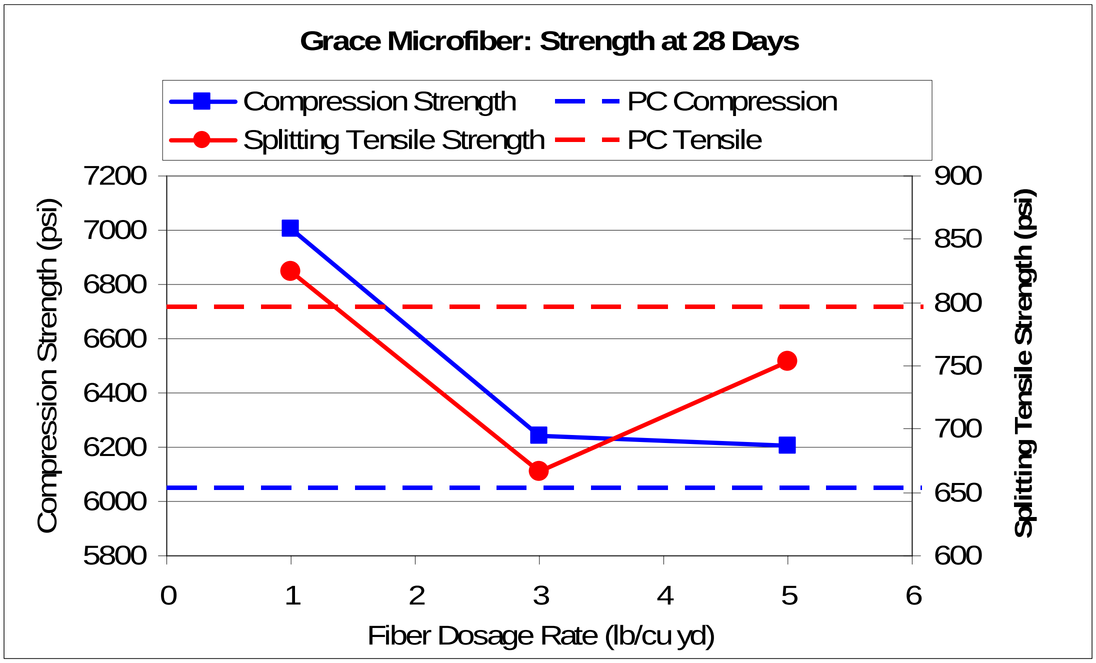

Figure 66: Grace Microfiber 28 day strengths

5.5.4 Strux 90/40 Optimum Dosage

The two macrofibers in this study have radically different shapes. Strux 90/40 is a thin, ribbon-like fiber, with no bending stiffness of note. This fiber was tested at dosage rates up to 15 lb per cubic yard, as the macrofibers do not dry out the mix, permitting higher dosage rates. The workability was not strongly affected, except in finishing. The higher dosage rates were hard to finish, as the fibers tended to stick out in all directions. The slumps decreased with increasing dosage rate (Figure 67), but even when the slump was very low, the mix was fairly easy to work with. Most of the slump reduction can be attributed to the physical network of fibers, rather than drying. At the higher dosages, the researchers did notice that the mixes became rather rough and rocky, making it harder to consolidate the concrete.

