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HIGH-PERFORMANCE BRIDGE CONCRETE

Sponsored by

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16. Abstract

Construction of a High-Performance Concrete (HPC) Showcase Bridge in Alabama and the materials research and testing performed in conjunction with that project are covered.

Tests were conducted to determine short-term and long-term strength and durability properties of the HPC used in the cast-in-place concrete and precast, prestressed girders. Comparisons are made between performance characteristics of HPC and standard concrete. Results show that the HPC mixtures used were superior to the standard concrete in both strength and durability properties.

The effects on strength of initial and final curing conditions, cylinder size, and method of capping of concrete test cylinders were evaluated. Core specimens were obtained and tested to compare strengths resulting from the use of AASHTO and ACI testing procedures with strengths obtained from match curing of standard cylinders. An alternate method of cutting and testing core samples immediately was also studied. Strengths measured using the alternate method compared well with strengths of match-cured cylinders.

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SUMMARY

Bridges with prestressed concrete girders and reinforced concrete decks are common in new bridge construction due to their low initial cost relative to other bridge systems. In recent years the Federal Highway Administration (FHWA) has stimulated the development and implementation of High-Performance Concrete (HPC) to improve these popular concrete bridges. A focus of the development of HPC is to produce not only higher strength concrete, but more durable concrete. To provide examples of successful implementation of HPC in real bridges, FHWA through state transportation departments has founded Showcase Bridge projects. A part of each Showcase Bridge project is the construction of a bridge using HPC. Documentation of the construction of Alabama's HPC Showcase Bridge is presented in this report along with discussion of the production and performance of the HPC utilized and concrete testing procedures.

Field and laboratory tests were conducted during the project to determine short-term and long-term strength and durability properties of the HPC used in cast-in-place and precast, prestressed girder applications. Comparisons were made between the performance characteristics of the HPC and standard concrete mixes to provide evidence of the usefulness and practicality of HPC.

Results from field and laboratory tests indicated that the HPC mixes in this project were superior to the standard concrete mixes in both strength and durability properties. Use of these HPC mix designs instead of standard concrete mix designs in both precast, prestressed girders and cast-in-place

bridge decks allows designers to optimize designs by ensuring long-term durability and performance.

The effects of initial and final curing conditions, cylinder size, and method of capping concrete test cylinders were evaluated to determine the effects on measured compressive strength of concrete used in precast, prestressed concrete girders. Core specimens were obtained and tested to compare various testing procedures with match cured cylinder strengths.

Results from the evaluation of initial and final curing methods, cylinder size comparison, and capping method comparisons indicate that the procedures used to make and test concrete test cylinders can have a significant impact on the perceived strength of the concrete. A reasonable combination of testing procedures are recommended for use in production of high-strength HPC girders. Compressive strength results from core testing of precast, prestressed HPC girders compare well to tests of match-cured cylinders. These results are contrary to the opinions of some previous researchers who have concluded that core strengths can practically never exceed 70 to 85% of the cylinder strength.

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CHAPTER ONE

INTRODUCTION

BACKGROUND

Bridges with precast, prestressed concrete girders and reinforced concrete decks are common in new bridge construction due to a lower initial cost relative to other bridge systems and relatively low maintenance costs through the life of the structure. In recent years the Federal Highway Administration (FWHA) has stimulated the development and implementation of High-Performance Concrete (HPC). The use of HPC in bridge design offers a way to utilize higher compressive strength while ensuring long-term durability in these already popular bridges. Increased span lengths and fewer structural components result in cost savings during construction, while the bridge's longer service life results in lower life-cycle costs.

The terminology "high-performance concrete" refers to concrete designed, or engineered, to meet specified strength and durability requirements, both short-term and long-term, needed for the structural and environmental requirements of the project. Not only is higher compressive strength considered in the design process, but also increases in service life resulting from improved durability characteristics such as permeability, freeze-thaw resistance, and resistance to scaling and abrasion.

Superior durability characteristics of HPC can typically be achieved by using the same materials as conventional concrete mixes. HPC mix designs typically utilize low water-to-cementitious materials (w/cm) ratios, chemical admixtures, and pozzolanic materials to achieve superior strength and durability characteristics over standard concrete mixes. To achieve these superior characteristics, some special techniques are needed for mixing, placing, finishing and curing. Rigorous quality control and quality assurance (QC/QA) steps are generally needed in the production of HPC to ensure the performance of the mix.

OBJECTIVE

The primary objective of the work reported here was to document the construction of the HPC bridge project located on Alabama Highway 199 while addressing topics regarding production and performance of the HPC mixtures and concrete testing procedures. Special Provisions to the ALDOT Standard Specifications were developed for the HPC Bridge Project. Testing was performed to verify that those Special Provisions actually resulted in high-performance concrete being supplied by the Contractor. Comparisons are made in this report between short-term and long-term performance characteristics of the HPC mixtures used by the Contractor and of standard concrete mixture from other ALDOT projects to address the question of whether HPC is better concrete.

METHOLOGY AND SCOPE

The project objectives stated above were met through tests and observations during construction of Alabama's HPC bridge project located on Alabama Highway 199 in Macon County, Alabama. Properties of a standard cast-in-place concrete mix from a typical state project located in Opelika, Alabama were compared with properties of the HPC mix. Properties of a standard concrete girder mix were determined for concrete from a production pour made by the precast concrete producer that supplied the HPC girders. The properties of the standard concrete girder mix were compared with the properties of the HPC girder mix.

Field and laboratory tests were conducted to determine short-term and long-term strength and durability properties of the HPC and standard concrete mixes. Performance tests were conducted with cooperation and assistance from ALDOT, Federal Highway Association (FHWA) Mobile Concrete Testing Lab, Auburn University and the contractors involved in the HPC project.

A review of other research concerning the production, performance and testing of HPC is presented in Chapter Two. Chapter Three discusses the construction of the HPC bridges. The production of the HPC girder and cast-in-place concrete is presented in Chapters Four and Five. The comparison and evaluation of the HPC and standard concrete properties for girder and cast-in-place applications are presented in Chapters Six and Seven. Chapter Eight discusses the evaluation of concrete production and testing procedures. Conclusions and recommendations are presented in Chapter Nine.

Tables and figures documenting the results of individual tests are provided in Appendices A and B. Results from standard pull-out tests and determinations of phosphate coating weights for the 0.6 inch diameter prestressing strand used in the HPC girders are provided in Appendices C and D. Temperature histories measured in the drilled shafts and bent caps of the Uphapee Creek Bridge are in Appendix E.

CHAPTER TWO

LITERATURE REVIEW

INTRODUCTION

A review of literature was conducted to examine HPC production and testing procedures. Findings from the review are presented in this chapter. Literature related to concrete testing and the effects on measured strength of cylinder size, capping method, and cylinder initial curing method are discussed. The use and role of pozzolans and admixtures in HPC mix design as well as other issues concerning production of HPC are addressed.

DEFINITION OF HPC

Individual researchers and groups have attempted to develop a definition for the term high-performance concrete. Based on the American Concrete Institute's (ACI) 1999 definition, "HPC is concrete meeting special combinations of performance and uniformity requirements that cannot always be achieved routinely using conventional constituents and normal mixing, placing, and curing practices." (Russell 1999). Commentary given on this definition indicates that HPC is concrete having certain characteristics developed for a particular application and environment. Examples of characteristics that may be considered critical for an application are:

1. Ease of placement
2. Compaction without segregation
3. Early age strength
4. Long-term mechanical properties
5. Permeability
6. Density
7. Heat of hydration
8. Toughness
9. Volume stability
10. Long life in severe environments

Goodspeed et al. (1996) proposed a definition based on long-term performance criteria for HPC that the FHWA is continuing to develop for bridges. The proposed definition consists of four durability and four strength parameters. Associated with each parameter are criteria for performance grades, testing procedures to measure performance, and recommendations to relate performance to adverse field conditions. The eight parameters are as follows:

1. Freeze-thaw durability
2. Scaling resistance
3. Abrasion resistance
4. Chloride penetration

5. Compressive strength
6. Modulus of elasticity
7. Shrinkage
8. Creep

The method for grading HPC based on performance characteristics of the concrete is given in Figure 2.1. Along with the table given in Figure 2.1, recommendations for the application of the HPC performance grades are given in Figure 2.2.

A Strategic Highway Research Program defined HPC as consisting of (Zia et al. 1991):

1. A maximum water-cementitious ratio of 0.35
2. A minimum freeze-thaw durability factor of 80 percent as determined by ASTM C 666, Procedure A
3. A minimum strength criteria of either
    a. 3000 psi within 4 hours after placement
    b. 5000 psi within 24 hours
    c. 10000 psi within 28 days

FHWA in a recent publication *High-Performance Concrete Bridges, Building Bridges for the 21st Century* (1998) states that HPC is concrete that has been designed to be more durable and if necessary, stronger than conventional concrete. HPC mixes are composed of essentially the same materials as conventional concrete mixes. But the proportions are designed, or engineered,

Performance characteristic[2]	Standard test method	FHWA HPC performance grade[3]				
		1	2	3	4	N/A
Freeze-thaw durability[4] (*x*=relative dynamic modulus of elasticity after 300 cycles	AASHTO T 161 ASTM C 666 Proc. A	$60\% \le x \le 80\%$	$80\% \le x$			
Scaling resistance[5] (x=visual rating of the surface after 50 cycles)	ASTM 627	x=4,5	x=2,3	x=0,1		
Abrasion resistance[6] (x=avg. depth of wear in mm)	ASTM C 944	$2.0 > x \ge 1.0$	$1.0 > x \ge 0.5$	$0.5 > x$		
Chloride penetration[7] (x=coulombs)	AASHTO T277 ASTM 1202	$3000 \ge x > 2000$	$2000 \ge x > 800$	$800 \ge x$		
Strength (x=compressive strength)	AASHTO T 2 ASTM C 39	$41 \le x < 55$ MPa ($6 \le x < 8$ ksi)	$55 \le x < 69$ MPa ($8 \le x <$ ksi)	$69 \le x < 97$ MPa ($10 \le x < 14$ ksi)	$x \ge 97$ MPa ($x \ge 14$ ksi)	
Elasticity [10] (x = modulus of elasticity)	ASTM C 469	$28 \le x < 40$ GPa ($4 \le x < 6x10^6$ psi)	$40 \le x < 50$ GPa ($6 \le x < 7.5x10^6$ psi)	$x \ge 50$ GPa ($x \ge 7.5x10^6$ psi)		
Shrinkage[8] (x=microstrain)	ASTM C 157	$800 > x \ge 600$	$600 > x \ge 400$	$400 > x$		
Creep[9] (x=microstrain/pressur e unit)	ASTM C 512	$75 \ge x > 60$/MPa ($0.52 \ge x > 0.41$psi)	$60 \ge x > 45$/MPa ($0.41 \ge x > 0.31$/psi)	$45 \ge x > 30$/MPa ($0.31 \ge x > 0.21$/psi)	30 MPa$\ge x$ (0.21 psi$\ge x$)	

[1]This table does not represent a comprehensive list of all characteristics that good concrete should exhibit. It does list characteristics that can quantifiably be divided into different performance groups. Other characteristics should be checked. For example, HPC aggregates should be tested for detrimental alkali silca reactivity according to ASTM C 227, cured at 38° C, and tested at 23° C and should yield less than 0.05 percent mean expansion at 3 months and less than 0.10% expansion at 6 months (based on SHRP C 342, p. 83). Due consideration should also be paid to (but not necessarily limited to) acidic environments and sulfate attack.
[2]All tests to be performed on concrete samples moist or submersion cured for 56 days. See Table 2 for additional information and exceptions.
[3]A given HPC mix design is specified by a grade for each desired performance characteristic. For example, a concrete may perform a Grade 4 in strength and elasticity, Grade 3 in shrinkage and scaling resistance, and Grade 2 in all other categories.
[4]Based on SHRP C/FR-91-103, p. 3.52.
[5]Based on SHRP S-360.
[6]Based on SHRP C/FR-91-103.
[7]Based on *PCA Engineering Properties of Commercially Available High-Strength Concretes.*
[8]Based on SHRP C/FR-91-103, p. 3.25.
[9]Based on SHRP C/FR-91-103, p.3.30.
[10]Based on SHRP C/FR-91-103, p. 3.17.

Figure 2.1. Grades of Performance Characteristics for HPC (from Goodspeed et al. 1996)

	Recommended HPC Grade for Given Exposure Condition				
Exposure Condition	N/A[2]	Grade 1	Grade 2	Grade 3	Grade 4
Freeze/Thaw Durability Exposure (x=F/T cycles per year)[1]	x<3	3≤x<50	50≤ x		
Scaling Resistance Applied Salt[3] (x=tons/lane-mile-year)	x<5.0	5.0≤x			
Abrasion Resistance (x=average daily traffic, studded tires allowed)	no studs/chains	x≤50,000	50,000<x<100,000	100,000≤x	
Chloride Penetration Applied Salt[3] (x=tons/lane-mile-year)	x<1	1.0≤x<3.0	3.0≤x<6.0	6.0≤x	

[1]F/T stands for "freeze/thaw." A freeze/thaw cycle is defined as an event where saturated concrete is subjected to an ambient temperature which drops below -2.2°C (28°F) followed by a rise in temperature above freezing.
[2]N/A stands for "not applicable" and indicates a situation in which specification of an HPC performance grade is unnecessary.
[3]As defined in SHRP S-360.

Figure 2.2 Recommendations for the Application of HPC Performance Grades (from Goodspeed et al. 1996) to provide the strength and durability needed for the structural and environmental requirements of the project.

USE OF POZZOLANS AND CHEMICAL ADMIXTURES IN HPC

Pozzolans and other admixtures are often not fully utilized in concrete mixtures. The addition of pozzolans and admixtures in concrete mix design enables the concrete producer to produce concrete with superior properties during the fresh and hardened states as discussed below. The pozzolans considered are fly ash and silica fume. The chemical admixtures examined for use in the production of HPC include: high-range water-reducers, retarders, water-reducers, and air entraining agents.

Use of Fly Ash in HPC

Because fly ash is typically less expensive than Portland cement, producers often view fly ask as a cement replacement. ACI 211 (1993) has established guidelines for replacement of cement by fly ash. The recommended replacement of Type F fly ash is 15-25% by weight, and 20-35% for Type C fly ash.

The Electric Power Research Institute (1984) states that the strength at any given age and rate of strength gain of concrete are affected by the characteristics of the particular fly ash, the cement with which it is used, and the proportions of each used in the concrete. Concrete containing fly ash typically continues to gain strength at ages beyond which concrete without fly ash has practically stopped gaining strength. Research by Berry and Malhotra (1980) shows that after the rate of strength gain of Portland cement slows, the continued pozzolanic activity of fly ash contributes to increased strength gain at later ages if the concrete is kept moist. Therefore, concrete containing fly ash with equivalent or lower strength at early ages may have equivalent or higher strength at later ages than concrete without fly ash.

Naik and Ramme (1990) recommended an approach for a precast plant to incorporate Type C fly ash into daily production. The approach is to start with 20 percent cement replacement by fly ash. Following several successful trial mixtures and gaining experience with the use of Type C fly ash, gradually the cement replacement is increased to 30 percent. Naik and Ramme's study on early age strength of precast, prestressed concrete products indicates that replacing cement with up to 30 percent Type C fly ash increases early age strength when compared to concrete made with no fly ash. These conclusions are not representative of typical findings on the replacement of cement with fly ash. The reason their results are not typical is that the tests were conducted on mix designs with water-to-cementitious-materials ratios *(w/cm)* that decreased as the percentage of fly ash in the mix increased. Also, each pound of cement removed from the mix was replaced with 1.25 pounds of fly ash. The *w/cm* of the mix with no fly ash was 0.45 as compared to 0.33 for the mix with 30 percent fly ash.

Neville's (1996) view on the use of fly ash in concrete mix design is that an excessive amount of fly ash is not beneficial from the point of view of strength development. He indicates that the limiting content should be around 30 percent by mass of total cementitious materials.

Fly ash has other benefits. Nawy (1996) states that there is improved workability when compared to mixes not containing fly ash because the volume of cementitious materials is normally larger. The increased volume produces a larger cementitious paste volume, hence better workability. Naik and Ramme's (1990) research showed that increasing fly ash replacement in concrete improves workability and that the water required for the same workability decreases. Neville (1996) indicated that the use of fly ash typically lowers water demand by 5 to 15%.

Research by Day (1995) showed a reduction in permeability which results in resistance to chloride attack of steel reinforcement for concrete with 25 % Type C fly ash compared to concrete with no fly ash. Manmohan and Mehta (1981) concluded that the long-term reaction of fly ash refines the pore structure of concrete and reduces the permeability.

Lane and Best (1982) showed that concrete with fly ash proportioned to have the same strength at the age of loading as concrete without fly ash, produced less creep strain at all subsequent ages. This is supported by the ACI Committee 232 (1996) report which states most investigations have shown if concretes with and without fly ash having equivalent 28-day strength are equally loaded at the same age, the fly ash concrete will exhibit lower long-term creep due to the greater rate of later-age strengths gain common to most fly ash concrete.

Properties that are not significantly affected by the use of fly ash include: resistance to freeze-thaw damage (Lane and Best 1982), modulus of elasticity (Cain 1979), and abrasion resistance (ACI 201 1992). The study by Cain concluded that cement and aggregate characteristics have a greater effect on modulus of elasticity than the use of fly ash. Modulus of elasticity is typically a function of the coarse aggregate characteristics.

Use of Silica Fume in HPC

Silica fume can significantly affect the strength and durability properties of a particular concrete mix. Currently there are no universally accepted standards for the recommended rate of cement replacement by silica fume in a concrete mix design. Past research has typically been conducted on silica fume mixes that comprise 5 to 30% of the total cementitious materials in a mix. Day (1995) indicates that the lowest permeability and highest degree of durability are provided by the use of silica fume at 10 to 15% of cement content, provided a superplasticizer is used to reduce the *w/cm*. An ACI (1987) report on high-strength concrete states that water demand is greatly increased with an increasing proportion of silica fume in the concrete mix. This increase in water demand typically requires the use of water-reducing and high-range water-reducing admixtures.

The most important characteristic associated with the use of silica fume in concrete mix design is that concrete with very low permeability can be produced. This is due to the very small silica fume particle sizes being dispersed to fill small voids typically occupied by the mix water. Research by Nawy (1996) suggests that cement replacement by 20 to 25% silica fume reduces the permeability to almost zero.

Silica fume's contribution to concrete strength development at normal curing temperatures takes place from about 3 to 28 days. ACI Committee 234 (1996) suggests that the contribution of silica fume to strength development after 28 days is minimal. Day (1995) states that prolonged moist curing is vital for concrete containing silica fume to achieve superior strength and permeability characteristics because of silica fume's contribution to strength gain between 3 and 28 days.

Concretes containing silica fume tend to be more cohesive and less prone to segregation than conventional concrete mixes. With increasing amounts of silica fume, fresh concrete tends to become sticky. Jahren (1983) suggests that to maintain the same apparent workability, industry experience has shown that it is necessary to increase the initial slump of the concrete with silica fume by 2 in. above that required for conventional portland-cement concrete. Grutzeck et al. (1982) found that concrete containing silica fume has significantly less bleeding. This is caused primarily by the high surface area of the silica fume that must be wetted which leaves very little free water in the mixture for bleeding. Grutzeck's research also suggested that silica fume reduces bleeding by physically blocking the pores in the fresh concrete. Because of the reduced bleeding in silica fume concrete, the potential for shrinkage cracks is increased. Research by Aitcin et al. (1981) in the laboratory and field indicated that concrete incorporating silica fume has an increased tendency to develop plastic shrinkage cracks.

Mixed results have been obtained regarding the effect of silica fume on freeze-thaw resistance of non-air-entrained concrete. Saucier (1984) found that non-air-entrained concrete with 15 percent silica fume that was cured for 28 days, had a durability factor of 95 percent when tested according to ASTM C 666 Procedure A. Hooton (1993) found similar results for high-strength concrete containing 10,15, and 20% silica fume at a *w/cm* of 0.35 which were moist cured for 14 days. In Hooton's research the control concrete's durability factor was 60 percent at 56 freeze-thaw cycles, while the durability factors of the silica fume concretes exceeded 90 percent at 300 freeze-thaw cycles. Malhotra et al. (1987) reported results that conflicted with those of Hooton. Malhotra et al. found that all non-air-entrained concretes failed at less than 50 freeze-thaw cycles regardless of *w/cm* or silica fume content. All of Malhotra's et al. test specimens were moist cured for 14 days prior to freezing. ACI Committee 234 (1996) recommends that the currently recommended values of air entrainment be used to provide adequate resistance to freeze-thaw damage even for concrete containing silica fume.

Role of Chemical Admixtures in HPC

The use of admixtures plays a significant role in the production of HPC. ACI Committee 212 (1991) states that admixtures are used to modify the properties of concrete or mortar to make them more suitable for the work at hand, or for economy, or for such other purposes as saving energy. In many instances, for example to achieve very high strength, resistance to freezing and thawing, or to retard or accelerate hydration, using an admixture may be the only feasible means of achieving the desired result. Kosmatka and Panarese (1988) state the major reasons for using admixtures are to: (1) reduce the cost of concrete construction, (2) achieve certain properties in concrete more effectively than by other means, (3) ensure the quality of concrete during the stages of mixing, transporting, placing, and curing in adverse weather conditions, and (4) overcome certain emergencies during concreting operations.

Use of High-Range Water-Reducing Admixtures in HPC

High-range water-reducers (superplasticizers) are primarily used to reduce the amount of water in the concrete mixture to increase the strength of the concrete and provide greatly increased workability. ACI Committee 212 (1993) states that superplasticizers can be used in concrete to: increase slump; increase strength by decreasing water content and *w/cm*; or decrease water and cement content, thus reducing temperature rise and volume change. Typical dosages of high-range water-reducers range from 10 to 25 oz per 100 pounds of cementitious materials.

Use of Water-Reducing Admixtures in HPC

Water-reducing admixtures are used in concrete mix design to reduce the water requirement of the mixture for a given slump, reduce the *w/cm*, or increase slump for the same *w/cm* and cement content. Kosmatka and Panarese (1988) indicate that water-reducers are used to reduce the water content by 5 to 10%, while high-range water-reducers are used to reduce water content by 12 to 30%. ACI Committee 212 (1991) states that water-reducing admixtures are used to produce concrete of higher strength, obtain a specified strength at lower cement content, or increase the slump of a given mixture without an increase in water content. Water-reducers may also improve the properties of concrete containing aggregates that are harsh or poorly graded, or both, or may be used in concrete that must be placed under difficult conditions. Water-reducers are also useful when placing concrete by means of a pump or tremie. Typical dosages of water-reducers range from 2 to 5 oz per 100 pounds of cementitious materials.

Use of Retarding Admixtures in HPC

Retarders act to delay the setting of fresh concrete. Neville (1996) states that retarders are used to: (1) offset the accelerating effect of hot weather on the setting of concrete, (2) delay the initial set of concrete or grout when difficult or unusual conditions of placement occur, such as placing concrete in large piers and foundations, cementing oil wells, or pumping grout or concrete over considerable distances, (3) delay the set for special finishing processes such as an exposed aggregate surface, or (4) for the prevention of cold joints caused by the early set of concrete placed in high temperatures.

Use of Air-Entraining Admixtures in HPC

Air-entraining admixtures are used to entrain microscopic air bubbles in concrete. The most important reason for having entrained air in concrete is to improve the concrete's ability to resist freeze-thaw damage. ACI Committee 212 (1993) states that air entrainment should always be required when concrete must withstand many cycles of freezing and thawing, particularly where the use of chemical deicing agents such as sodium or calcium chloride is anticipated. ACI Committee 201 (1992) recommends air content for concrete range from 3 to 7.5 percent depending on the exposure condition and nominal maximum aggregate size. Bridge decks in the Alabama are rarely exposed to deicing salts and fit into a moderate exposure condition.

Research conducted by Powers (1964) indicated that at equal slump, air-entrained concrete is considerably more workable and cohesive than similar non-air-entrained concrete except at higher cement contents. At high cement contents, air-entrained concrete becomes sticky and difficult to finish. Other improvements of concrete in the fresh state provided by entrained air include reduced segregation and bleeding (Neville 1996). A major draw back of air-entrainment in hardened concrete is the reduction of strength, especially in concretes with moderate to high cement contents. The reduction in compressive strength is typically around a five percent decrease for each percent of air entrained into the mix (Aitcin et al. 1981). Other difficulties encountered when using entrained air include: compatibility with other materials, mainly cement, high carbon fly ash, water-reducers, and superplasticizers (Neville 1996). The dosages needed to properly entrain air into concrete must be obtained through trial mixes using the proposed materials.

TESTING METHODS AND PROCEDURES

The compressive strength of a particular concrete has been found to vary depending on the size of the cylinder, the capping method used to test the cylinder, and the method in which the test cylinders are initially cured. Standard test cylinders are typically 4 or 6 in. in diameter and 8 or 12 in. in length. Currently, American Society for Testing and Material's "Standard Practice for Making and Curing Concrete Test Specimens in the Field" (ASTM C 31, 1996) specifies the standard specimen as a 6 x 12 in. cylinder when test specimens are made in the field. Practical considerations have necessitated the use of 4 x 8 in. test cylinders, particularly when testing high-strength concrete. The loads needed to fail 6 x 12 in. test specimens of high-strength concrete may exceed the capacity of available testing machines. Also, commercially available match-curing systems used for precast applications typically use 4 x 8 in. cylinder molds. Standards of The American Association of State Highway and Transportation Officials, "Standard Specification for Making and Curing Concrete

Test Specimens in the Field" (AASHTO T23, 1997) allow the use of 4 x 8 in. cylinders when the maximum aggregate size is smaller than 1 in.

Cook (1989) performed comparison tests to identify the effects of cylinder size. Cook states that for a mixture with a design strength of 10,000 psi, the strengths of 4 x 8 in. cylinders were approximately 5% higher than those of 6 x 12 in. cylinders. Research by Burg and Ost (1992) indicated that, for concrete strengths in the range of 10,000 to 20,000 psi, 4 x 8 in. cylinder strengths were generally within about one percent of the 6 x 12 in. cylinder strengths. The Canadian Standard Association (1995) which governs concrete materials and methods of concrete construction in Canada requires a five percent reduction in the measured strength when 4 x 8 in. cylinders are used. This is the result of a definitive study by Day (1994). Research by Carrasquillo et al. (1981) concluded that the ratio of strength of 6 x 12 in. cylinders to 4 x 8 in. cylinders was close to 0.90 regardless of the strength for strengths between 3000 and 11,000 psi. Recent unpublished research by Burg and Ost indicates that load transfer from spherically-seated platens into test specimens can affect the measured strength (ACI 363, 1998). Some platens that meet the dimensional requirements of ASTM C 39 (1996) "Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens" produce non-standard load transfer into high-strength 6 x 12 in. cylinders, but can transfer load properly for 4 x 8 in. cylinders. Burg and Ost state that this may explain the cases where 4 x 8 in. cylinders have higher measured compressive strength than 6 x 12 in. cylinders. They state that the higher strength measured for the 4 x 8 in. cylinders may actually be a more realistic measure of the compressive strength.

Capping Methods

There are three standard methods for preparing the ends of a test cylinder for compressive strength testing: sulfur mortar capping, end grinding, and unbonded neoprene cap systems. ASTM C 39 (1996) requires that the ends of test cylinders be ground or sulfur capped so that the loading surfaces are plane within 0.002 in. The AASHTO T 22 (1997) specification is identical to ASTM C 39 with the exception of T 22 Sections 5.2.1.3, 6.2, 7.4, and the Annex. The T 22 Annex permits the use of neoprene caps with steel extrusion controllers for compressive strength testing of 6 x 12 in. concrete cylinders as an suitable alternative to sulfur capping or grinding. ASTM C 1231 (1996) "Standard Practice for Use of Unbonded Caps in Determination of Compressive Strength of Hardened Concrete Cylinders" covers the use of unbonded caps in the determination of compressive strength of hardened concrete cylinders. Section 1.2 states that unbonded caps are not to be used for acceptance testing of concrete with compressive strengths above 7000 psi. AASHTO T 22 has no mention of an upper limit of compressive strength with the use of neoprene pads. Goodspeed et al. (1996) lists the capping methods discussed above for compressive strength testing in the article "High-Performance Concrete Defined for Highway Structures." Also, Goodspeed et al. suggests that when neoprene pads are used for early age testing of concrete with strength exceeding 10,000 psi at 56 days, neoprene pads should also be used for the 56-day tests as well.

Research by Pistilli and Willems (1993) indicates that unbonded cap systems composed of polymeric pads in restraining rings have been used successfully on high-strength concrete up to 19,000 psi. Grygiel and Amsler (1977) compared the strengths obtained with neoprene pads to those with sulfur caps over the range of 1500 to 7000 psi. They showed that neoprene pads produced slightly higher strengths, but the difference was of little practical significance. They also found that the neoprene pads produced lower within-test variability than the sulfur caps. Carrasquillo and Carrasquillo (1988) compared the performance of neoprene pads and sulfur caps for strengths between 6000 and 17,000 psi. They also showed that using pads results in lower within-test variability and higher compressive strengths for strengths above 10,000 psi. Neville (1996) states that grinding produces very satisfactory results but is slow and expensive. Neville states that even if there are small systematic differences in measured strength between these capping methods, that is not important because every method of capping introduces a systematic influence on the observed strength, so there is no "true" strength of concrete. It is important to use a single method on a given construction project.

Initial Curing Procedures

The use of accelerated curing methods in precast applications accentuates the need for producing concrete test specimens that accurately predict the strength of the actual member. Different initial curing methods for concrete test specimens affect the perceived initial and long-term strength of the concrete. Typical methods of initial curing used in precast applications include: standard ASTM C 31 (1996) curing, use of commercially available match-curing systems, and curing of standard plastic molded cylinders under the tarp along with the member. Match-curing systems incorporate insulated steel molds that include heated internal coils that cure the cylinders at the temperature history measured by a thermocouple located in the member.

Being able to accurately determine the early age strength of concrete is vital to a precast plant's productivity. Myers and Carrasquillo (1997) studied the effects of the three curing conditions mentioned above on the compressive strength of concrete for prestressed bridge girders. They found that the strength of match-cured cylinders exceeded the compressive strength of the member-cured cylinders by an average of 19.1% at release. In the same study, ASTM C 31 (1996) cured cylinders exceeded the compressive strength of the match-cured cylinders by an average of 9.8% at 56 days. The authors of that study concluded that, the use of match-cured cylinders would allow the precaster to release the prestressing force earlier and thereby increase plant productivity. Also, the use of match-cured cylinders for 56-day compressive strength tests would allow the designer to not overestimate the compressive strength of the concrete if it had been based on cylinders cured using ASTM C31 (1996).

CURING OF CAST-IN-PLACE HPC DECKS

Proper curing of concrete after placement and finishing has a strong influence on hardened properties such as strength, abrasion resistance, durability, and resistance to freezing and thawing and deicer salts. Kosmatka and Panarese (1988) state the objectives of curing are to prevent (or replenish) the loss of moisture from concrete, and to maintain a favorable concrete temperature for a definite period of time.

The extent to which the chemical reaction of portland cement, pozzolans, and water is completed, will influence the strength, durability, and density of the concrete. Kosmatka and Panarese (1988) state that most freshly mixed concrete contains considerably more water than is required for complete hydration to take place; however, any appreciable loss of water by evaporation or otherwise will delay or prevent complete hydration. It is very important for water to be retained during the first few days after concrete is placed because hydration is typically very rapid during these first few days. Hydration continues to take place as long as the internal relative humidity of the concrete is above approximately 80%.

Methods used to keep concrete decks moist during the early hardening period are: ponding or immersion, spraying or fogging, and covering by a wet mat. These methods can be used along with the following methods of sealing the surface to prevent the loss of moisture: covering the concrete with impervious paper or plastic sheets, or applying membrane-forming curing compounds (Kosmatka and Panarese 1988).

Since all the desirable properties of concrete are improved by curing, the curing time should be as long as practical. The recommended length of curing for cast-in-place bridge decks is 7 days, or the time necessary to attain 70 percent of the specified compressive strength (Kosmatka and Panarese 1988). Problems associated with inadequate curing include: reduced strength, reduced durability, and excessive drying shrinkage.

MONITORING OF TEMPERATURES IN MASS CONCRETE STRUCTURES

Mass concrete is defined by ACI 116 (1990) as "Any large volume of cast-in-place concrete with dimensions large enough to require that measures be taken to cope with the generation of heat and attendant volume change to minimize cracking." Mass concrete includes not only low-cement-content concrete used in dams and other massive structures but also moderate to high cement content concrete in structural members that require special considerations to handle heat of hydration and temperature rise. ACI 211 (1991) states that "Many large structural elements may be massive enough that heat generation should be considered, particularly when the minimum cross-sectional dimensions of a solid concrete member approach or exceed 2 to 3 ft or when cement contents above 600 lbs per cubic yard are being considered." HPC mixes are rich in cementitious materials, with cementitious materials contents typically over 700 lbs per cubic yard. Others factors contributing to temperature rise in mass concrete include: initial concrete temperature, ambient temperature, size of the element (volume to surface ratio), and the amount of reinforcement.

Kosmatka and Panarese (1988) state that a potential problem associated with high internal temperatures in mass concrete is that as the interior concrete rises in temperature, the surface concrete may be cooling and contracting. This causes tensile stress and cracks at the surface if the temperature differential is too great. The width and depth of the cracks depends upon the temperature gradient between the hot internal concrete and the cooler concrete surface. Kosmatka and Panarese indicate from previous studies and experience that the maximum temperature differential between the interior and exterior concrete should not exceed about 36° F to avoid surface cracks. Other problems include: drying shrinkage, reduced strength, and less durable concrete.

Kosmatka and Panarese (1988) state that methods used to control internal temperature gain of mass concrete include:

1. Using a low cement content of 200 to 450 lbs per cubic yard.
2. Using low heat of hydration Type II or IV portland or blended cement.
3. Using pozzolans since the heat of hydration of pozzolans is approximately 25% to 50% that of cement.
4. Reducing the initial concrete temperature to approximately 50° F by cooling the concrete ingredients.
5. Using steel forms for rapid heat dissipation.
6. Water curing.
7. Using low lifts of 5 ft or less during placement.

MONITORING OF TEMPERATURES IN PRECAST APPLICATIONS

Temperature rise of precast, prestressed concrete during initial curing prior to release of the prestressing force is typically affected by the steaming process used as well as the amount of cementitious materials in the mix. Steam-curing is advantageous where early strength gain in concrete is important or where additional heat is required to accomplish hydration, as in cold weather (Kosmatka and Panarese 1988). Temperature limits are typically placed on precast concrete exposed to steam-curing. Problems associated with steam-cured concrete exposed to temperatures in excess of 160° F include: microcracking and delayed ettringite formation (DEF) which lead to premature deterioration of precast structural elements. DEF represents internal sulfate attack, which is strongly influenced by the initial sulfate content of the cement, temperature at which the concrete is cured, exposure condition, and permeability of the concrete (Sarkar 1998). Collepardi (1999) states that microcracks developed during the steaming process is one of the essential factors for DEF related damage to occur.

With internal temperatures of steam-cured concrete reaching temperatures in excess of 150° F, proper curing of test cylinders is vital to accurately represent the concrete in the member. Currently, commercially available match-cure systems are the most accurate means of curing test cylinders at the same temperature as the precast concrete members. Match-curing systems are controlled by an embedded thermocouple that is placed in a specified location inside the member.

Research by Myers and Carrasquillo (1997) has shown that temperatures across the depth of a steam-cured member can vary up to 40° F. This research showed that the location of the thermocouple used to control the match-curing system has an effect on compressive strength and permeability properties of the match-cured cylinders. Thermocouples are usually placed at the centroid of the largest mass of concrete in a member because this area typically experiences the highest overall temperature. Depending on the steaming process, other locations such as the web or the top flange may experience higher temperatures (Cress 1996).

CORE TESTING PROCEDURES

When concrete test cylinders fail to reach the specified design strength, core specimens are typically taken from the member in question. Currently, AASHTO T 24 (1997), ASTM C 42 (1996) and ACI 318-95 (1995) have testing procedures for cases where adequate strength is not obtained from concrete test cylinders. AASHTO T 24 (1997), "Standard Specification for Obtaining and Testing Drilled Core and Sawed Beams of Concrete," includes a test procedure for testing core samples which have been submerged in lime-saturated water for 48 hours. This procedure is supported by ASTM C 42 (1996) "Standard Test Method for Obtaining and Testing Drilled Core and Sawed Beams of Concrete" as well. A report by The Concrete Society Working Party (1976) states that dry cores achieve higher failure loads than wet cores. ACI 318-95 (1995) states that if concrete in the structure will be dry under service conditions, core specimens shall be air dried for 7 days before the test, and shall be tested dry.

ACI 318-95 (1995) requires that three core samples be obtained for testing, but permits additional testing of cores extracted from locations represented by erratic core strength results. ACI 318-95 deems a structural member adequate if three cores are equal to at least 85 percent of the specified strength, and no single core has a strength lower than 75 percent of the specified value. Research by Malhotra (1977) suggests that even under excellent conditions of placing and curing, the strength of cores is unlikely to exceed 70 to 85 percent of the strength of standard test specimens. This view is supported by research conducted by Yuan et al. (1991) on high-strength concrete exceeding 10,000 psi. Yuan et al. concluded that at any given age, core strength is about 75 percent of moist cured 6 x 12 in. cylinder strength and 85 percent of air cured 6 x 12 in. cylinder strength. Lower core compressive strengths are attributed to the size of the core specimens, the drilling process used to obtain the core specimens, the core location, and the moisture condition of the cores during testing.

Conflicting with the information given above, results of core compressive strengths that were higher than companion 4 x 8 in. test cylinders were obtained during research conducted on a high-performance concrete bridge project in Sarpy County, Nebraska. The core compressive strength ranged from 5 to 15%

higher than standard 4 x 8 in. cylinders in plastic molds that were steam-cured along with the member. Cores were obtained from steam-cured precast, prestressed girders. The reason given for the higher compressive strengths of the cores was the superior quality of vibration and curing the girders received (Nebraska 1996).

CHAPTER THREE

CONSTRUCTION OF THE HPC BRIDGES

LOCATION OF THE HPC BRIDGES

The main bridge over Uphapee Creek is located on Alabama Highway 199 in Macon County, Alabama. The Uphapee Creek Bridge is a replacement for a bridge that was built in the 1940's that had suffered from streambed scour resulting from sand and gravel mining downstream. Alabama Highway 199 is frequently traveled upon by heavily loaded gravel and logging trucks. On the same project located within 1 mile of the Uphapee Creek Bridge is the Uphapee Creek Relief Bridge and Bulger Creek Bridge which were replaced but only utilized HPC for the cast-in-place concrete. A map showing the location of the bridges is shown in Figure 3.1.

DESCRIPTION OF HPC BRIDGE

The Uphapee Creek Bridge consists of seven 114 ft 54-in. AASHTO Bulb-Tee prestressed concrete girder spans supporting a 7 in. thick cast-in-place deck. The spans are simple-span construction supported on reinforced concrete bents on drilled-shafts. There are 5 girders per span spaced at 8.75 ft giving a 40 ft wide roadway. The overall length of the bridge is 798 ft. An elevation view

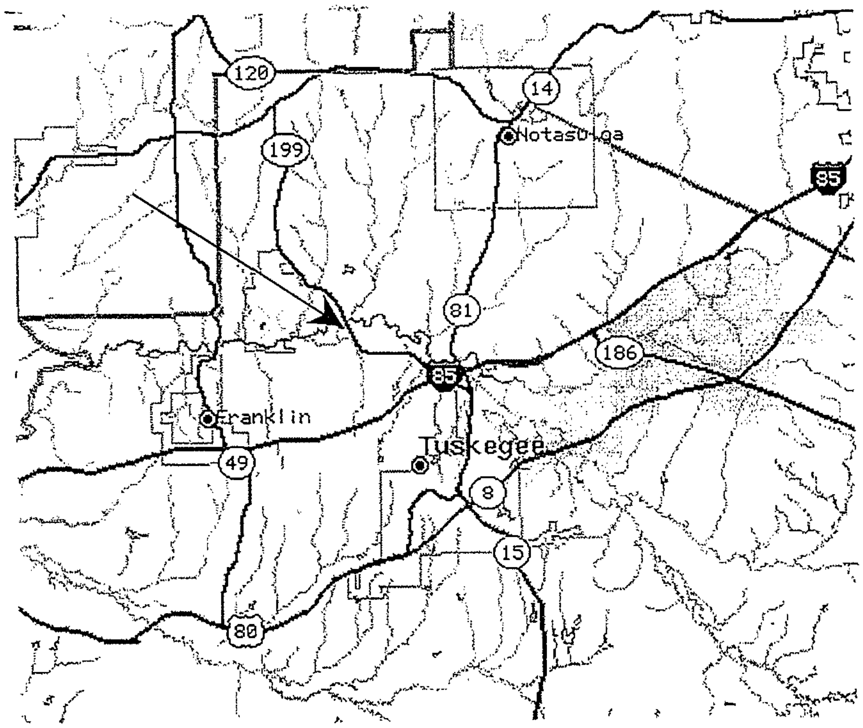


Figure 3.1. Map Showing Location of Bridges on this Project

of the bridge and typical cross-section taken from the construction drawings are shown in Figure 3.2 and 3.3.

The Uphapee Creek Relief Bridge consists of six 41 ft AASHTO Type I prestressed concrete girder spans supporting a 7 in. thick cast-in-place deck. The spans are simple-span construction supported on reinforced concrete bents on driven steel pile bents. There are 6 girders per span spaced at 7 feet giving a 40 ft wide roadway. The overall length of the bridge is 246 ft. An elevation view of the bridge and typical cross-section taken from the construction drawing are shown in Figure 3.4 and 3.5.

The Bulger Creek Bridge consists of three 55 ft AASHTO Type II prestressed concrete girder spans supporting a 7 in. thick cast-in-place deck. The spans are simple-span construction supported on reinforced concrete bents on drilled-shafts. There are 6 girders per span spaced at 7 feet giving a 40 ft wide roadway. The overall length of the bridge is 165 ft. An elevation view of the bridge and typical cross-section taken from the construction drawing are shown in Figure 3.6 and 3.7.