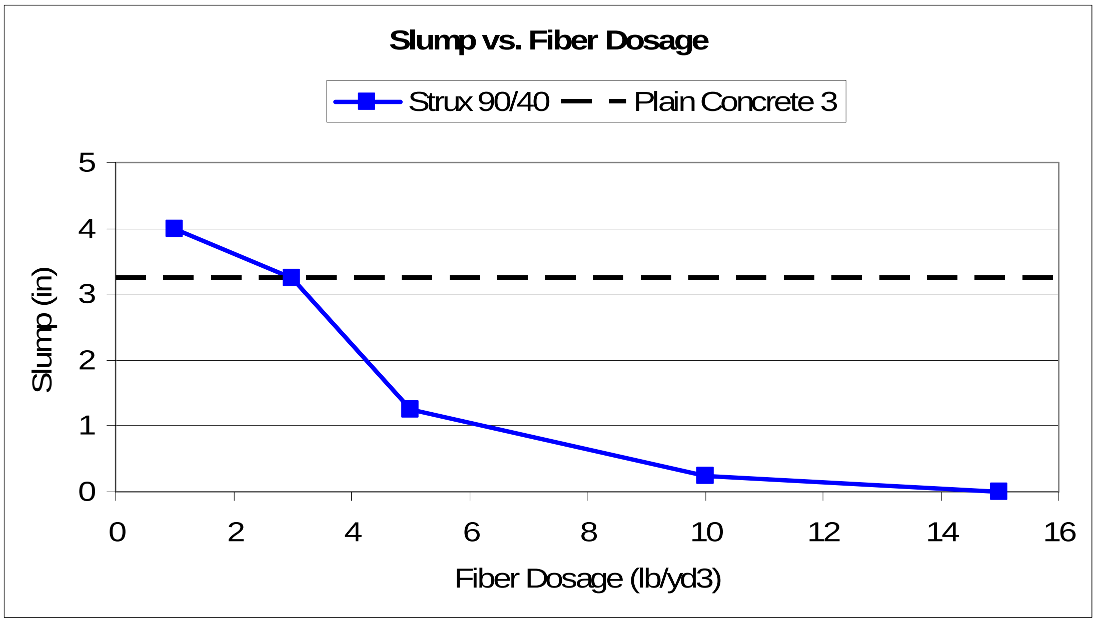

Figure 67: Strux 90/40 slumps

The plastic shrinkage, as read by the time zero test, revealed a very nice curve for the Strux 90/40 fibers (Figure 68). Shrinkage rose with the 1 lb mix, then decreased steadily down to the 10 lb mix, and rose again at the 15 lb mix. It appears, then, that the 10 lb mix is near the optimum dosage for reducing plastic shrinkage with Strux 90/40 fibers.

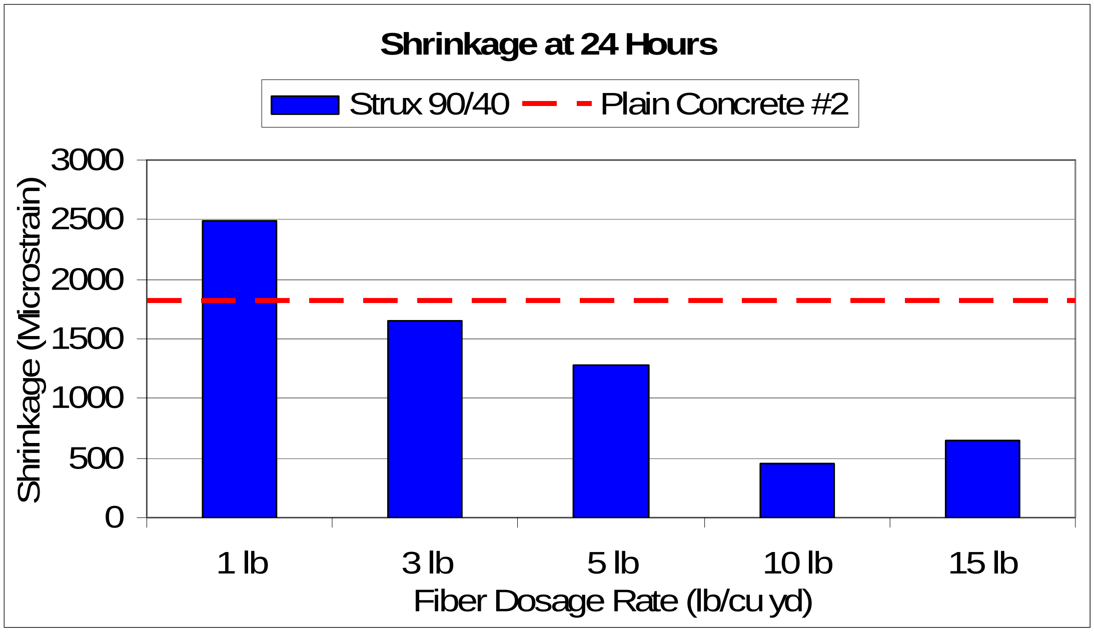

Figure 68: Strux 90/40 plastic shrinkage

The 24 hour strength of the Strux mixes (Figure 69) shows a strong increasing trend in splitting tensile strength. However, this trend starts well below the plain concrete strengths, so it is not until past the 5 lb dosage mix that the tensile strength passes the plain concrete value at 24 hours. The compression strength shows no clear trend whatsoever, but all mixes have strengths above the plain concrete control mix.

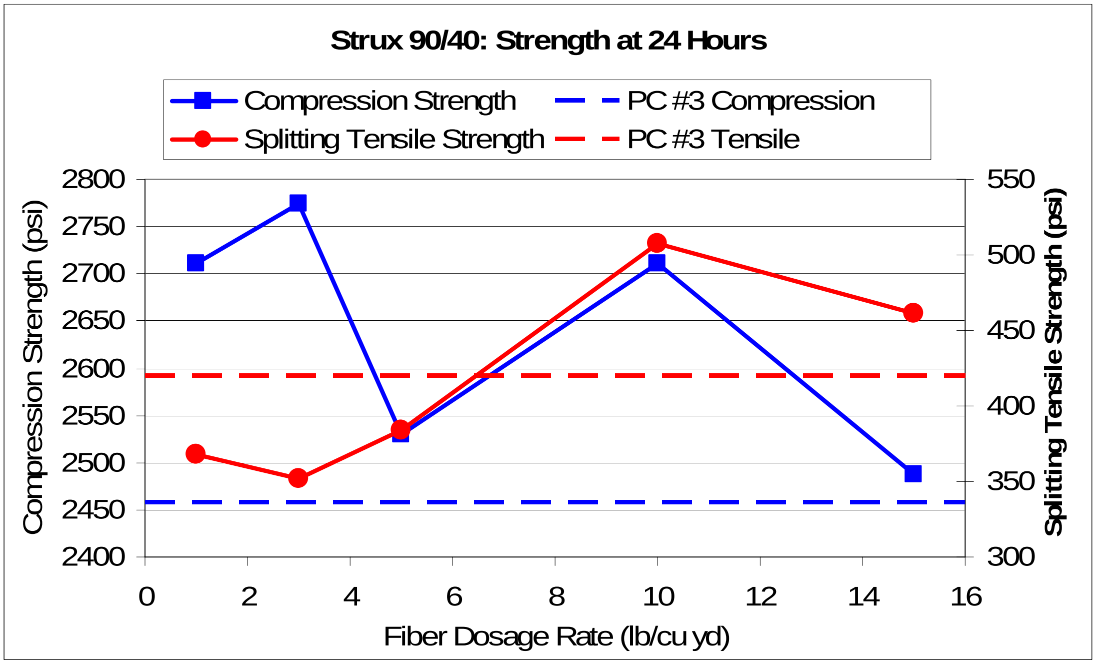

Figure 69: Strux 90/40 24 hour strength

The 28 day strengths show similar trends to the 24 day strengths: the splitting tensile strength clearly increases up to the 10 lb dosage mix, while the compression strength bounces around, showing no clear trend (Figure 70). However, all the compression strength results are well below the plain concrete strength at 28 days.

The optimum dosage for Strux 90/40 appears to be near 10 lb per cubic yard. The 10 lb mix has the best plastic shrinkage and tensile strength results, and decent compression results. The manufacturer recommends between 3 and 11.8 lb per cubic yard dosage, so this falls toward the upper end of those recommendations. The Strux fiber shows a larger beneficial impact than either Stealth or Grace Microfiber; the only weaker area is 28 day compression strength.