PRETENSIONED HPC GIRDERS

The AASHTO BT-54 girders are 54 inches deep and are pretensioned with 0.6 in. diameter, low-relaxation, 270 ksi, 7-wire strand. The strands are draped to reduce stresses on the ends of the girders. The BT-54's compressive strength was specified as 8000 psi at release and 10,000 psi at 28-days.

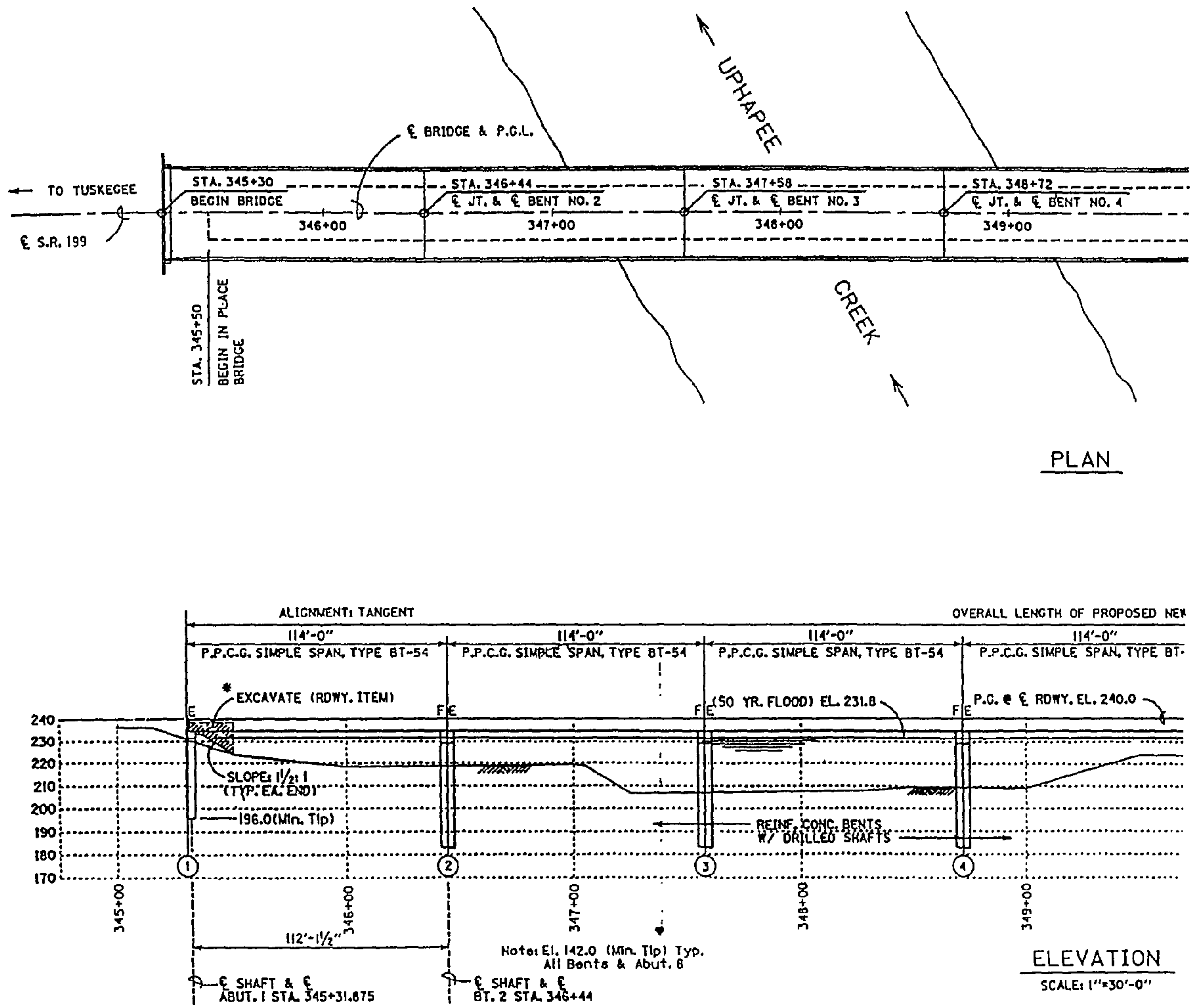
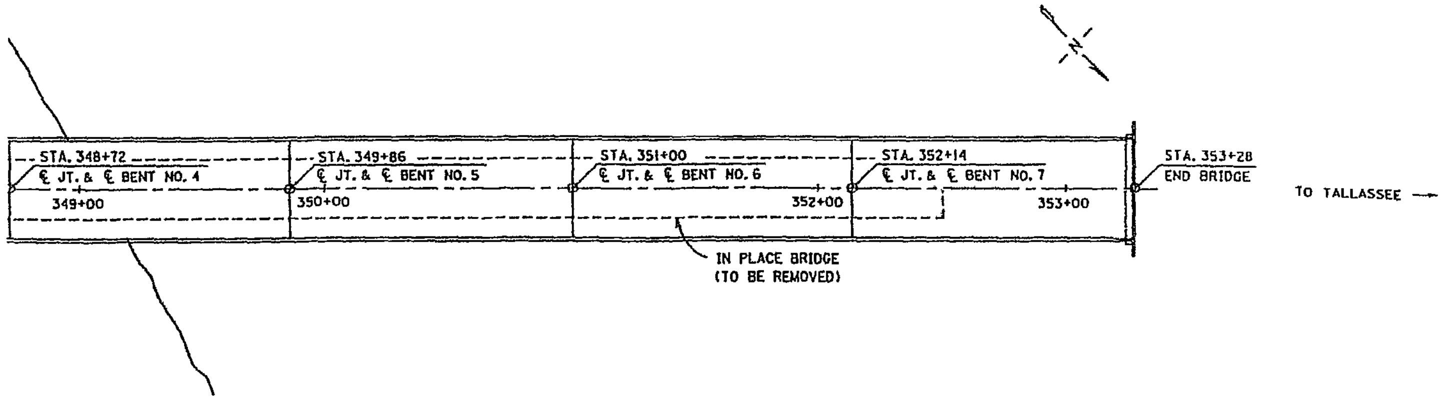
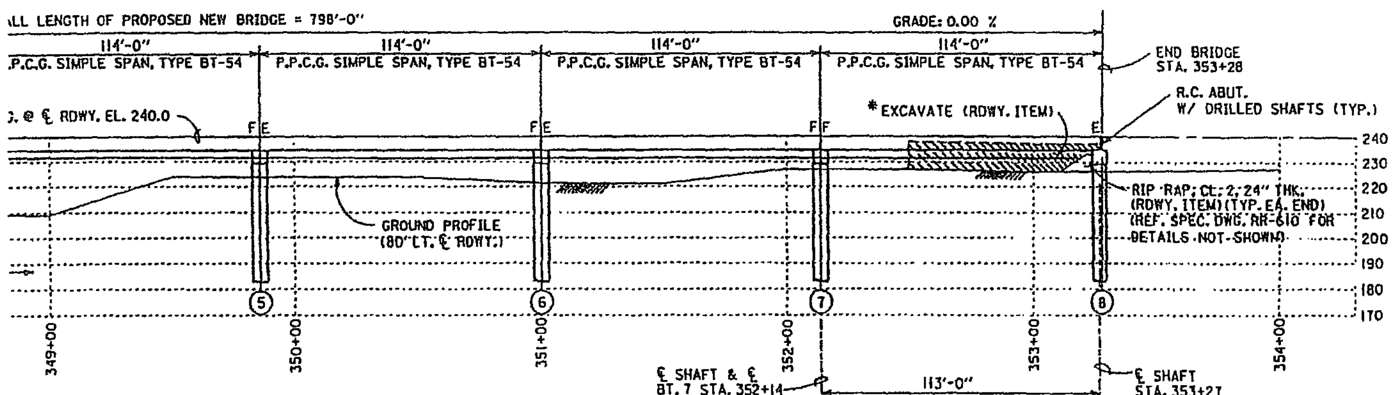


Figure 3.2. Plan and Elevation Views of the Uphapee Creek Bridge



PLAN



ELEVATION

SCALE: 1"=30'-0"