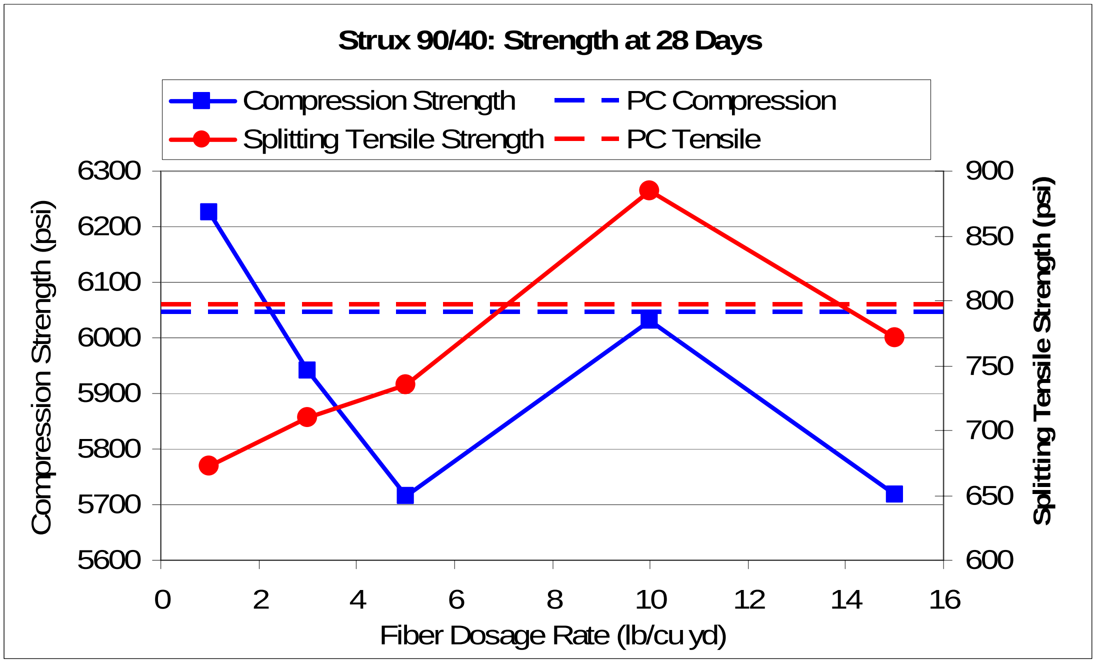

Figure 70: Strux 90/40 28 day strength

5.5.5 HPP Optimum Dosage

The high performance polymer (HPP) fibers are a different type of macrofiber than the Strux. They are shaped more like wires, and have significant bending stiffness. The fibers themselves are fibrillated to help with pull out resistance. Like the Strux, the fibers do not dry out the mixes significantly, but they do make the mix hard to finish. The slumps of the HPP mixes (Figure 71) show a general trend downward, but the slump never approaches zero. This fiber was the coarsest fiber tested, and this was reflected in the slump—even if the dosage were higher than 15 lb per cubic yard, it is doubted whether the slump will ever drop to zero.

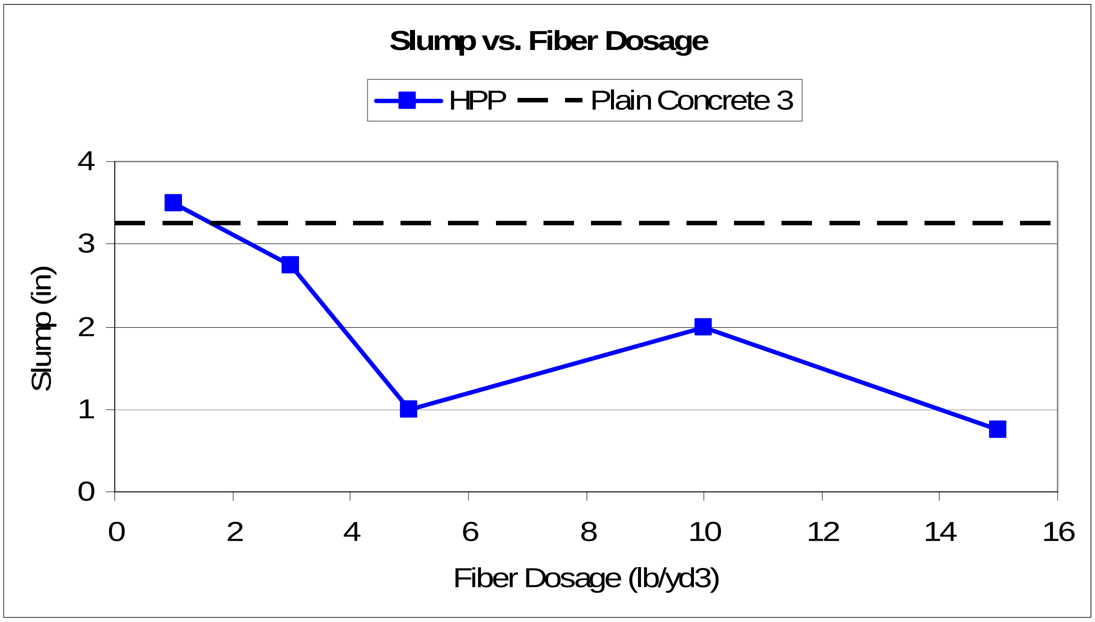

Figure 71: HPP slumps

The plastic shrinkage results (Figure 72) showed no strong trend. The 1 lb mix did not have decreased shrinkage, but the four other mixes did have significantly decreased plastic shrinkage. The lack of differentiation here means that the strength will be the deciding factor on what fiber dosage is the optimum one for HPP.