Figure 3.2. (Continued) Plan and Elevation Views of the Uphapee Creek Bridge

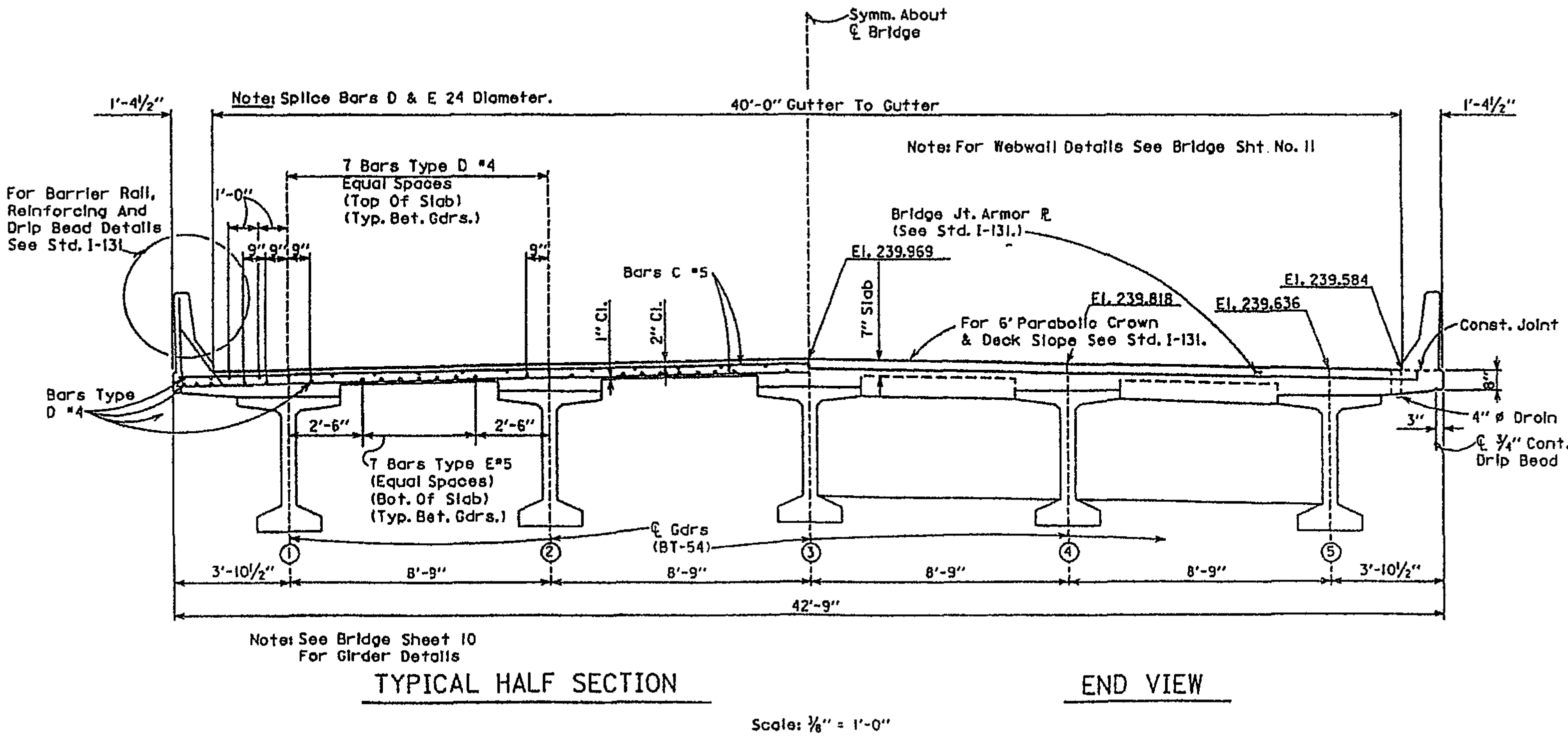


Figure 3.3. Cross Section of the Uphapee Creek Bridge

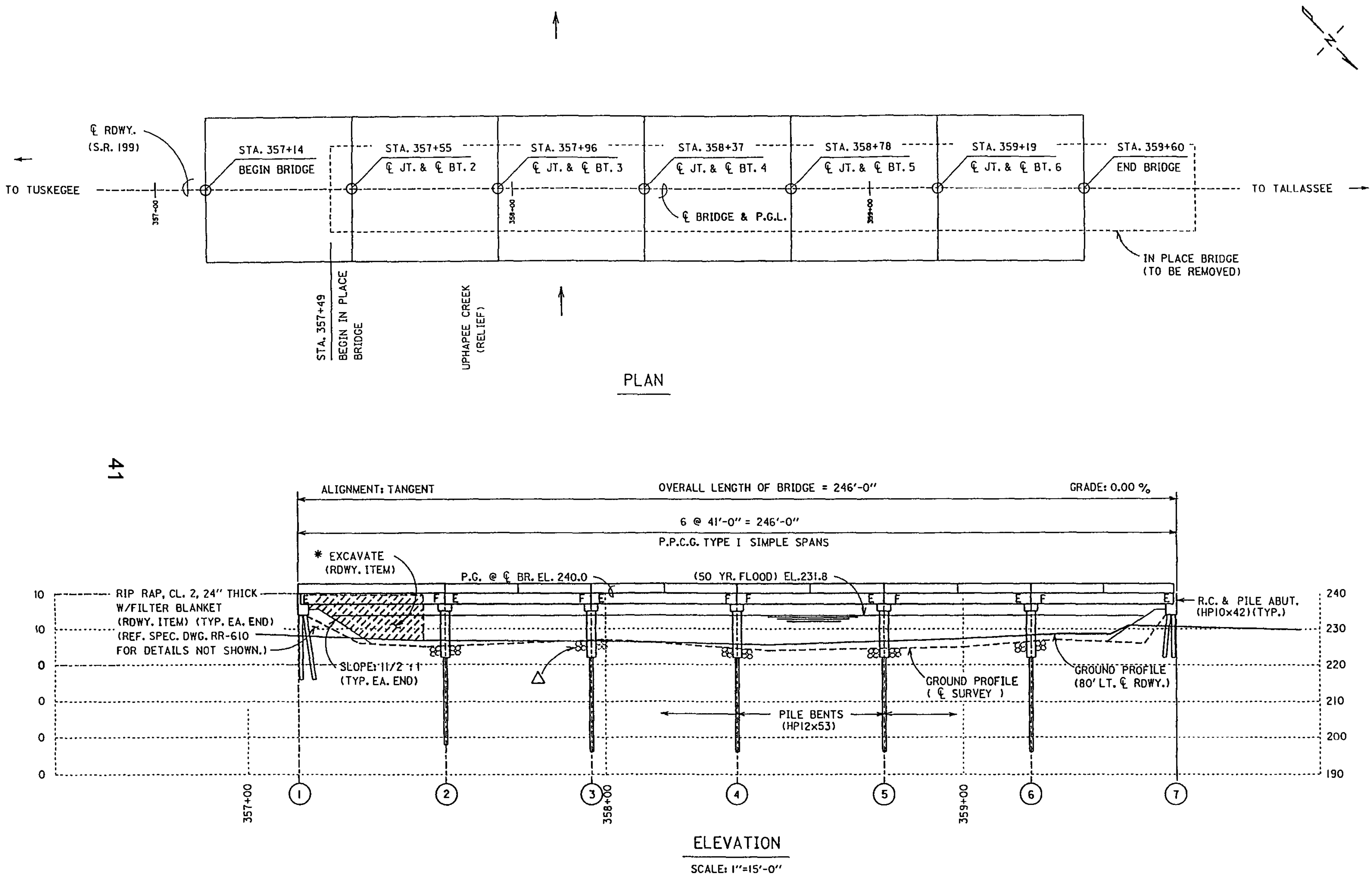


Figure 3.4. Plan and Elevation Views of the Uphapee Creek Bridge

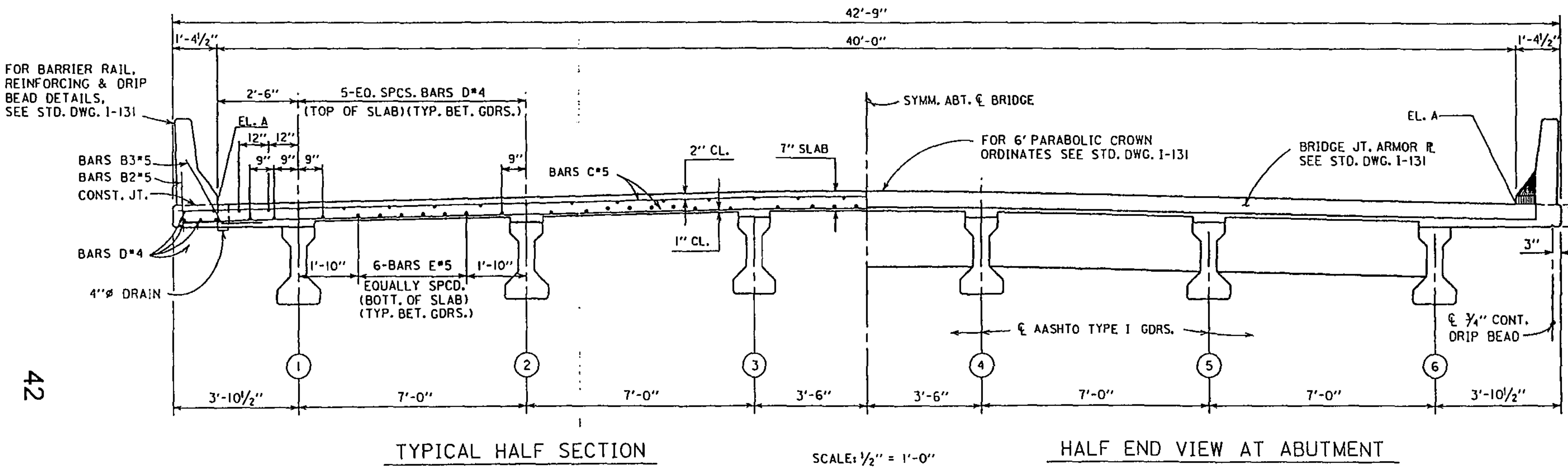


Figure 3.5. Cross Section of the Uphapee Creek Relief Bridge

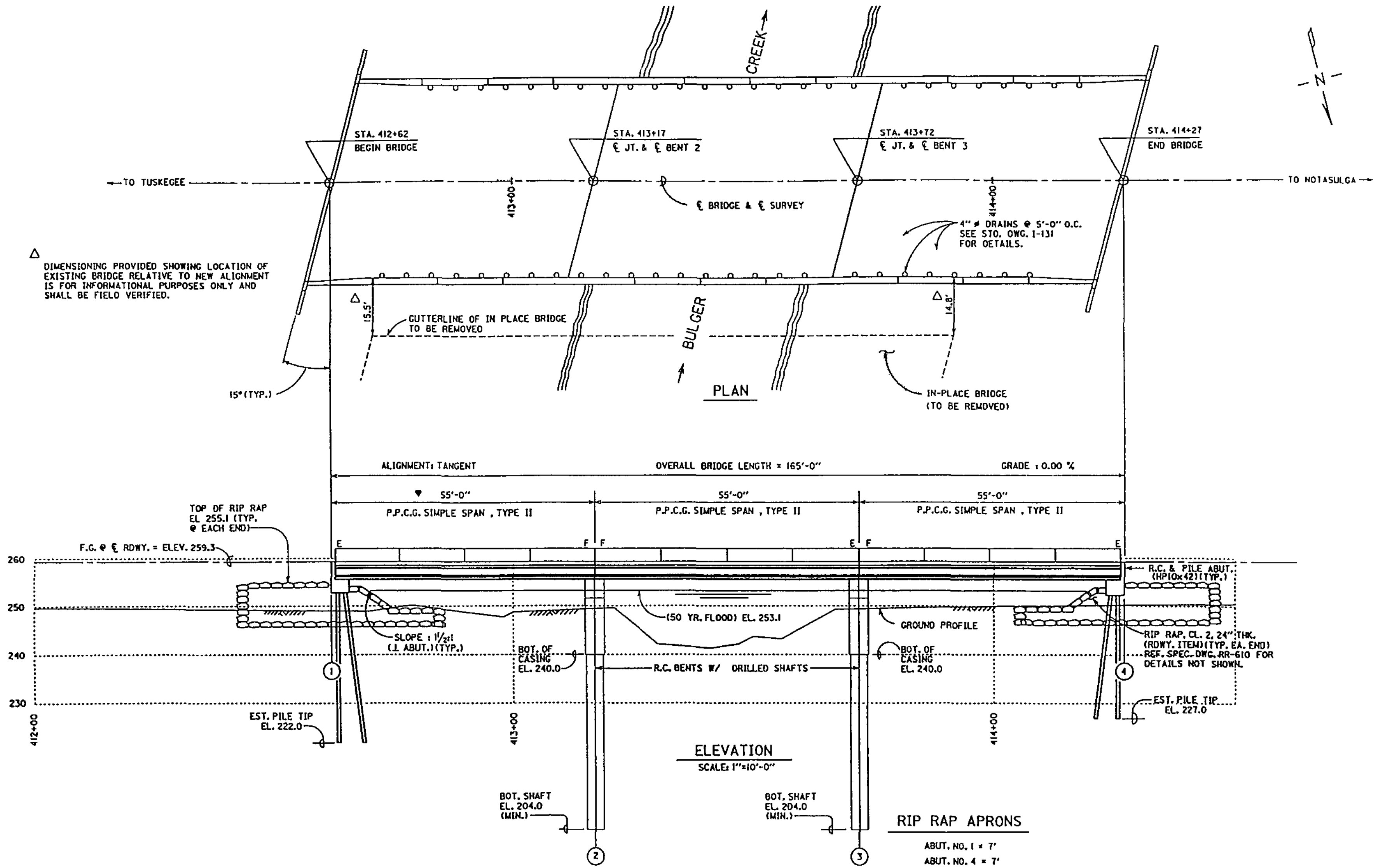


Figure 3.6. Plan and Elevation Views of the Bulger Creek Bridge

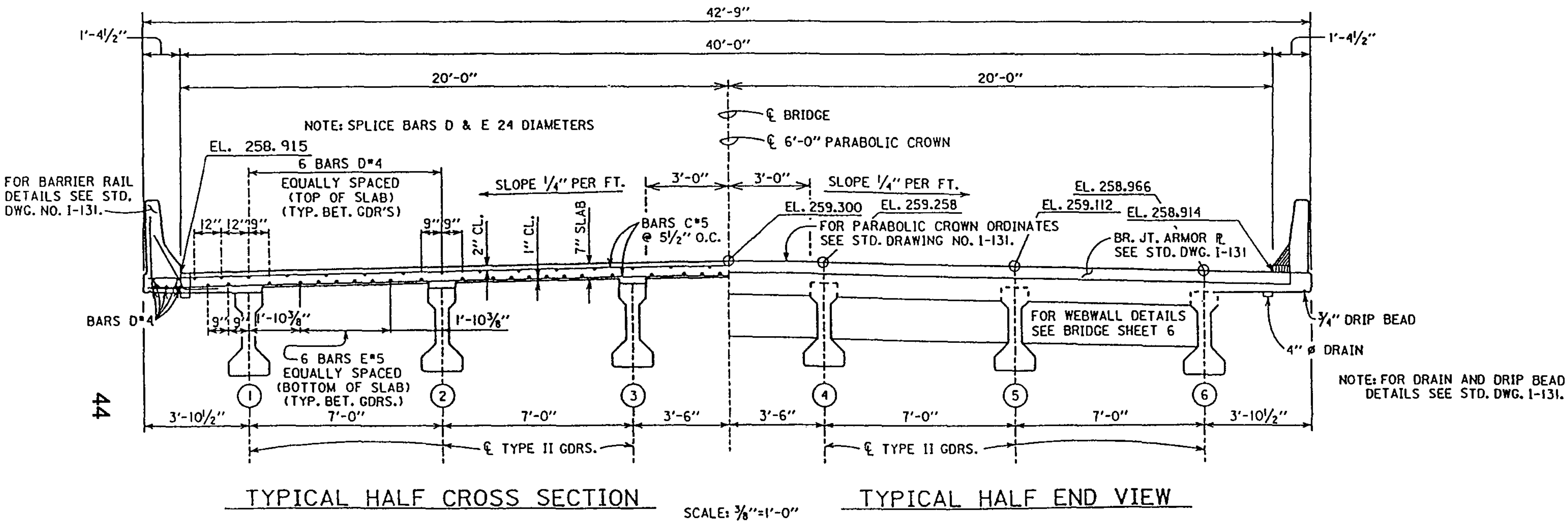


Figure 3.7. Cross Section of the Bulger Creek Bridge

New features for the HPC girders that were not typical to standard ALDOT projects are discussed in the following. The specified compressive strength of the concrete was higher than normal ALDOT projects. Typically bridge girders designed by ALDOT are based on 4000 psi release and 5000 psi 28-day compressive strengths with the prestressing force provided by 0.5 in. diameter, 7-wire strand. The HPC girders utilize the 0.6 in. diameter strand which permits a higher prestressing force to be applied to the member. This higher prestressing force allows the HPC's higher compressive strengths to be utilized in the design of the girders. Also, the use of pozzolans were required in the mix design to promote improved durability characteristics in the HPC girders. Another change in the mix design specification was allowing use of #7 crushed limestone. This was the first time that #7 crushed limestone was allowed in precast prestressed concrete girders for an ALDOT project. The use of a match curing system was required for the compressive strength test cylinders. In the past, cylinders were cured along side the member under the tarp covering the formwork, and no additional steps were taken to ensure that the cylinders cured at the same temperature as the member. Match curing was specified to produce strength test cylinders that most accurately represented the concrete in the member.

Materials savings resulted from the use of HPC instead of standard concrete in the girders. The bridge design was changed from an 800 ft bridge made up of eight 100 ft spans to a bridge made up of seven 114 ft spans, using one less line of girders. This results in the saving of two drilled shafts, two piers, a pier cap and the production of five less girders.

Instrumentation was placed in five of the HPC girders prior to casting. These five girders were located on the same span (span 5 from Figure 3.2) of the bridge. The instrumentation consisted of electrical resistance and vibrating wire strain gages and thermocouples. Strain gages were placed at various locations along the length of the girders to monitor strains before and after release of the prestressing force, during live load testing, and to monitor long-term strains in the girders during the bridge's service life. Thermocouples were placed along the full depth of the girder cross section to monitor the temperature gradient along the depth of the member and to make temperature corrections for strain measurements.

HPC DECKS, PIERS, AND CAPS

Cast-in-place substructure and superstructure HPC was used on all three bridges in this project. The specified compressive strength of the cast-in-place concrete was 6,000 psi. Design calculations for the cast-in-place concrete members were based on 4,000 psi. While the higher compressive strength of the cast-in-place concrete was not fully utilized in the design, HPC was specified to give enhanced performance and durability characteristics.

New features for the HPC deck that were not typical to standard ALDOT projects are discussed in the following. The specified compressive strength of the concrete was higher than a normal ALDOT project. The use of crushed limestone was permitted in the deck mix. In the past, only river gravel was specified by ALDOT for bridge decks. Again, the use of pozzolans in the mix design was required. The new requirements were specified to improve the strength and durability of the deck concrete. Wet curing of the deck concrete which is not usually specified by the ALDOT was required for this project. After finishing, the deck concrete was kept moist by mist spray, or fogging, as required until the deck could be covered with a wet curing blanket. Continuous wet curing was required for 7 days. This new feature was specified to help develop the strength and long-term durability characteristics of the HPC deck.

Instrumentation was placed in the deck of the main bridge on the same span as the instrumented girders. The instrumentation consisted of electrical resistance strain gages in both the longitudinal and transverse directions. This instrumentation was placed to measure strains in the deck during live load testing of the bridge. Two piers and a pier cap on the main bridge were instrumented with thermocouples to monitor the temperature history of the concrete during the first 96 hours after placement of the concrete. Further discussion of the strain gage instrumentation and results is provided in a companion report of this project. A description of the thermocouple measurements is provided in Appendix E.

DRILLED SHAFTS

Drilled shafts for the Uphapee and Bulger Creek Bridges did not utilize HPC, and the design was based upon 4000 psi. Two drilled shafts in the

Uphapee Creek Bridge were instrumented with thermocouples to monitor the temperature history at of the concrete during the first 96 hours after the placement of concrete. Discussion of these results is presented in Appendix E.

ORDER OF CONSTRUCTION AND MILESTONE DATES

Clark Construction Company, Inc. of Headland, Alabama was the general contractor for this project. Table 3.1 shows the order in which the Uphapee Creek Bridge was constructed as well as milestone dates of the HPC project as a whole. Tables 3.2 and 3.3 show the order in which the Uphapee Creek Relief Bridge and Bulger Creek Bridge were constructed respectively.

PROJECT COST

Prior to the bidding there was no way to know what if any increase in construction cost would result from the use of HPC. A savings of materials resulted from using HPC in the design of the Uphapee Creek Bridge. One less line of girders and one less pier were used as a result of the high compressive strengths specified for the girders. However, an increase in the unit cost of the cast-in-place and prestressed concrete was expected. ALDOT estimated the cost of the HPC project (all three bridges) by increasing the unit cost of the HPC concrete items by 30 to 40% above the normal cost of standard concrete. A comparison of the estimated costs and the two lowest bids is shown in Table 3.4. The results indicate that the impact on the project cost due to using HPC was much less than expected. Much of the materials savings was realized as cost savings.

Table 3.1. Order of Construction for the Uphapee Creek Bridge and Milestone Dates for the HPC Project

Activity	Start Date	Finish Date
Project bid date	3/27/98	-
Notice to proceed issued	6/23/98	-
Clark Construction, Inc. began mobilization on project site	7/8/98	-
Demolition of existing bridges	7/9/98	10/15/99
HPC girder test pour at Sherman Prestressed Concrete	9/1/98	-
Results and approval of HPC girder concrete	9/29/98	-
First pour of HPC cast-in-place concrete on relief bridge	10/5/98	-
Production of HPC girders	12/9/98	3/9/99
Instrumentation of HPC girders	12/16/98	1/26/99
Drilled shaft construction on main bridge	11/17/98	2/19/99
Instrumentation of drilled shafts for temperature history	1/8/99	1/15/99
Instrumentation of substructure for temperature history	10/29/99	11/13/99
Substructure poured on main bridge	10/05/99	10/29/99
Pour abutment on north end of main bridge	10/22/99	10/25/99
Pour abutment on south end of main bridge	10/26/99	10/28/99
HPC deck test pour	4/22/99	-
Instrumentation of HPC deck on main bridge	11/2/99	11/9/99
Casting of HPC deck on main bridge	11/15/99	12/23/99
Casting of diaphragms on main bridge	11/13/99	11/20/99
Casting of barrier rails on main bridge	1/20/00	1/21/00
Bridge open to traffic	-	4/2000
HPC bridge showcase in Auburn, AL	6/29/99	7/1/99

Table 3.2. Order of Construction for the Uphapee Creek Relief Bridge

Activity	Start Date	Finish Date
Demolition of existing bridge	9/4/98	10/15/98
Production of girders for relief bridge	8/17/98	9/9/98
Pour substructure on relief bridge	10/5/98	11/18/98
Pour abutment on south end of relief bridge	10/5/98	10/14/98
Pour abutment on north end of relief bridge	10/28/98	11/3/98
Girders set on relief bridge	12/3/98	12/12/98
Casting of HPC deck on relief bridge	4/27/99	5/7/99
Casting of diaphragms on relief bridge	2/26/99	3/5/99
Casting of barrier rails on relief bridge	1/18/00	1/18/00
Bridge open to traffic	4/2000	4/2000

Table 3.3. Order of Construction for the Bulger Creek Bridge

Activity	Start Date	Finish Date
Demolition of existing bridge	n.a.	n.a.
Production of girders for Bulger Creek Bridge	10/12/98	10/24/98
Pour substructure on Bulger Creek Bridge	10/15/98	11/30/98
Pour abutment on south end of Bulger Creek Bridge	11/30/98	12/2/98
Pour abutment on north end of Bulger Creek Bridge	11/16/98	11/19/98
Girders set on Bulger Creek Bridge	1/15/99	1/20/99
Casting of HPC deck on Bulger Creek Bridge	5/12/99	5/18/99
Casting of diaphragms on Bulger Creek Bridge	3/17/99	3/26/99
Casting of barrier rails on Bulger Creek Bridge	1/12/00	1/12/00
Bridge open to traffic	4/2000	4/2000

Table 3.4. Bid Results ($1,000's) for Alabama's HPC Bridge Project

Source	HPC Girders	6000 psi C.I.P.	Project Total
Estimate	555	1,061	4,160
Bid 1	476	695	3,464
Bid 2	511	865	3,609

CHAPTER FOUR

PRODUCTION OF HPC GIRDERS

INTRODUCTION

The HPC girders for the Uphapee Creek Bridge were produced by Sherman Prestressed in Pelham, Alabama. Contractors in the state of Alabama had limited experience working with HPC. To help bridge the information gap between HPC and standard concrete mix designs, preliminary mix studies were performed by Auburn University, and the results were presented to Sherman Prestressed Concrete (referred to here as Sherman) during the initial stages of the project. Sherman chose proportions for a preliminary mix and performed some testing on full size batches. After these preliminary tests, a test pour, as required in the project specifications, was conducted by Sherman using the proposed HPC mix. Tests were conducted by Auburn University, the FHWA Mobile Concrete Testing Laboratory, and Sherman to determine properties of the concrete. Through cooperation between Sherman Prestressed Concrete, the ALDOT, Auburn University, and the FHWA Mobile Concrete Testing Lab, a mix design was developed and accepted for use in the production of the HPC girders.

REQUIREMENTS FOR GIRDER CONCRETE

The project requirements for the HPC girder concrete featured specifications not normally used by the ALDOT, as well as those normal to any project. The project requirements for the HPC girder concrete were documented in a Special Provision to the ALDOT Standard Specifications. General requirements for the concrete, sampling and testing, and materials were covered by the ALDOT Standard Specifications without changes. A summary of the special requirements for the HPC girder concrete is included in Table 4.1. The most obvious changes from a normal ALDOT project were the higher than normal compressive strengths, the required use of pozzolans, and the required use of match curing for the concrete test cylinders.

HPC is intended to provide improved durability and strength characteristics of the completed structure. Various approaches have been used in HPC Showcase projects to achieve improved performance of the concrete. Some states have mandated mix proportions for use by the contractor, and other states have used specific performance requirements such as specifying the maximum permissible permeability of the concrete. In the development of the project specifications for the Alabama Showcase Project, it was desirable to provide a minimum of new, or unusual, specifications. The approach of specifying the mix proportions was undesirable because this is done on normal ALDOT projects. This project was used as an opportunity to increase the flexibility allowed to the contractor. A pure performance specification such as one specifying values of permeability, abrasion resistance, shrinkage and creep was avoided because it was expected to be too large of a change for the local construction industry, thus causing a large increase of costs. The goal of the final project specifications was to ensure the use of HPC in the girders with

Table 4.1. Special Requirements for Prestressed HPC Girder Concrete

Release Strength: 28 Day Strength:	f′ci: 8,000 psi f′c: 10,000 psi
Maximum Water-to-Cremetitious-Material Ratio *(w/cm)*	0.32
Air Content:	3.5% to 6%
Maximum Internal Temperature:	160˚F
Maximum Slump:	8 in.
Coarse Aggregate:	Crushed Limestone ALDOT # 7 or # 67
Fine Aggregate:	Natural Sand
Pozzolan Requirement: (percentage of total crementitous material)	
Class F fly ash	15 to 25%
Class C fly ash	15 to 35%
Microsilica	7 to 15%
When combinations of fly ash and microsilica are used, the fly ash content shall satisfy the above-specified ranges, and the microsilica will be considered an addition. The microsilica addition shall be at least 3% of the total cement plus fly ash content and no more that 15%.	
Match curing of all test cylinders within 5˚F of products is required. 4 in. x 8 in. test cylinders allowed.	

Table 4.2. Mix Proportions Used for HPC Girders

Material	Source	Cubic Yard Proportions
Type III Cement	Citadel	752 lbs
Type C Fly Ash	Holnam	152 lbs
#67 Limestone	Sherman	1822 lbs
#89 Sand	Sherman	374 lbs
#100 Sand	Sherman	695 lbs
Water	---	265 lbs
Superplasticizer	MB Rheobuild® 1000	14 oz/cwt
Retarder	MB Delvo®	3 oz/cwt
Air Entrainment	MB Micro Air®	4 oz

Water/Cementitous Materials Ratio = 0.30

a minimum amount of changes in normal construction specifications and practices. This approach was chosen so that a realistic evaluation of HPC could be made. Testing of the girder concrete was then used to determine the performance characteristics resulting from use of the specification.

Match Curing System

The project documents called for acceptance of the members based on test cylinders cured using match curing. The specification stated that the match curing system shall continuously monitor the internal temperature of a member in the line of production and maintain the internal temperature of the concrete test cylinders within 5°F of the internal temperature of the member up to the time of release of the prestressing force. The internal temperature of all members shall be continuously monitored and recorded at a total of three points specified by the Engineer based on the configuration of the casting beds and steam curing equipment. Discussion of the match curing systems used is presented later in this chapter.

MIX PROPORTIONS

During the initial stages of the project, a series of trial mixes were made by Auburn University to identify the characteristics of HPC mixes with various combinations of pozzolans and locally available aggregates. These trial mix designs were based upon past HPC projects, information obtained from review of literature, and the ACI (1993) "Guide to Selecting Properties for High-Strength Concrete with Portland Cement and Fly Ash." Various combinations of materials, *w/cm*, and curing procedures were studied. The results of the trial mixes were used with other information to develop the requirements shown in Table 4.1. The results were also presented to Sherman Prestressed Concrete and served as a starting point for their own preliminary mix testing, which resulted in development of a HPC mix design. The mix proportions are shown in Table 4.2. Actual testing of the mix prior to production of the HPC girders was conducted by Sherman Prestressed Concrete on a series of AASHTO Type I girders and various other products. Problems associated with high internal temperatures, low air content, and workability were addressed during these pours. Also, as part of the HPC project specifications, a test pour was required.

TEST POUR

On September 1, 1998 a test pour was conducted at Sherman Prestressed Concrete on a ten foot long section of standard AASHTO BT-63 girder to provide evidence of the Contractor's ability to mix, handle, place, consolidate and cure the HPC mix proposed for use in the production of the HPC girders. Other objectives of the test pour included: meeting strength requirements, acceptable use and performance of the match-curing system, and demonstration of the ability to meet the air, slump, and internal temperature requirements.

Core Testing

Other requirements for the test pour included obtaining core samples. Based on procedures recommended by ACI 318-95 (1995) and AASHTO T 24 (1997) as well as experimental procedures recommended by Auburn University, a core testing plan was developed and implemented on the test beam. The core testing plan is described in Table 4.3. Results of compressive strength tests of the cores are shown in Table 4.4. The results given in Table 4.4 are uncorrected compressive strength values. AASHTO T 24 (1997) specifies a reduction factor to be applied to compressive strengths under 6000 psi when the length-to-diameter ratio for the core specimens are less then two. All cores were approximately 3.8 x 6.2 inches which results in a length-to-diameter correction factor of 0.972. Further discussion on the development of the core testing plan and interpretation of the results is presented in Chapter 8.

Properties of HPC Measured at Test Pour

Properties of the test pour concrete were obtained from cylinders cured using the following initial curing methods: contractor's match curing system, Sure Cure™ match curing system, cylinders cured along with the member under the tarp, and standard ASTM curing. Compressive strength cylinders were tested to failure at release, 7, 14, 28, and 56 days to determine the strength gain characteristics of the HPC. The cylinders were tested using a pad-cap system with 70 durometer neoprene pads. Splitting tensile strength and modulus of elasticity were also determined to further identify the strength characteristics of

Table 4.3. Core Testing Plan

Testing Procedure	Cut[1]	Test
(1) Cut and Test Same Day	28 Days 30 Days 35 Days 56 Days	28 Days 30 Days 35 Days 56 Days
(2) Cut, Soak 48 Hours in Lime Bath, Test	28 Days	30 Days
(3) Cut, Air Dry 7 Days, Test	28 Days	35 Days
(4) Sure Cure™ Match-Cured Cylinders		28 Days 30 Days 35 Days 56 Days

1. Four core samples approximately 3.8 x 6.2 in. were obtained according to AASHTO T-24.

Table 4.4 Core Compressive Strength Test Results (psi)[1]

Test Procedure[2]	28-Day	30-Day	35-Day	56-Day
(1)	11770	10660	12030	12680
(2)		11470		
(3)			12990	
(4)	11550	11760	11860	11820

1. Core compressive strength is the average of the best 3 out of 4 samples tested on 70 durometer neoprene pads in FHWA Mobile Testing Lab. See Appendix A.1 for individual core compressive strength test results.
2. Testing procedure described in Table 4.3.

Table 4.5. Test Pour Results[1] for Different Initial Curing Conditions and Cylinder Sizes

Cylinder Initial Cure	Final Cure	Compressive Strength (psi)					Modulus of Elasticity[3] (psi)		Tensile (psi)
		Release[2]	7-Day	14-Day	28-Day	56-Day	Release	28-Day	Release
4 x 8 in. CMC	Air	10170	-	11320	11420	13150	-	-	-
4 x 8 in. Sure Cure	Air	9970	10800	10930	11550	11820	6,500,000	6,650,000	860
4 x 8 in. Under Tarp	Air	10060	-	12210	12150	-	-	-	-
4 x8 in. ASTM	Lime	8780	10550	11380	12440	13580	6,500,000	8,100,000	630
6 x 12 in. CMC	Air	9500	-	10370	11120	-	-	-	-
6 x 12 in. Sure Cure	Air	8520	-	-	10390	-	6,700,000	7,050,000	670
6 x 12 in. Under Tarp	Air	9010	-	11090	10750	-	-	-	-

CMC – Contractor's match-cure system. Cylinders made and tested by Sherman Prestressed Concrete.
Sure Cure – Sure Cure™ match-cure system. Cylinders tested in FHWA Mobile Concrete Laboratory.
Under Tarp – Cylinders cured under tarp with member. Cylinders made and tested by Sherman Prestressed concrete.
ASTM – Cylinders cured according to ASTM C31. Cylinders made and tested by FHWA Mobile Concrete Laboratory.