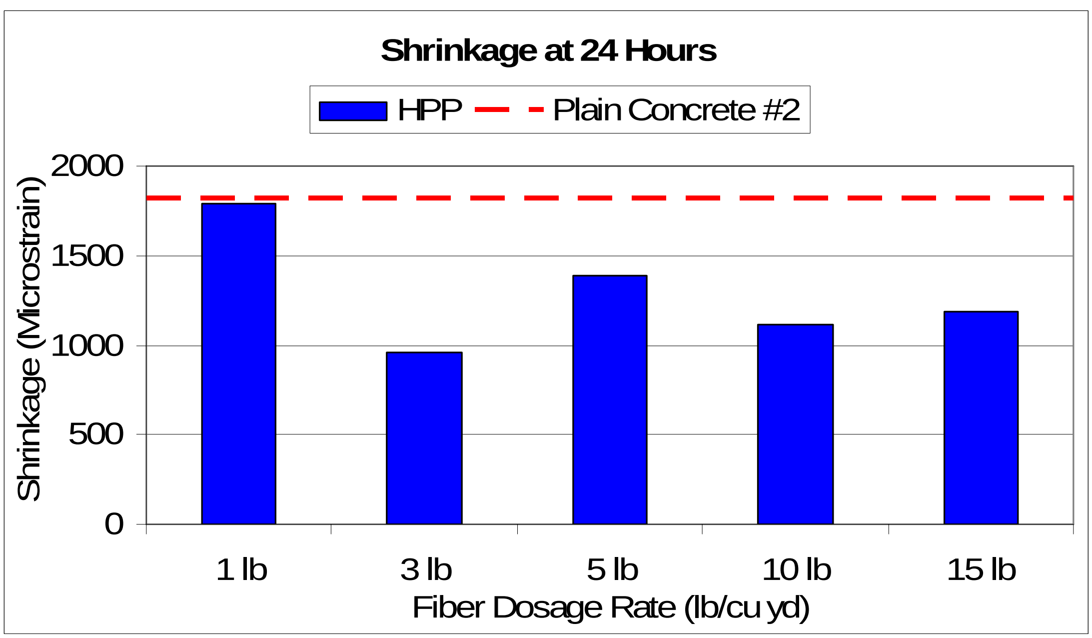

Figure 72: HPP plastic shrinkage results

The 24 hour strengths for HPP (Figure 73) show strong trends for both tensile and compressive strength. The strengths climb to the 5 lb dosage mix, and drop thereafter. The 1, 10, and 15 lb mixes do not show significant strength benefits at 24 hours, but the 3 and 5 lb mixes do.