1. Refer to Appendix A.2 for individual cylinder test results.
2. Release time was 24 hours.
3. Result of one cylinder.

the HPC. The results are summarized in Table 4.5. A discussion of these results and conclusions are presented in the Chapter 8.

Additional Testing Conducted at Test Pour

Additional tests were conducted as part of the test pour by the FHWA Mobile Concrete Laboratory, Auburn University, and Sherman Prestressed Concrete. Tests were performed to study the effects of cylinder size, final curing conditions, and cylinder capping methods on compressive strength. Effects of cylinder size on strength properties are shown in Table 4.5. Compressive strength results illustrating the effects of final curing conditions and cylinder capping method are shown in Table 4.6 and 4.7. A discussion of these results and conclusions are presented in Chapter 8.

Other information obtained from the test pour include the time-temperature histories for the match curing systems proposed for use in production of the HPC girders. The contractor's match cured cylinders as well as Sure Cure™ match cured cylinders followed the internal temperature of the product measured by a thermocouple embedded at mid-length of the test pour beam, 5 in. from the bottom of the cross section. This thermocouple location corresponds with the thickest part of the cross section of the member, which typically produces the highest internal temperature. Descriptions of the two match curing systems are provided in the following section.

Table 4.6. Effects of Final Curing Conditions on Compressive Strength for Test Pour

Cylinder Initial Cure	Final Cure[2]	Compressive Strength (psi)[1]		
		7-Day	28-Day	56-Day
4 x 8 in. Under Tarp	Air	11670	11110	11760
4 x 8 in. Under Tarp	Moist	10680	11510	12910
4 x 8 in. Under Tarp	Lime	11220	11490	12630

Under Tarp – Cylinders cured under tarp with member.

1. Individual cylinder test results are given in Appendix A.3. Results shown are the average of two cylinders tested on 70 durometer neoprene pads. All cylinders tested at Auburn.
2. Air – cured on Sherman Prestressed Concrete casting yard.
   Moist – cured in Auburn's moist room with 90-95% humidity at 70-74°F.
   Lime – cured in Auburn's lime bath at 70-74°F.

Table 4.7. Effects of Cylinder Capping Method on Compressive Strength for Test Pour

Cylinder Initial Cure	Final Cure	Capping Method[2]	Compressive Strength (psi)[1]			
			Release[3]	7-Day	14-Day	28-Day
4 x 8 in. CMC	Air	Neoprene	10170	-	11320	11420
4 x 8 in. CMC	Air	Sulfur	8990	10340	10340	10940
4 x 8 in. Under Tarp	Air	Neoprene	10060	-	12210	12150
4 x 8 in. Under Tarp	Air	Sulfur	9210	-	10430	11640
6 x 12 in. CMC	Air	Neoprene	9500	-	10370	11120
6 x 12 in. CMC	Air	Sulfur	9020	9950	10300	10580
6 x 12 in. Under Tarp	Air	Neoprene	9010	-	11090	10750
6 x 12 in. Under Tarp	Air	Sulfur	8390	10620	10240	10550

CMC – Contractor's match cure system. Cylinders made and tested by Sherman Prestressed Concrete.
Under Tarp – Cylinders cured under tarp with member. Cylinders made and tested by Sherman Prestressed Concrete.

1. Individual cylinder test results are given in Appendix A.4. Results shown are the average of two cylinders.
2. Neoprene – A pad cap system using 70 durometer neoprene pads. Sulfur - high strength sulfur capping compound.
3. Release time was 24 hours.

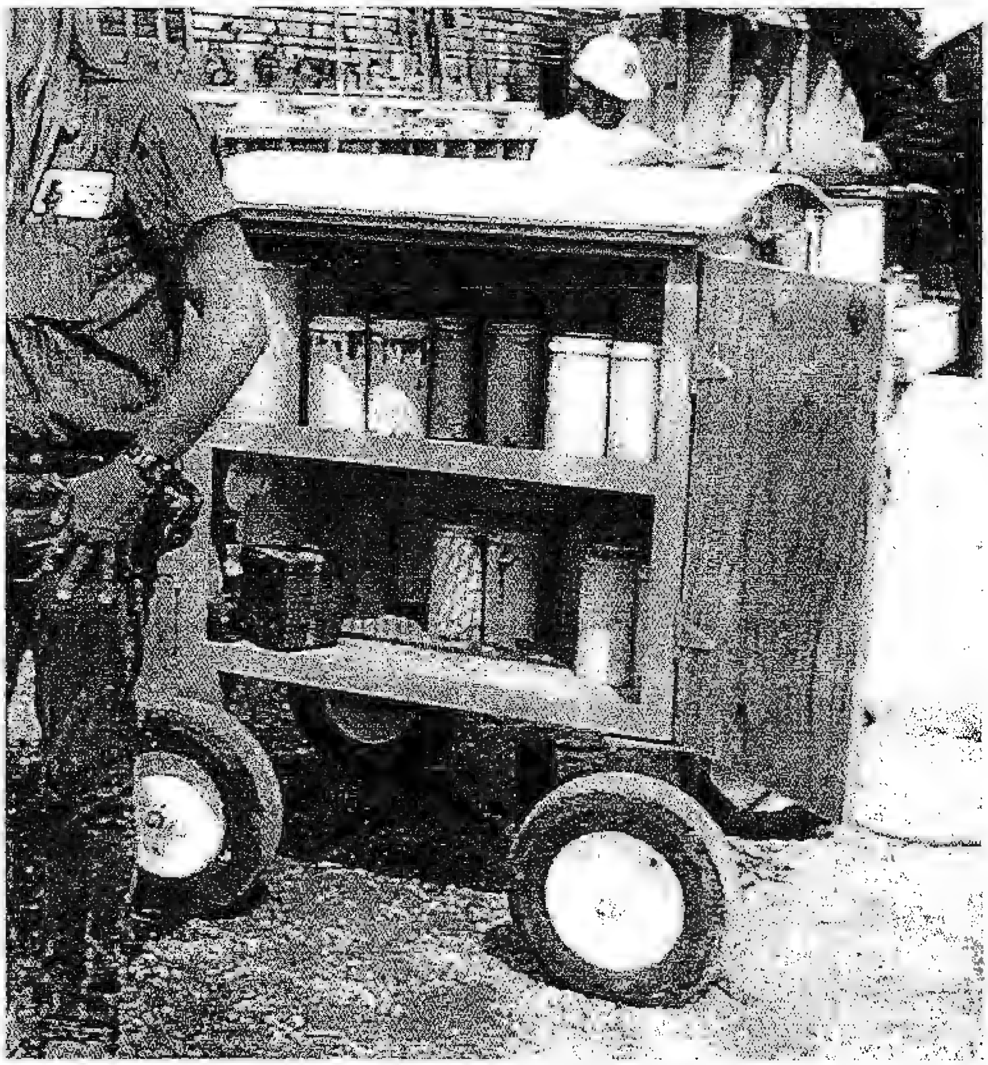


Figure 4.1. Contractor's First Match Curing System

Contractor's Match Curing System Used at Test Pour

The contractor's match curing system consisted of a 3 ft x 4 ft insulated wooden box, as shown in Figure 4.1, in which cylinders in standard plastic molds were placed. A controller, which monitored the internal temperature of the girder by means of an embedded thermocouple was used to regulate a small electric heater with a fan that was placed inside the wooden box. The box was vented by circular holes cut in the sides of the curing box. The temperature of the test cylinders was monitored by a thermocouple placed at the center of a concrete cylinder inside the box. Both the cylinder's and member's temperature history was continuously recorded by a rotating dial temperature recorder. Results and discussion of the contractor's match curing system are presented in the Test Pour Results section.

Sure Cure™ Match Curing System used at Test Pour

The Sure Cure™ match curing system consisted of electrically heated 4 x 8 in. insulated steel cylinder molds. The mold's heaters were driven by a controller which monitored the internal temperature of the member by means of an embedded thermocouple. Temperature history of the Sure Cure™ match cured cylinders and member were recorded by a Humbolt H-2680 Multi-Channel Maturity Meter every half an hour. Both systems are shown in Figures 4.2 and 4.3. Discussion on the performance of the Sure Cure™ match curing system is given in the following section.

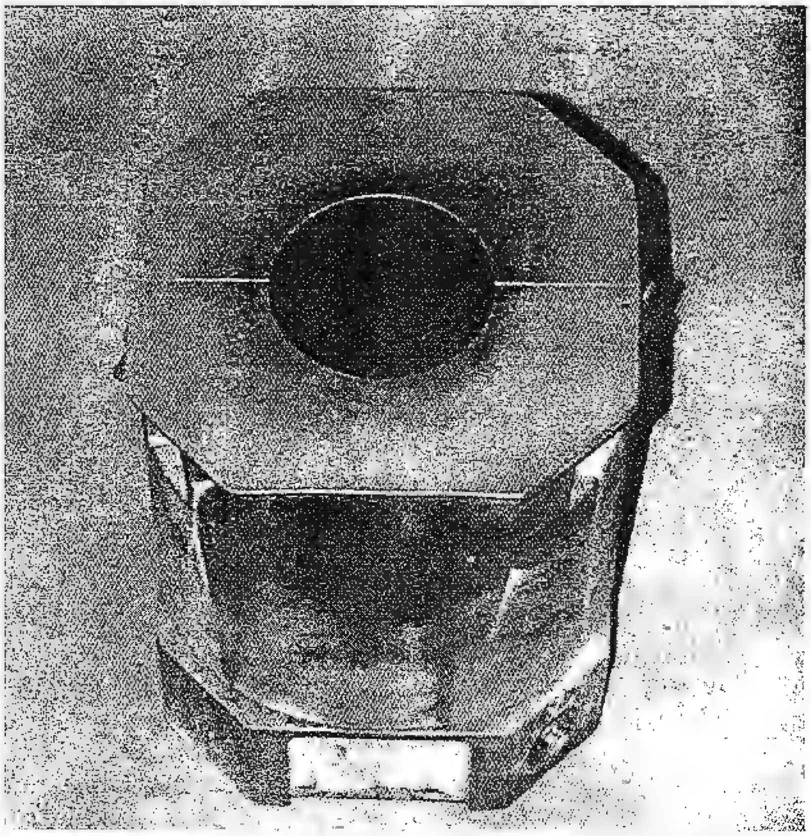


Figure 4.2. Sure Cure™ Match-Cure Cylinder Mold

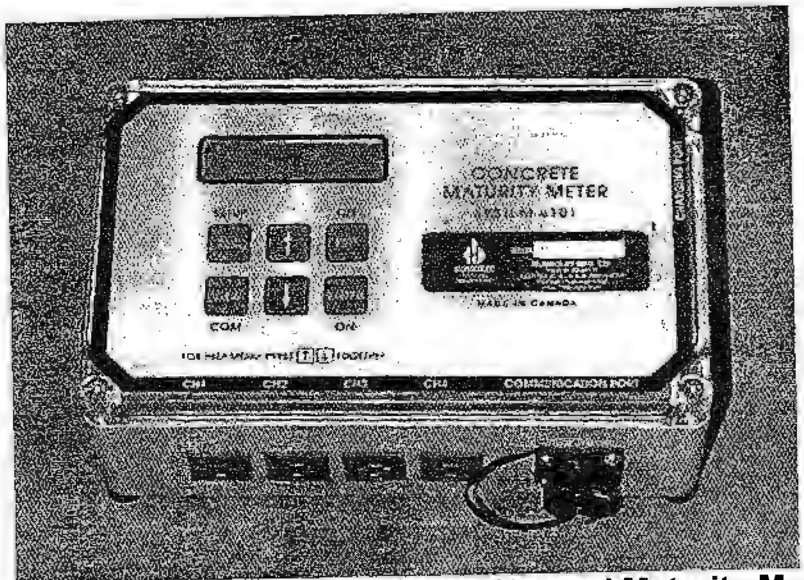


Figure 4.3. Humbolt H-2680 Multi-Channel Maturity Meter

TEST POUR RESULTS

Batch ticket information and fresh concrete properties from the test pour are given in Table 4.8. Time-temperature results of both match curing systems used at the test pour are given in Tables 4.9 and 4.10. Plots of this data are shown in Figures 4.4 and 4.5.

Project specifications required the use of a match curing system capable of maintaining the temperature of the test cylinders within 5°F of the product temperature. Results from the test pour indicated that the Contractor's match curing system did not meet this specification, while the Sure Cure™ match curing system performed satisfactorily.

DESCRIPTION OF BATCHING, MIXING, PLACING, CURING, AND TESTING PROCEDURES USED IN THE PRODUCTION OF THE HPC GIRDERS

Batching of the concrete was computer controlled with the desired weights of all the materials entered in the computer and controlled from a room next to the mixer. The mixer used at was a Simem™ with a capacity of 5 cubic yards. Type I and III cement and Type C fly ash were stored in separate silos above the mixer. Stockpiles of the different size coarse aggregates and sand were located next to the batching facility. The coarse aggregate and sand were delivered to the storage bins located above the mixer via the conveyor belt system shown in Figure 4.6. Separate bins were used to store the different size aggregates and sand before they were dropped into the hopper and weighed. The moisture content of the fine aggregate was measured by a probe located at the bottom of

Table 4.8. Batch Ticket Information and Fresh Concrete Properties from the Test Pour

		Average Mix Proportions (lbs/yd^3)						Admixtures			Fresh Concrete Properties	
								(oz/cwt)		(oz)		
Total Yards	Number Of Batches	Cement Type III	Fly Ash Type C	#67 Limestone	#89 Sand	#100 Sand	Water	Rheobuild® 1000	Delvo®	Micro Air®	Air (%)	Slump (in.)
8	2	755	135	1763	360	710	265	16	3.5	4	2.7	7.5

Figure 4.4. Temperature History of Sure Cure™ Match-Cure Cylinders Used at Test Pour

Table 4.9 Time-Temperature Data for Sure Cure™ Match-Cure Cylinders used at Test Pour

Time (hrs)	0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
Product Temperature (ºF)	94	96	97	103	126	150	156	157	155	153	150	146	142	139	135	131	127	122
Sure Cure™ Temperature (ºF)	94	97	97	104	126	149	155	156	155	153	150	146	143	139	135	131	126	123

Figure 4.5. Temperature History of Sure Cure™ Match-Cure Cylinders Used at Test Pour

Table 4.10 Time-Temperature Data for Contractor's Match-Cured Cylinders used at Test Pour

Time (hrs)	0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
Product Temperature (°F)	95	95	93	90	110	149	157	158	157	155	153	150	146	142	137	-	-	-
CMC Temperature (°F)	96	100	95	95	106	133	146	150	151	153	154	153	153	151	148	-	-	-

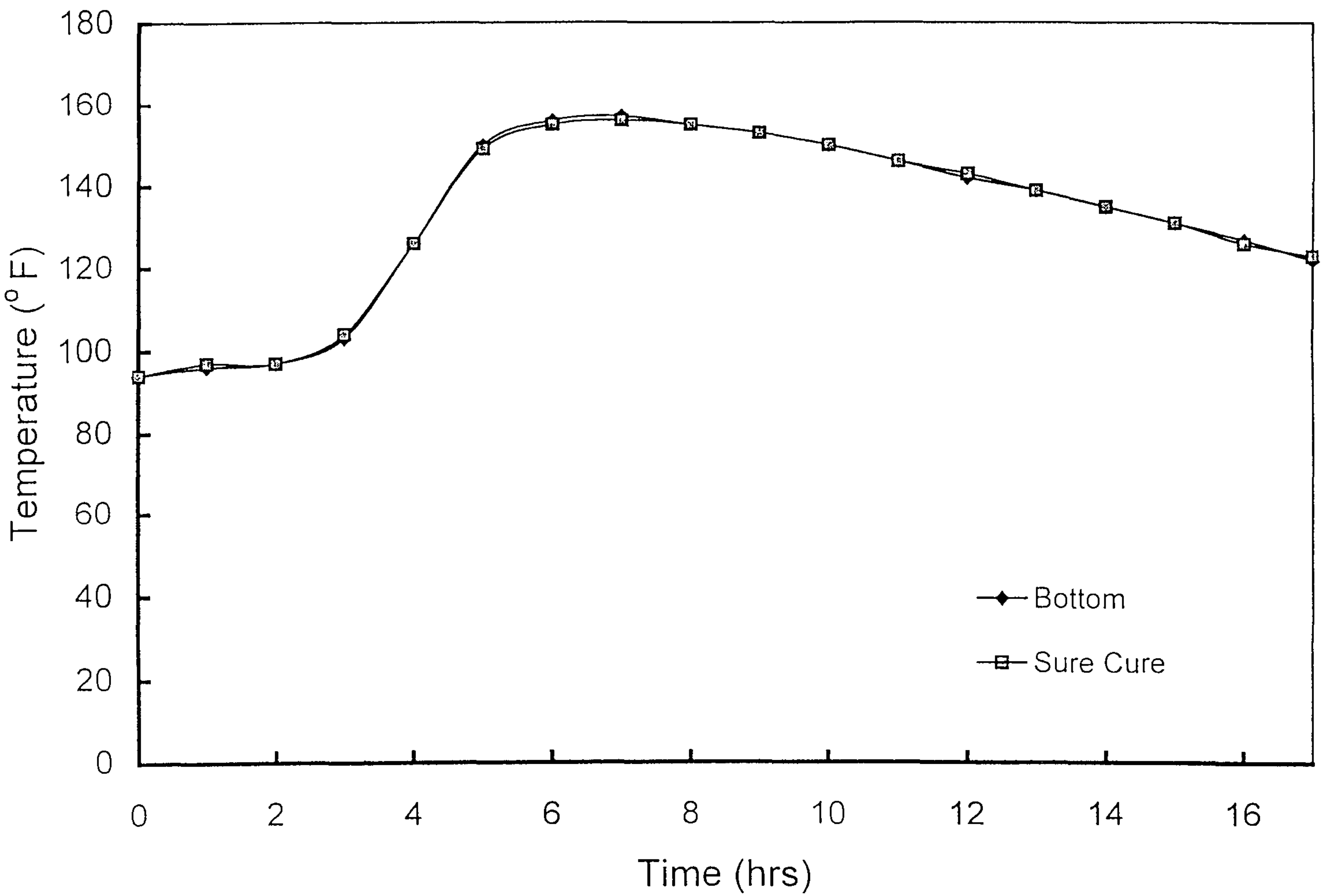


Figure 4.4. Temperature History of Sure Cure™ Match-Cure Cylinders Used at Test Pour

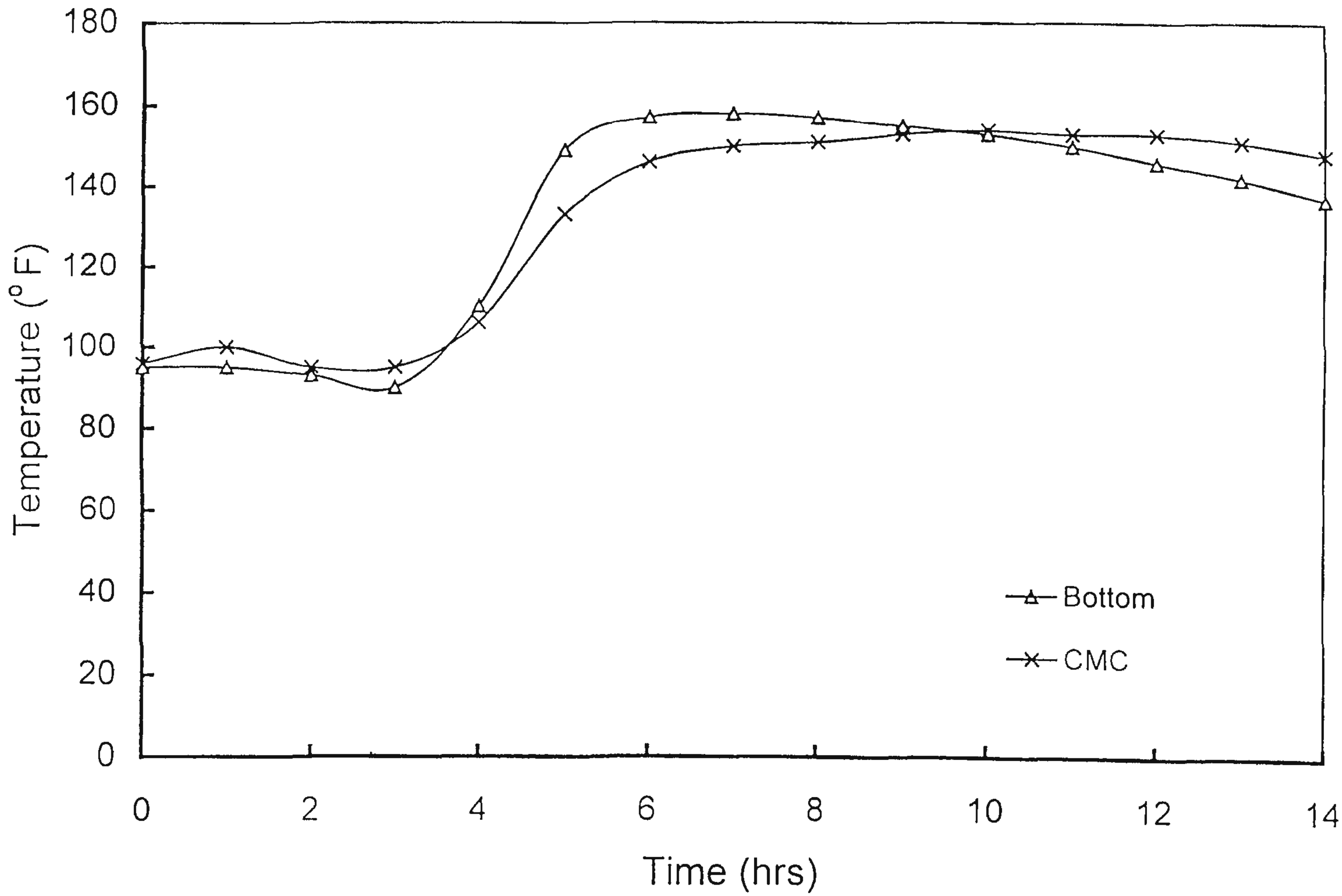


Figure 4.5. Temperature History of Contractor's Match-Cure Cylinders Used at Test Pour