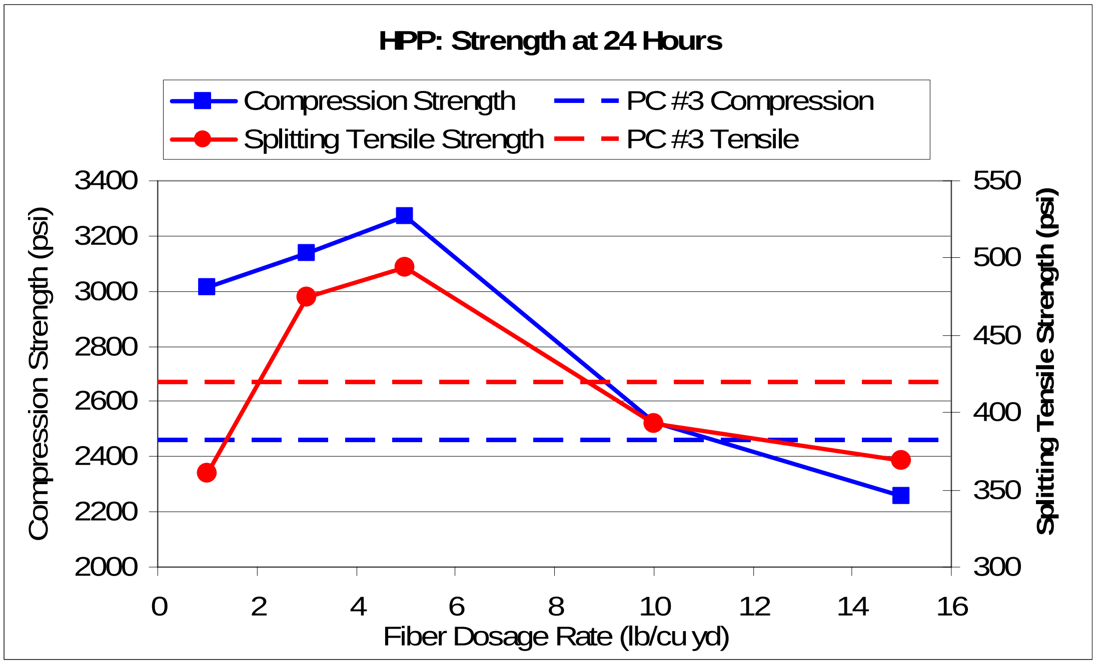

Figure 73: HPP 24 hour strengths

The final criterion to consider is the 28 day strength, shown in Figure 74. Here, the compression strength of the mixes is, for the most part, below that of the plain concrete. The 5 lb mix has the highest compression strength. Oddly, the splitting tensile strengths show exactly the opposite trend from the compression strength. However, all of the tensile strengths are significantly above the plain concrete control mix.

The optimum dosage for the HPP fibers seems to be either 3 or 5 lb per cubic yard. The 3 lb mix showed better plastic shrinkage results, and both mixes showed generally good results on in the strength evaluations. HPP showed better strength results than any other fiber in this study, but the Strux fiber showed better plastic shrinkage reduction.

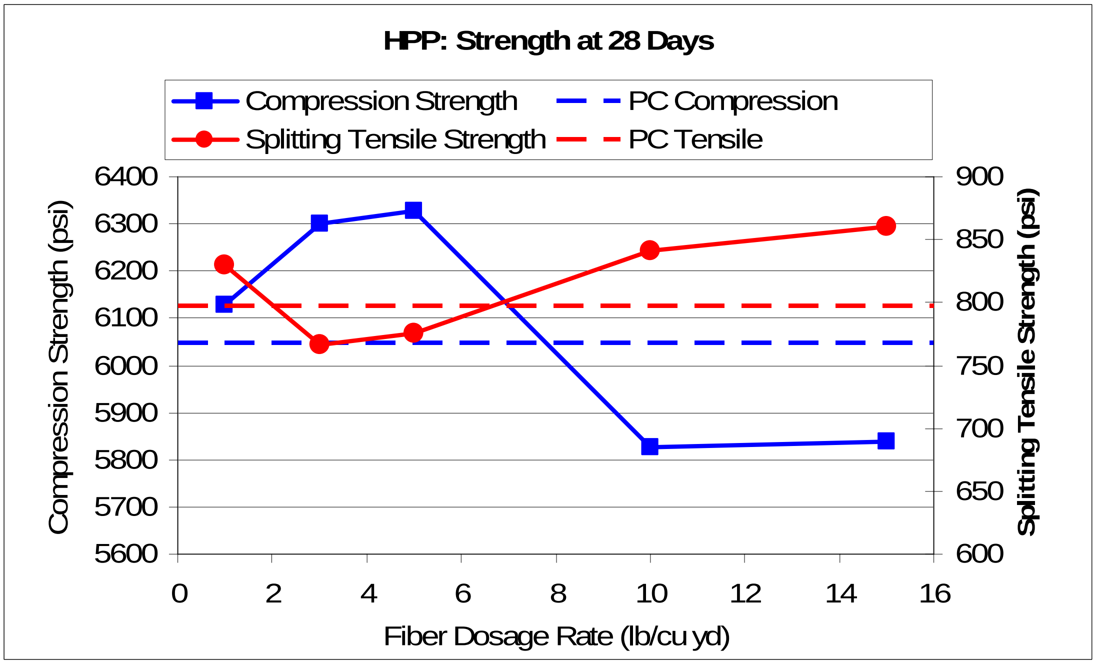

Figure 74: HPP 28 day strength

5.6 Microfiber and Macrofiber comparison

This research tested two microfibers and two macrofibers, and this section attempts to provide some trends associated with the two classifications. This is not meant to imply that all fibers of those classifications will follow the trends seen; these are simply observations. It must also be remembered that there was considerably experimental scatter in some of these tests (like the splitting tensile tests), so further testing is required to verify those results.