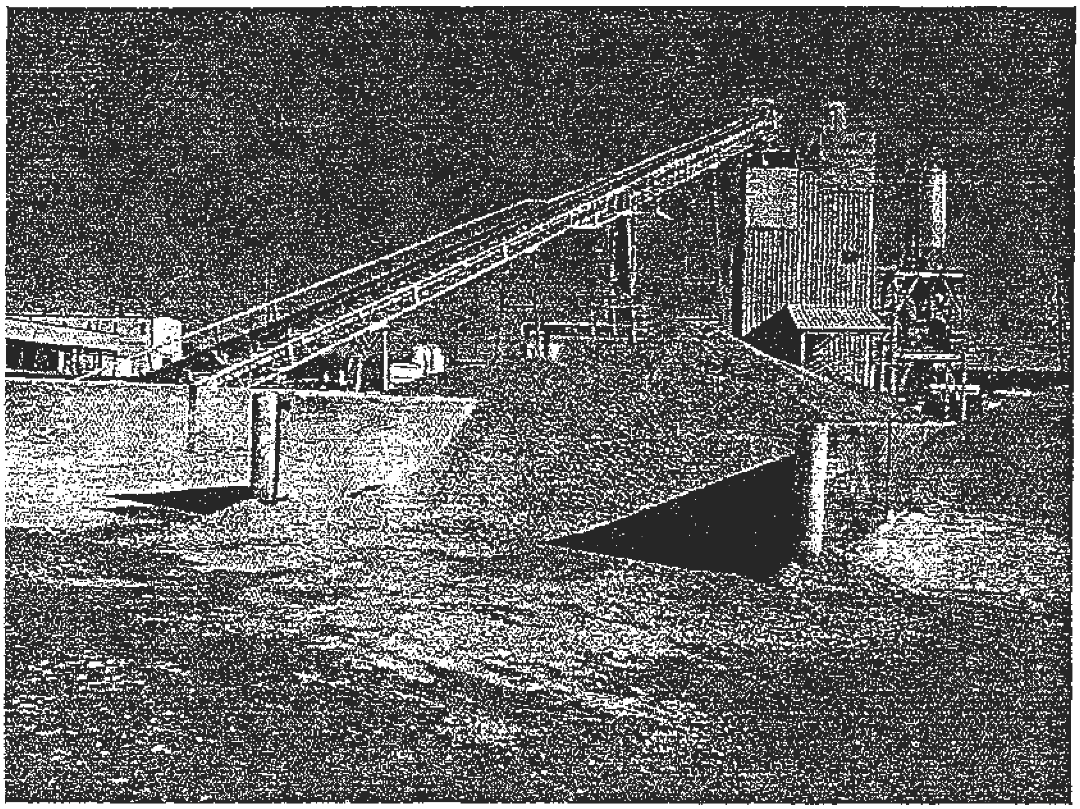


Figure 4.6. Conveyor System Used to Transport Aggregate to Batch Plant

the bin as the material was dropped into the hopper. The moisture content of the coarse aggregate was assumed to be 1.5%, which was verified once a day. The batch plant at Sherman Prestressed Concrete is shown in Figure 4.7.

Mixing of HPC

The HPC was mixed in the following sequence:

1. The sand and coarse aggregate were released into the hopper and weighed.
2. The sand and coarse aggregate were then dropped into the rotating mixer.
3. An air-entraining agent and retarding admixture were added along with cement, fly ash and water.
4. Superplasticizer was added approximately 20 seconds after all the materials were added.
5. Once all the materials were in, the concrete was mixed for approximately 90 seconds.

The mix was deemed ready for placement after the amperage to rotate the mixer dropped from its initial reading of 60 amps to approximately 30-35 amps. This drop in amperage indicated that the slump of the concrete was in the desired range.

Placement of HPC

Once the concrete was ready to leave the batch plant, it was dropped from the mixer into a sidewinder, which brought the concrete to the front of the batch

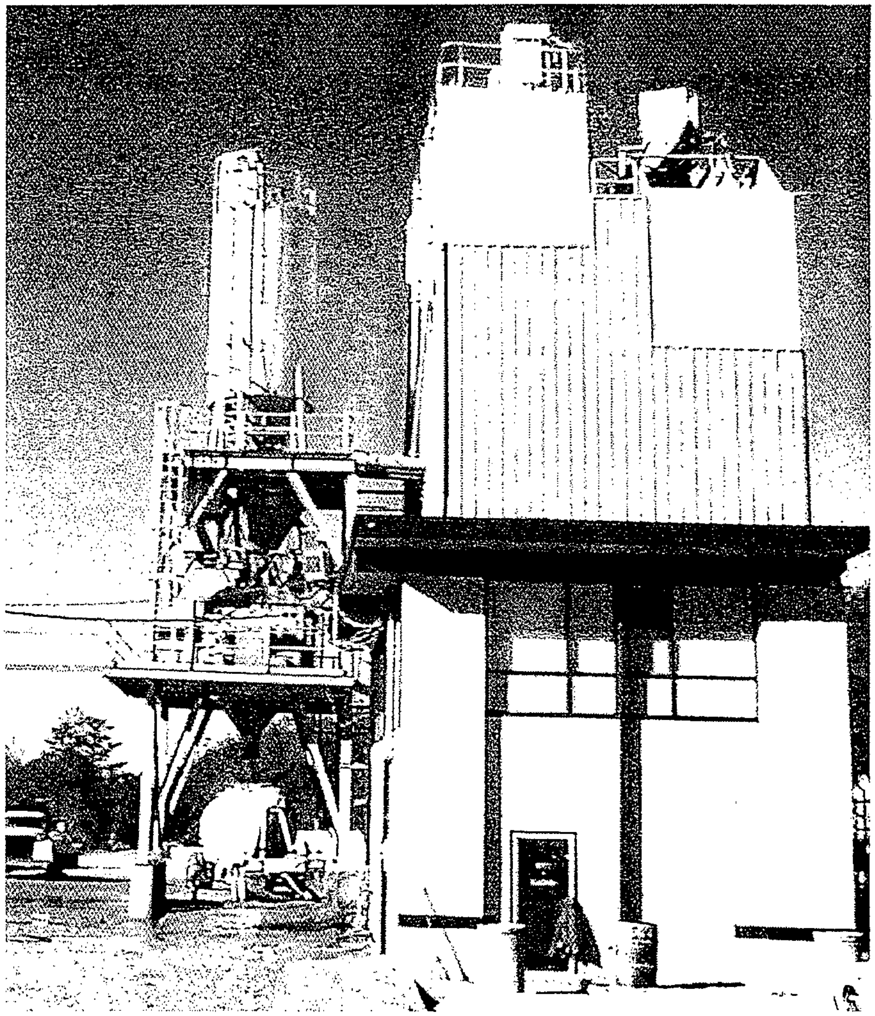


Figure 4.7. Batch Plant at Sherman Prestressed Concrete

plant where a sample was taken from the delivery chute for air content and slump tests. Upon approval by the ALDOT inspector, the sidewinder was driven to the casting bed where the concrete was placed. A picture of a sidewinder placing concrete in the forms is shown in Figure 4.8. This entire process took approximately 5 minutes. It should be noted that the concrete was not being mixed while in the sidewinder. Concrete was placed in each of the girders in 3 lifts. A vibrator located on the side of the formwork as well as workers with hand held immersion vibrators were used to consolidate the concrete into the forms. This is shown in Figure 4.9. Finishing of the concrete was accomplished by troweling off the top and leaving a rough surface. Insulated tarps were placed over both of the girders using Mi Jack Travel Lift™ cranes. This process is shown in Figures 4.10 and 4.11.

Curing of HPC Girders

An intermittent steaming process was used to cure the girders. After initial set of the concrete, which typically took 4 to 5 hours, steam was applied to the member through a pipe located directly underneath the casting bed. The pipe ran the full length of the casting bed and distributed the steam to both sides of the member. The steam was produced by a Weldon Steam Generator™. Steam was applied to the girders to aid in the hydration process until the internal temperature of the concrete reached 140°F. Through the use of a thermocouple located at the center of mass at the bottom of the girder cross section, the steam generator automatically turned off when the internal temperature of the concrete

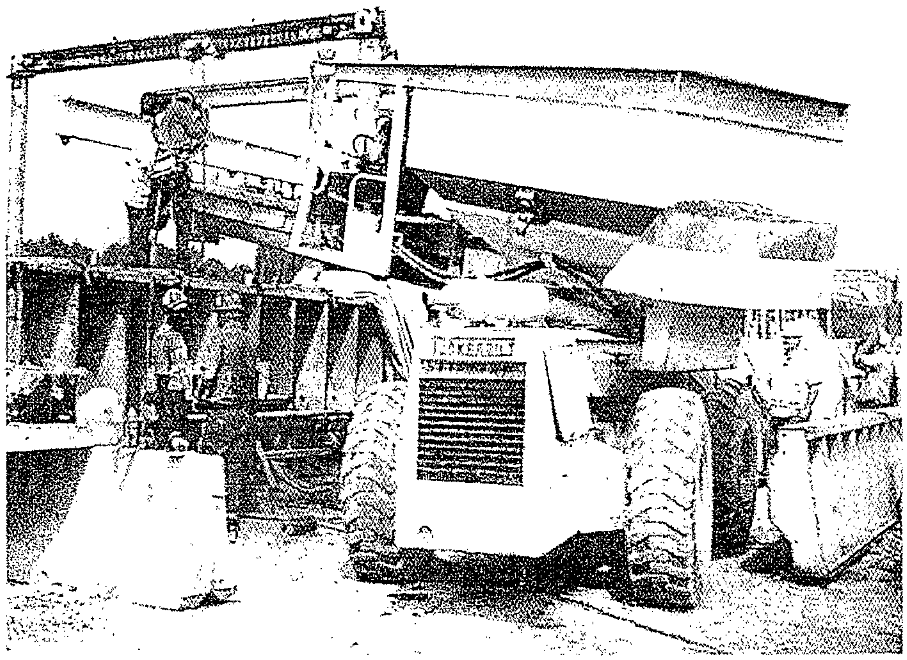


Figure 4.8. Concrete Being Placed Into Form by Sidewinder

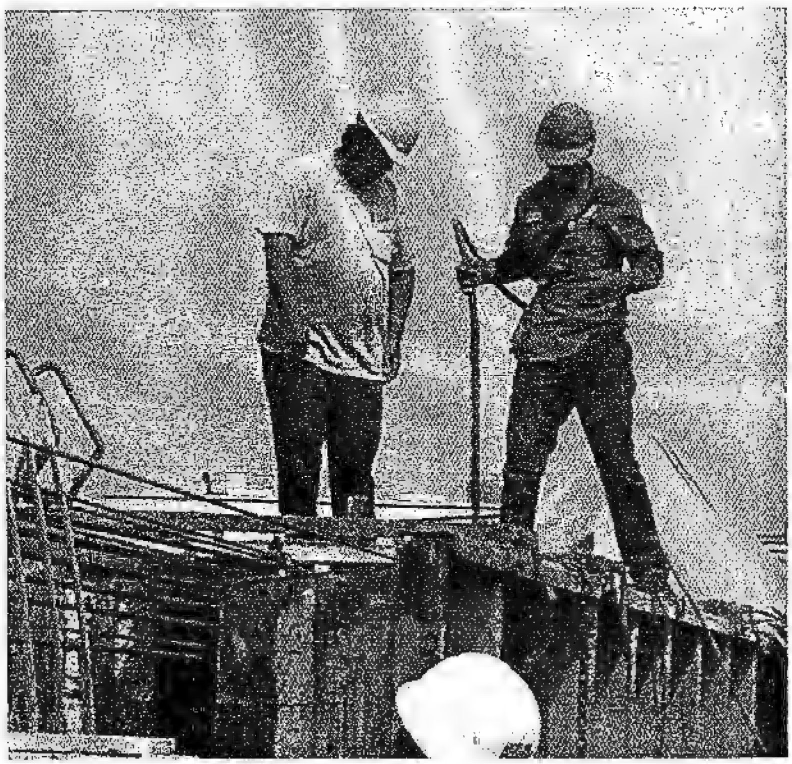


Figure 4.9. Concrete Being Vibrated Into the Form

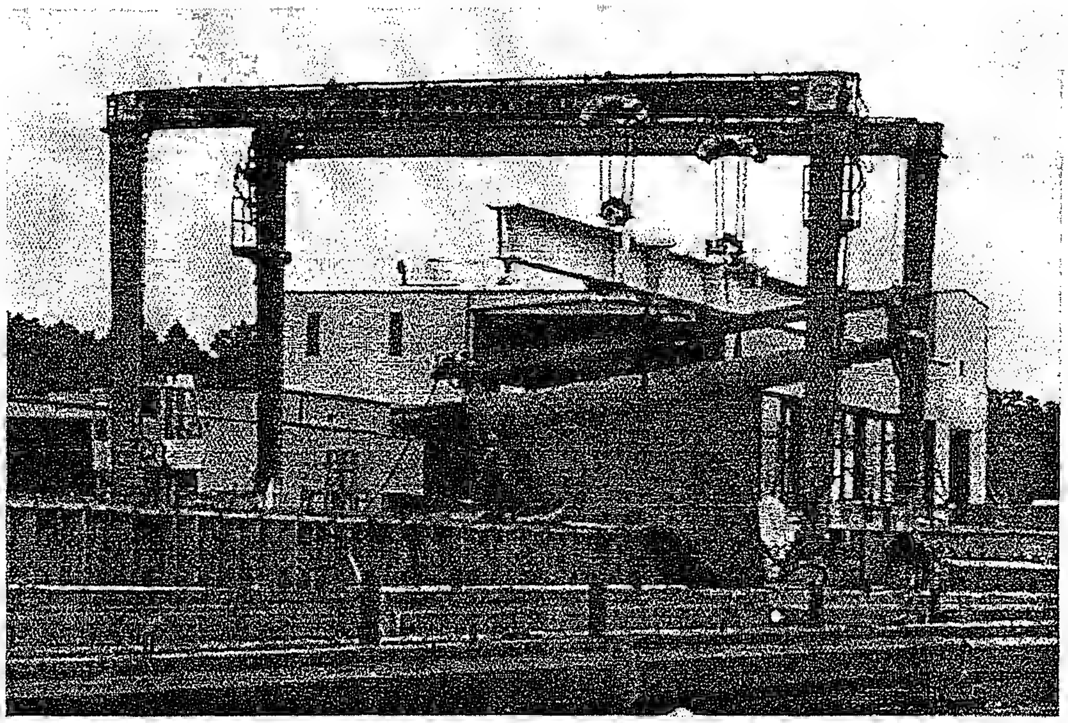


Figure 4.10 Insulated Tarps Being Placed Over Girders

Figure 4.11. AASHTO BT-54 Girder Covered With Insulated Tarp

got above 140ºF. The use of insulated tarps covering the girders aided in retaining the heat generated by this process. The internal temperature of the concrete never got above 160°F. The duration of steam being applied to the girders was highly dependent on weather conditions. If the internal temperature of the concrete fell below 140ºF at any time during curing, the steam generator automatically turned back on and reapplied steam to the girders. During production of the HPC girders, there were many instances when the steam was reapplied to the girders to maintain the temperature above 140ºF.

Testing Procedures

Release strength of the member was determined from the Sure Cure™ match-cured cylinders shown in Figure 4.12. Release strength cylinders were typically tested between 18 and 24 hours after concrete was placed in the forms. Release of the prestressing force was permitted once the average compressive strength of two match-cured cylinders tested on sulfur caps was above 8000 psi, with neither of the two cylinder strengths being below 7600 psi. After the release strength was achieved; the remainder of the Sure Cure™ match-cured cylinders were placed on storage racks located next to the batch plant for final curing. The cylinder storage racks are shown in Figure 4.13. Tests conducted on the concrete in the fresh and hardened states were performed by ACI certified personal from Sherman Prestressed Concrete.

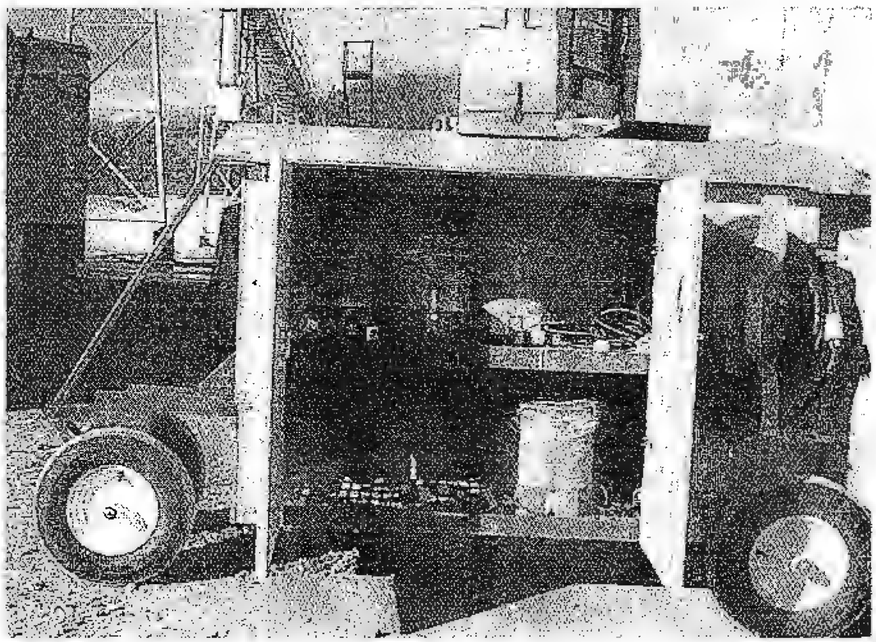


Figure 4.12. Sure Cure™ Cylinders Being Cured

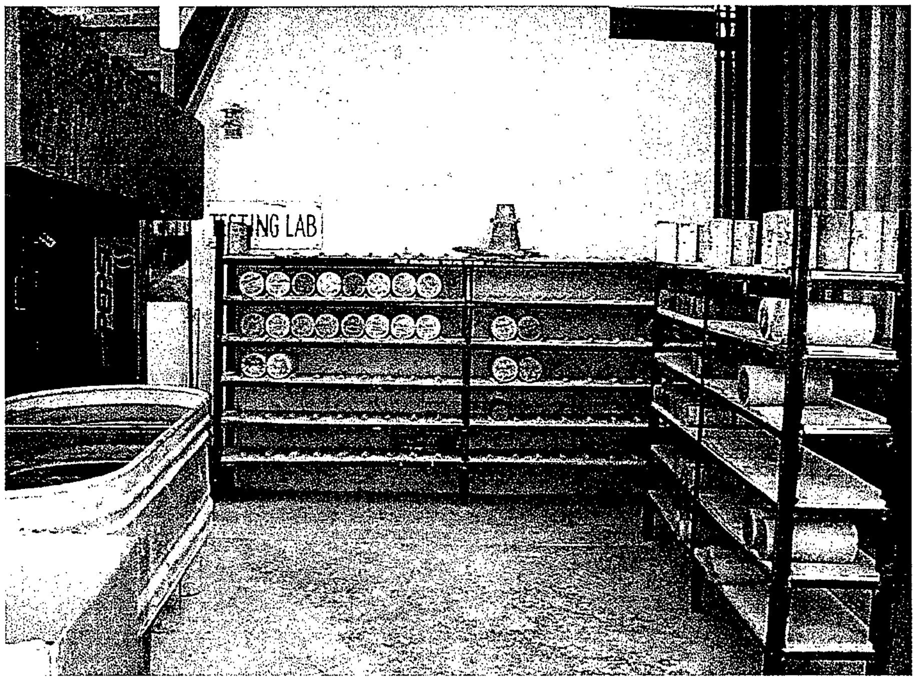


Figure 4.13 Cylinder Curing Racks at Sherman Prestressed Concrete

Storage of Girders

Upon release of the prestressing force, the girders were transported by two Mi Jack Travel Lift™ 50-ton cranes to the yard for storage. A picture showing an AASHTO BT-54 girder being transported is shown in Figure 4.14. The girders were stored on the casting yard for curing until the required 28-day strength was achieved. The girders were subjected to all weather conditions and were not covered during final curing. The girders were approved for shipping when the average of two consecutive cylinder tests results exceeded 10000 psi and neither of the two cylinder test results was below 9500 psi. The girders were also inspected by ALDOT personal for any damage or irregularities in the girders. Once approved, the girders were shipped as needed to the bridge site and set into place.

CONTRACTOR'S TEST RESULTS FROM PRODUCTION OF HPC GIRDERS

Production of the 35 HPC AASHTO BT-54 girders was accomplished in 18 pours. Girders were cast two at a time on the same line of production. The contractor chose to cast two girders on the last pour because a girder on a previous pour did not achieve the specified strength. Pour numbers 1 through 12 utilized the mix design given in Table 4.2. The remainder of the pours 13 through 18 used the same mix design with the exception that #7 limestone was used in place of the #67 limestone. Results of compressive strength tests performed by Sherman Prestressed Concrete from production of the HPC girders are presented in Table 4.11. Batch ticket information is given in Appendix A, Tables

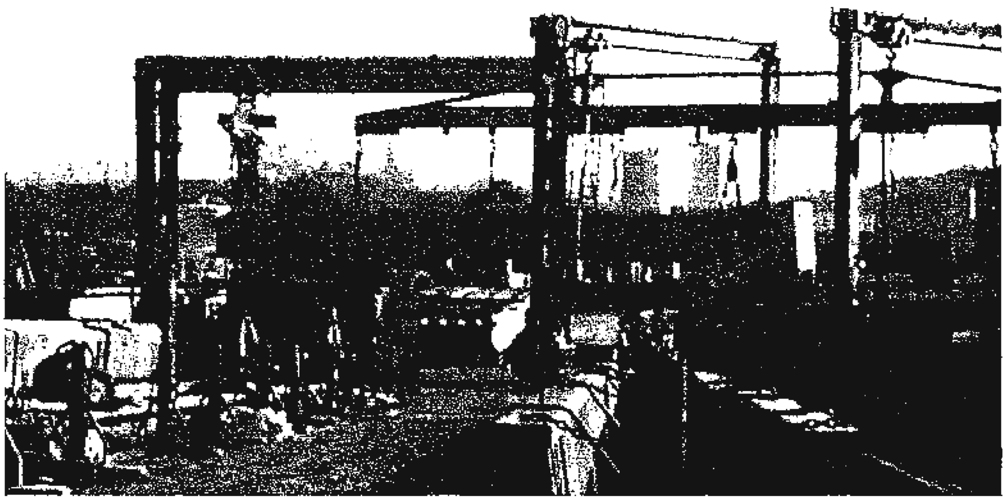


Figure 4.14. AASHTO BT-54 Girder Being Transported to Storage on the Casting Yard

Table 4.11. Results from Production of the HPC Girders[1]

Pour Number[3]	Release Time	Compressive Strength (psi) [2]			
		Release	28-Day	56-Day	Core[4]
1	24 hrs	8080	9890	10110	9730
2	42 hrs	8080	8440	9610	8200
3	19 hrs	8240	10240	10720	-
4	22 hrs	8080	9750	9470	9450
5	20.5 hrs	8300	10060	10360	-
6	22 hrs	8120	9210	8440	9720
7	5 days	8160	8540	8600	8950
8	21.5 hrs	8830	10000	10180	-
9	20.5 hrs	8040	9710	9550	8450
10	19.5 hrs	8480	9830	9670	10030
11	20 hrs	8080	10030	9710	-
12	45 hrs	8360	8950	8620	9530
13	21 hrs	9130	10820	10320	-
14	21 hrs	8680	10250	8980	-
15	19 hrs	9370	11060	11320	-
16	20 hrs	8340	10260	10540	-
17	20 hrs	9450	10900	11380	-
18	22 hrs	9810	10660	11600	-

1. Refer to Appendix A.5 for individual cylinder test results.
2. Average of 2 – 4 x 8 in. Sure Cure™ cylinders tested on sulfur caps.
3. No. 67 limestone used in pours 1 through 12, No. 7 limestone used in pours 13 through 18.
4. Core testing was conducted if the required 28-day strength was not achieved. 4 cores were cut according to AASHTO T 24. Core strengths were based on the best 3 out of 4 cores tested dry on sulfur caps without the AASHTO (L/D) ratio reduction factor applied.

A.6-A.8. Measured air content, slump, and placement temperatures are given in Appendix A, Table A.9. Project specifications required air, slump, and temperature to be measured on the first batch of concrete leaving the plant, and every 50 cubic yards thereafter. Each pour required over 50 cubic yards of concrete, resulting in two sets of air, slump, and temperature measurements. A plot of the temperature history of the girders and match curing system is given in Appendix B, Figures B.1-B.18 for each pour during the production of the HPC girders. These temperature histories were recorded by Sherman Prestressed Concrete as required by the project specifications.

CHAPTER FIVE

PRODUCTION OF HPC CAST-IN-PLACE CONCRETE

INTRODUCTION

The HPC for the cast-in-place (CIP) construction on the bridge project was supplied by Blue Circle Material's Chehaw ready-mix plant in Macon County, Alabama. The CIP construction was accomplished by the general contractor, Clark Construction, Inc. of Headland, Alabama. Contractors in the state of Alabama had limited experience working with CIP HPC. To provide a starting point for development of the HPC mixes, preliminary mix studies were performed by Auburn University, and the results were presented to all potential bidders for the project at a pre-bid meeting. Several mix designs were tested by Blue Circle Materials using three-yard batches delivered to the job site prior to casting of the substructure concrete on the Uphapee Creek Relief Bridge and Bulger Creek Bridge. An acceptable mix was chosen for the substructure, but the mix was not approved for use in the decks. The concrete mix was refined for use in the decks, and a slab test pour, as required in the project specifications was conducted by the contractor to illustrate the acceptability of the HPC mix.

REQUIRMENTS FOR CAST-IN-PLACE CONCRETE

The project requirements for the CIP HPC featured specifications not normally used by the ALDOT, as well as requirements normal for all projects. The project requirements for the CIP HPC were documented in a Special Provision to the ALDOT Standard Specifications. General requirements for the concrete, sampling and testing, and materials were covered by the ALDOT Standard Specification without changes. A summary of the special materials requirements for the CIP HPC is included in Table 5.1. The most obvious changes from a normal ALDOT project were the higher than normal compressive strength, and the required use of pozzolans and crushed limestone coarse aggregate. An additional special requirement on the construction of the bridge was the required use of wet curing for the deck slabs.

MIX PROPORTIONS

During the initial stages of the project, a series of trial mixes were made by Auburn University to identify the characteristics of HPC mixes using various combinations of pozzolans and locally available aggregates for possible use in CIP applications. These trial mix designs were based upon past HPC projects, information obtained from review of literature, and ACI (1993) "Guide to Selecting Properties for High-Strength Concrete with Portland Cement and Fly Ash." Various combinations of materials and *w/cm* ratios were studied during these trial mixes. The results of the trial mixes were used with other information to develop the concrete requirements shown in Table 5.1. The results were also presented

Table 5.1. Requirements for Cast-In-Place HPC

28-Day Strength, f'c:	6000 psi
Maximum water to cementitious materials ratio (w/cm):	0.38
Air Content:	3.5% to 6%
Maximum Temperature of Fresh Concrete:	95° F
Maximum Slump:	
Superstructure	5 in.
Substructure	8 in.
Coarse Aggregate:	Crushed Limestone ALDOT #57 or #67
Fine Aggregate:	Natural Sand

At least one pozzolan must be used.

Pozzolan Content: (percentage of total cementitious material)

Class F fly ash	15 to 25%
Class C fly ash	20 to 35%
Microsilica	7 to 15%

When combinations of fly ash and microsilica are used, the fly ash content shall satisfy the above-specified ranges, and the microsilica will be considered an addition. The microsilica addition shall be at least 3% of the total cement plus fly ash content and no more than 15%.

to Blue Circle Materials and served as a starting point for their own preliminary mix testing. Testing of potential mix designs prior to production of the CIP HPC was conducted at the Blue Circle Material's Lab as well as on three yard batches delivered to the jobsite. After approval from the ALDOT to place substructure concrete, the mix proportions shown in Table 5.2 were used on the Uphapee Creek Relief Bridge. Problems with low compressive strength test results and concerns over workability of the concrete caused the contractor to refine the mix design to the one shown in Table 5.3. The mix design given in Table 5.3 was used for the remainder of the Uphapee Creek Relief Bridge substructure and Bulger Creek substructure. Also, as part of the HPC project specifications, a slab test pour was required before any superstructure concrete could be placed.

TEST POUR

On April 22, 1999 a slab test pour was conducted at the bridge site on a 20 ft x 20 ft and 3½ in. thick test slab to provide evidence of the Contractor's ability to mix, haul, handle, place, consolidate, finish, and cure the HPC mix proposed for use in the production of the CIP HPC decks. The mix proportions used at the test pour are shown in Table 5.4. This mix design was refined slightly from the previous mix design given in Table 5.3. A major concern of the Contractor was having a concrete mixture that was not too sticky and could be finished by conventional methods. Other objectives of the test pour included: meeting 28-day strength requirements, and demonstration of the ability to meet the air content, slump, finishing and curing requirements.

Table 5.2. Mix Proportions of Mix HPC-2

Material	Source	Cubic Yard Proportions
Type II Cement	Blue Circle Roberta	640 lbs
Type C Fly Ash	Holnam	160 lbs
#57 Limestone	Martin Marietta Auburn	1950 lbs
Sand	Martin Marietta Shorter	990 lbs
Water	---	300 lbs
Water Reducer	MB Polyheed® 977	8 oz/cwt
Retarder	MB Pozzolith® 100-XR	---
Air Entrainment	MB AE® 90	0.45 oz/cwt
Water/Cementitious Materials Ratio = 0.38		

Table 5.3. Mix Proportions of Mix HPC-3

Material	Source	Cubic Yard Proportions
Type II Cement	Blue Circle Roberta	640 lbs
Type C Fly Ash	Holnam	160 lbs
#57 Limestone	Martin Marietta Auburn	1950 lbs
Sand	Martin Marietta Shorter	990 lbs
Water	---	300 lbs
Water Reducer	MB Polyheed® 977	10 oz/cwt
Retarder	MB Pozzolith® 100-XR	2 oz/cwt
Air Entrainment	MB AE® 90	0.45 oz/cwt
Water/Cementitious Materials Ratio = 0.38		

Table 5.4. Mix Proportions of Mix HPC-4

Material	Source	Cubic Yard Proportions
Type II Cement	Blue Circle Roberta	658 lbs
Type C Fly Ash	Holnam	165 lbs
#57 Limestone	Martin Marietta Auburn	1860 lbs
Sand	Martin Marietta Shorter	1042 lbs
Water	---	288 lbs
Water Reducer	MB Polyheed® 977	10 oz/cwt
Retarder	MB Pozzolith® 100-XR	3 oz/cwt
Air Entrainment	MB AE® 90	As reqd.
Water/Cementitious Materials Ratio = 0.37		

The procedure used to place the concrete at the test pour was basically the same as if the contractor was placing an actual bridge deck. Formwork was prepared on site in advance with no reinforcement in the formed area. Shugart™ screed equipment was set up and ready for finish work as shown in Figure 5.1. After slump and air content were measured, the formwork was filled with concrete by means of a concrete bottom dump bucket with clamshell gate lifted by crane to simulate deck pour conditions. Concrete was moved manually with shovels and consolidated with a mechanical immersion vibrator. After distribution of concrete over the form, the Shugart™ screed was used for initial leveling. Because of the concern of the finishability of the mix, five different methods of finishing the slab were employed at the test pour.

On approximately two-thirds of the slab area, an evaporation reduction compound was applied to the slab after the first and second pass of the screed. This compound was used due to concerns about finishing the surface of the relatively dry mix. The evaporation reduction compound called CONFILM® was a product of Master Builders, Inc. and contained a distinct fluorescent green-yellow tint that disappeared completely upon drying when sprayed onto the surface immediately after screeding. After the application of the CONFILM®, hand finishing was performed by either a wood float or bull float on areas with and without CONFILM®. In general, areas treated with CONFILM® were noticeably easier to finish to the desired surface texture. Completion of the test slab took

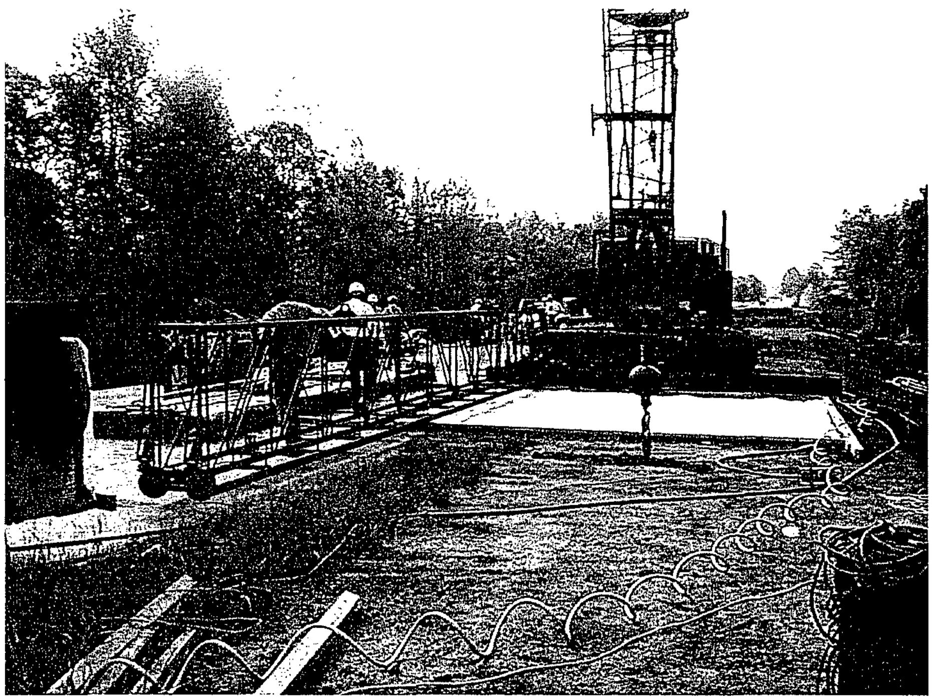


Figure 5.1. Shugart™ Screed Equipment Used at Test Pour

approximately 1-3/4 hours. Curing of the test slab involved brooming the slab areas finished by the bull float and fogging the entire slab with water vapor to help retard evaporation. Fogging of the test slab with water is shown in Figure 5.2. Water-soaked burlap was applied over the entire slab for a curing membrane after the slab had set for approximately 3-1/2 hours. The burlap was then covered with a plastic tarp, and soaker hoses were placed under the burlap to continuously wet cure the slab for seven days.

TEST POUR RESULTS

Batch ticket information from the test pour is given in Table 5.5. Standard 6 x 12 in. cylinders made in plastic molds were tested by the ALDOT at 7 and 28 days on neoprene pads to determine the compressive strength properties of the proposed superstructure concrete. Compressive strength and fresh properties results for the test slab are given in Table 5.6. Results from the slab test pour deemed the mix acceptable for use on the CIP superstructures.

DESCRIPTION OF BATCHING, PLACING, FINISHING, CURING, AND TESTING PROCEDURES

The method used by the Blue Circle Materials ready-mix plant to batch concrete and deliver it to the bridge site is presented in this section. Also the procedure used by the contractor to place, finish and cure the CIP HPC as well ALDOT's testing procedures for the CIP HPC are discussed.

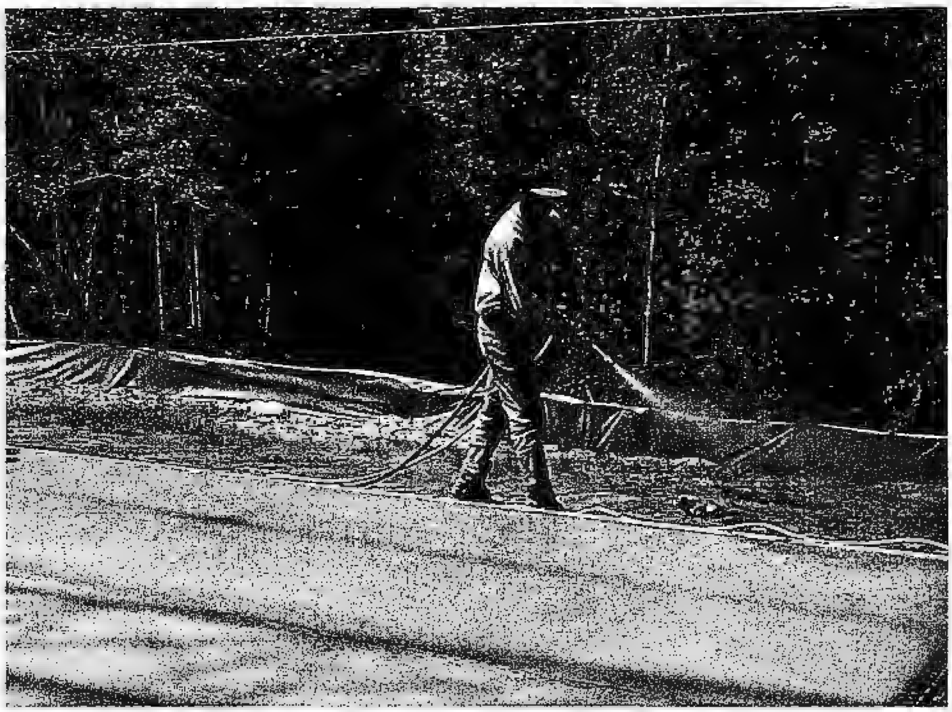


Figure 5.2. Water Fogging Being Applied to Test Slab

Table 5.5. Batch Ticket Information from the Test Pour

Mix Design	HPC-4
Cubic Yards	6.0
Delivery Time	105 min
Average Mix Proportions (lbs/yd^3)	
Type II Cement	660
Type C Fly Ash	168
Coarse Aggregate	1855
Fine Aggregate	1035
Water	303
Admixtures (oz)	
Polyheed® 997	82.3
Pozzolith® 100-XR	19.7
MB AE® 90	2.0
Average w/cm ratio	0.37

Mix proportions are dry weights.

Table 5.6. Compressive Strength and Fresh Properties Results from Test Pour

Compressive Strength (psi)				Fresh Properties		
7-Day		28-Day		Air Content (%)	Slump (in.)	Temperature (ºF)
	Avg.		Avg.			
5730 5750	**5740**	6580 7050	**6820**	3.4	6.5	72

Batching of the concrete at the ready-mix plant was a dry mix process. Materials were weighed at the ready-mix plant and dropped into concrete trucks that mixed the concrete for 70 revolutions before leaving the batch plant. Batching of the concrete was computer controlled with the desired weights of all the materials entered in the computer and controlled from a room next to the drop chute. Rear discharge ready-mix trucks with a ten cubic yard capacity were used by Blue Circle Materials ready-mix plant to deliver concrete to the jobsite. Type II cement and Type C fly ash were stored in separate silos above the drop chute where materials were released into the ready-mix trucks. Stockpiles of coarse aggregate and sand were located next to the batching facility. The coarse aggregate and sand were delivered to the storage bins via a conveyor system as shown in Figure 5.3. Separate bins were used to store the coarse aggregate and sand before they were dropped into the hopper and weighed. The moisture content of both materials was measured daily. Blue Circle Material's ready-mix plant is shown in Figure 5.4

Batching of HPC

Blue Circle Materials ready-mix plant batched the HPC in the following sequence:

1. Sand and coarse aggregate were released into the hopper and weighed. Also, cement and fly ash were weighed and released into the drop chute located above the ready-mix truck at this time.