Table 16 presents the trends on a metric by metric basis. The microfibers had a more significant detrimental impact on the workability of the mixes than the macrofibers. The microfibers quickly reduced the slump close to zero. The macrofibers made finishing more difficult, but this was not as problematic as the drying caused by microfibers. The macrofibers also had excellent plastic shrinkage reduction. Drying shrinkage was reduced more by the microfibers than macrofibers. The microfibers and macrofibers had different impacts on the strengths of the concrete mixes. Both helped compression strength at early age, the microfibers somewhat more so. At 28 days, neither helped with compression strength or tensile strength much. Each class of fibers has different strengths in general, helping in different areas. Overall, the macrofibers seemed to have slightly better performance, but they also require higher dosage levels to reach optimum performance.

Table 16: Macrofiber and microfiber comparison

Property	Macrofibers	Microfibers
Slump decrease	Moderate decrease	Major decrease
Finishing difficulty	Much more difficult	Slightly more difficult
Drying of mixture	Minimal drying	Major drying
Plastic shrinkage	Major decrease	Moderate decrease
Drying (long term) shrinkage	Slight decrease	Moderate decrease
24 hour compression strength	Moderate increase	Moderate increase
24 hour tensile strength	Slight increase	Slight increase
28 day compression strength	Minimal change	Slight increase
28 day tensile strength	Minimal change	Slight decrease

5.7 Impact of Fibers: Summary

Fibers have been shown to help in several areas of concrete performance. The actual impact depends on the type and dosage rate of fibers. As discussed above, the macrofibers and microfibers have different effects on the behavior of the concrete mix. Table 17 below is meant as an evaluation of the general benefits that may be expected with the optimum dosages of these polymer fibers. Workability is reduced with the addition of polymer fibers, no matter the dosage rate and fiber type. With an appropriate dosage rate of fibers, plastic shrinkage can be cut in half or better. Drying shrinkage was reduced slightly with the addition of fibers. Twenty-four hour strengths were increased somewhat when optimum dosage rates of fibers were added. The twenty-eight days strengths were not influenced significantly by the addition of the polymer fibers.

Table 17: General impact of fibers

Property	Impact of Fibers
Workability	Moderate reduction
Plastic shrinkage	Major reduction
Drying shrinkage	Slight reduction
24 hour compression strength	Moderate increase
24 hour tensile strength	Slight increase
28 day compression strength	No impact
28 day tensile strength	No impact

Chapter 6: Conclusions

Bridge decks have problems with cracking. These problems are caused to a large extent by thermal movement, early-age shrinkage, and early age settlement. All three of these issues may be counteracted by the addition of polymer fibers. Polymer fibers also assist in reducing crack widths after cracking.

Macrofibers and microfibers behave differently, and should be treated differently. Microfibers affect workability by drying the mix out; macrofibers by making finishing difficult.

Low to moderate dosages of fibers improve early age compression strength significantly, but 28 day compression strengths are not influenced much.

The addition of fibers slightly increases 24 hour splitting tensile strengths; 28 day effects are insignificant.

Fibers slightly decrease ASTM unrestrained shrinkage results, measured from 24 hours to 28 days.

Fibers drastically reduce early age shrinkage, depending on the dosage level; higher is better, up to a certain point that is different for each fiber.

Fibers dramatically change failure types; all failures were more ductile.

The optimum dosage rate for Stealth fiber seemed was approximately 3 lb per cubic yard; the benefits were moderate. Grace Microfiber's optimum dosage rate was 3 lb per cubic yard, and the benefits seen were significant. The best dosage rate for Strux 90/40 was about 10 lb per cubic yard, and that dosage showed exceptionally good plastic shrinkage benefits, greater than any other mix in this research. Finally, the HPP fiber had its optimum dosage rate at either 3 or 5 lb per cubic yard, and had the best strength results in this study.

The unrestrained shrinkage from time zero test performed excellently. This test allowed good quantitative measurements to be made of plastic shrinkage starting at the batch time. The results correlated well with the ASTM unrestrained shrinkage test.

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