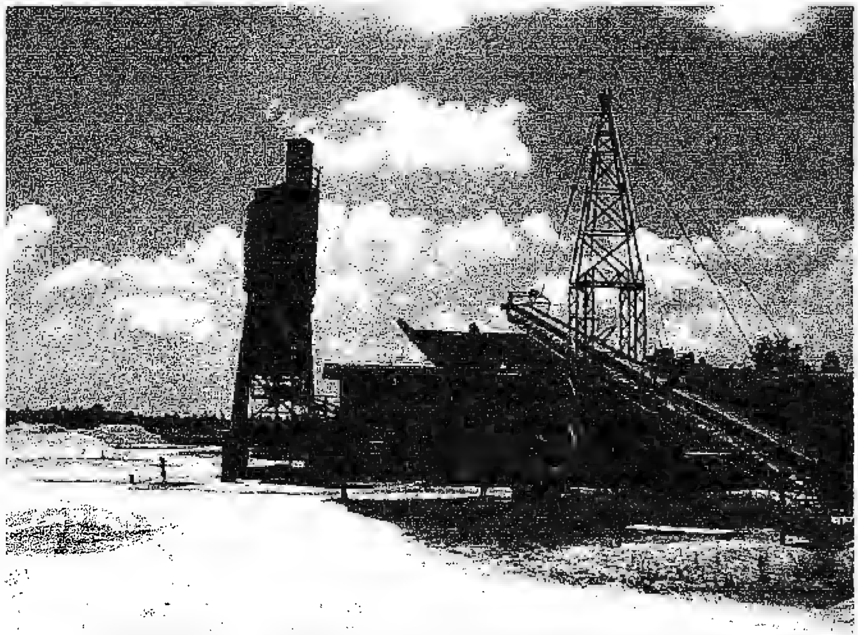


Figure 5.3. Conveyor System Used to Deliver Aggregate to Storage Bins

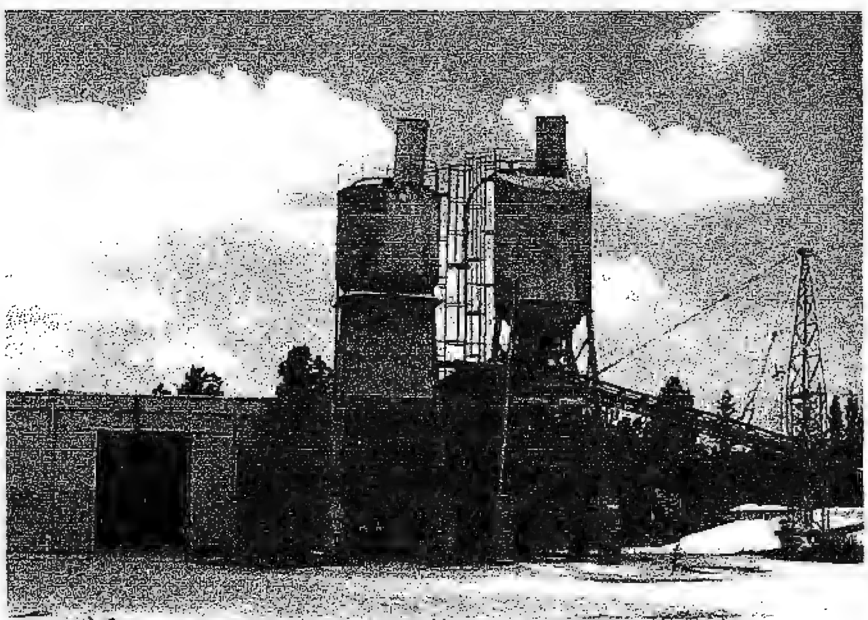


Figure 5.4. Blue Circle Material's Chehaw Ready-Mix Plant

2. A ready-mix truck located under the drop chute was loaded with approximately three-quarters of the mix water, the air entraining and retarding admixtures.
3. Coarse aggregate and sand were then released from the hopper onto a conveyor belt that brought the materials to the top of the ready-mix truck and dropped the materials into the ready-mix truck.
4. After approximately one-third of the coarse aggregate and sand were placed in the ready-mix truck, cement and fly ash were released from the drop chute into the ready-mix truck while the remainder of the coarse aggregate and sand were being let into the truck.
5. The remainder of the water was then released into the truck, followed by the water-reducing admixture.

Before leaving the ready-mix plant, air content and slump were measured by Quality Assurance personnel from Blue Circle Materials. Also, the counter measuring the number of revolutions the mixing drum turned was reset at this time. Ready-mix trucks typically hauled 7.5 to 8 cubic yard loads of concrete to the bridge site. It took approximately 25 minutes to haul the concrete from the ready-mix plant to the bridge site.

Placement of HPC

Upon arrival of the ready-mix truck at the bridge site, it was driven to a central location on the jobsite where a sample was taken from the truck to measure air content, temperature, unit weight and slump. If needed, additional air entrainment admixture and water could be added to the ready-mix truck at this time. The addition of water was only allowed if part of the approved amount of mix water had been withheld during the original batching. The amount of water that could be added was limited to the amount that was withheld. Upon approval from the ALDOT inspector, the number of revolutions the mixing drum turned was recorded and the truck was driven to the location where concrete was placed. It typically took an hour to an hour and a half from the time the mix water was added until all the concrete was emptied from the ready-mix truck.

Concrete was unloaded from the ready-mix truck into a one cubic yard bottom dump bucket as shown in Figure 5.5. The bucket was lifted by crane to the location where the concrete was placed into the formwork. Substructure concrete was dropped into wooden formwork in predetermined lifts and vibrated by mechanical immersion vibrators to consolidate each layer of concrete. Substructure concrete being placed on the Uphapee Creek Relief Bridge is shown in Figure 5.6. The Uphapee Creek Relief Bridge substructure with AASHTO Type I girders in place is shown in Figure 5.7.

Deck slab concrete on the Uphapee Creek Relief Bridge and Bulger Creek Bridge was placed one span at a time. Deck concrete was released from the bottom dump bucket onto metal deck forms and workers moved the concrete with shovels and used mechanical immersion vibrators to consolidate the concrete before the screeding process began. Deck concrete being placed on the Uphapee Creek Relief Bridge is shown in Figure 5.8.

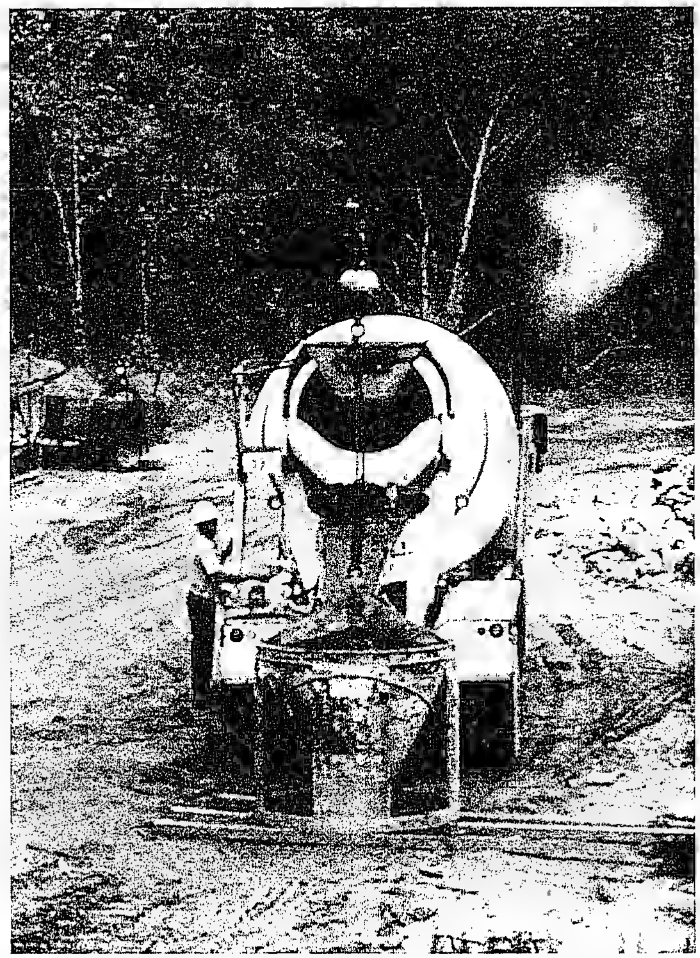


Figure 5.5. Concrete Being Loaded into Bucket

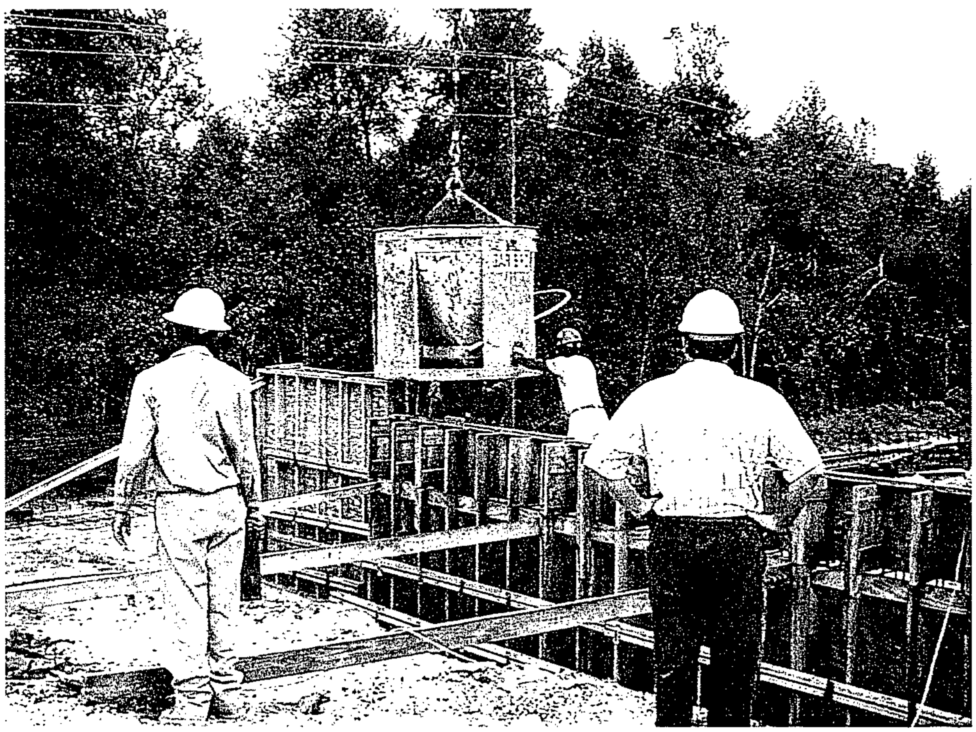


Figure 5.6. Concrete Being Placed on the Uphapee Creek Relief Bridge Substructure

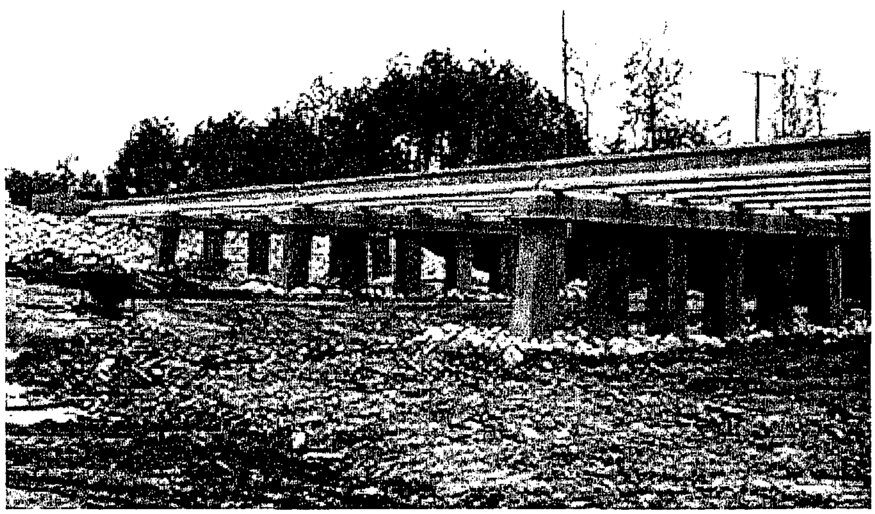


Figure 5.7. Uphapee Creek Relief Bridge Substructure with AASHTO Type I Girders

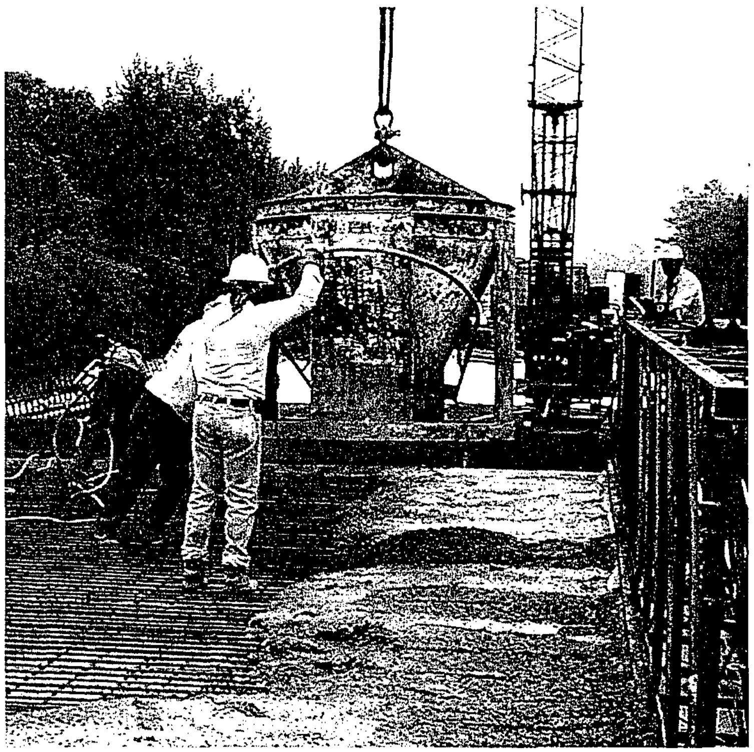


Figure 5.8. Superstructure Concrete Being Placed on the Uphapee Creek Relief Bridge

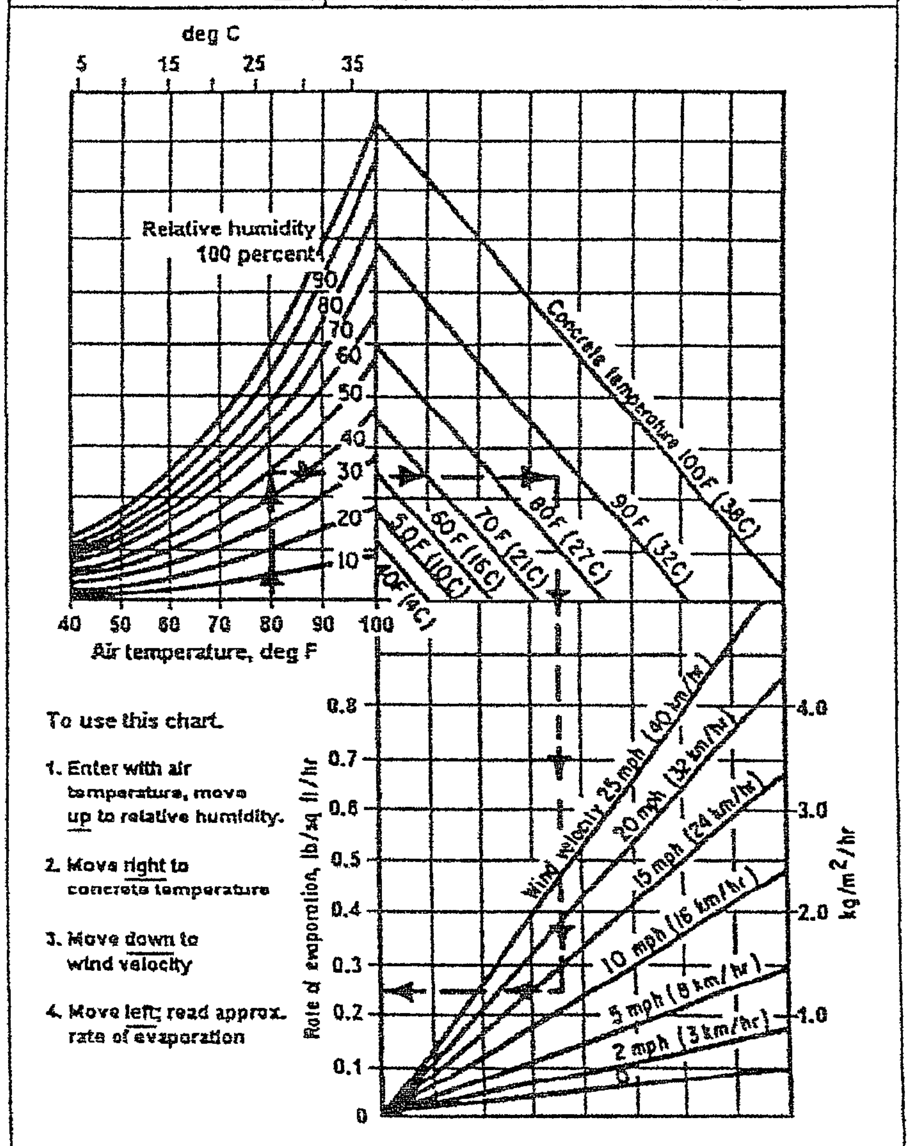
The span lengths at the Uphapee Creek Bridge were sufficiently long that multiple pours were required to cast the deck slab for each span. First the center half of each span was cast. Then pours were made which included one quarter of two adjacent spans. These pours were repeated until all deck slabs were completed.

Finishing the HPC Decks

Following the consolidation of the deck concrete, the Shugart™ screed was used for initial leveling. A longitudinal screeding process was used on the bridge decks as shown in Figure 5.9. Confilm® was applied in front of the screed that typically required three passes of the screed before hand finishing began. As needed, water fogging was used to keep the deck concrete's rate of evaporation at acceptable levels. To reduce the chance of plastic shrinkage cracking from occurring, ALDOT required the use of the chart given in Figure 5.10. Generally, deck concrete was placed early in the morning when the air temperature and wind speeds were low and the deck was immediately covered with water soaked burlap after initial set. This resulted in a very low rate of evaporation water from the bridge deck, minimizing the chance of plastic shrinkage from occurring. Final finishing of the deck concrete was accomplished by the use of a hand held wood float. Workers finishing the deck on the Uphapee Creek Relief Bridge are shown in Figure 5.11.

Figure 5.9. Longitudinal Screeding Process Used to Level the Deck Concrete

Evaporation Rate of Surface Moisture



Note: Example shown by dashed lines is for an air temperature of 80°F, relative humidity of 50 percent, concrete temperature of 87°F, and a wind velocity of 12 miles per hour. This result is a rate of evaporation of 0.24 pounds per square foot per hour.

Figure 5.10. Chart Used to Determine Evaporation Rate of Surface Moisture on Bridge Decks

Figure 5.11. Deck Concrete Being Finished by Wood Float

Curing the HPC Decks

Project specifications required the deck concrete to be kept continuously wet for the first seven days after casting. Wet curing of the HPC decks was accomplished with the following sequence. After wood floating and initial set of the concrete, water soaked burlap was placed over the entire deck surface. Soaker hoses were placed in the longitudinal direction on each side of the centerline of the bridge deck under the burlap. Water flowed away from the crown of the roadway and soaked the burlap. The burlap had a plastic coating on one side to retain moisture. Also, an additional plastic cover was applied over the wet burlap to aid in reducing the evaporation of water being applied to the deck. The Contractor used two portable water tanks of 500 and 1000 gallons each to supply water to the bridge deck. Water was pumped under the tarp a few hours a day for seven days to maintain adequate moisture on the deck concrete. A portable water tank used by the Contractor to hold water is shown in Figure 5.12.

Curing of HPC Members

Project specifications required structural members other than the deck concrete to be cured by either a membrane forming curing compound, or impervious sheeting. The contractor used a membrane forming curing compound on all of the substructure concrete. The membrane forming curing compound was applied immediately after removal of the formwork. Before applying the curing compound, the surfaces of the concrete was thoroughly

Figure 5.12. Water Tank Used to Hold Curing Water

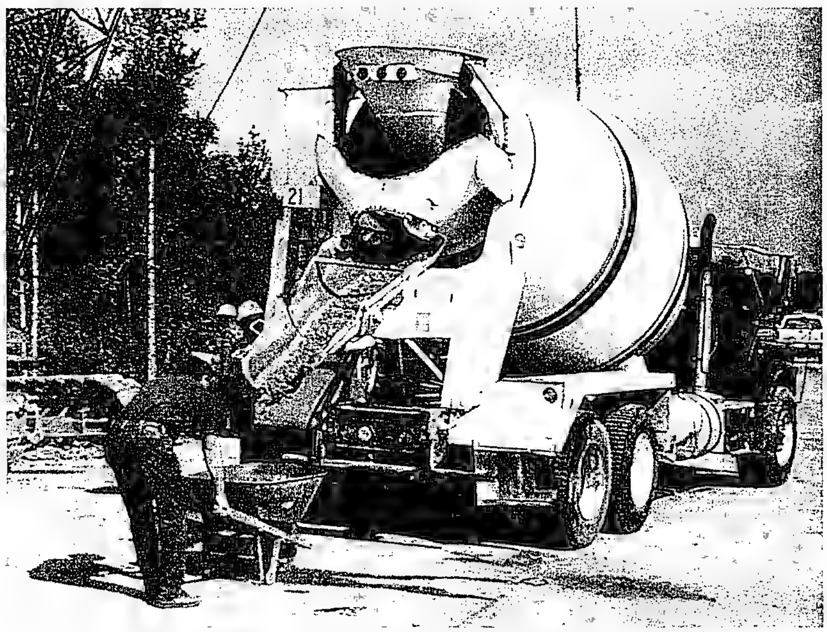


Figure 5.13. ALDOT Personnel Obtaining Concrete Samples for Testing

moistened with water, then the curing compound was applied.

Testing the HPC

Air content, concrete temperature, unit weight and slump was measured on every load of concrete arriving at the bridge site. An ALDOT worker obtaining concrete for testing is shown in Figure 5.13. Also, ALDOT made a set of compressive strength test cylinders consisting of four standard 6 x 12 in. cylinders made in plastic molds for every 50 cubic yards of concrete arriving at the bridge site. Test cylinders were made by ACI certified personnel from ALDOT and were placed in a water bath for initial curing as shown in Figure 5.14. The water bath maintained the test cylinders between 60 and 80 degrees Fahrenheit. Test cylinders were removed from the water bath between 24 and 48 hours after they were made and taken to the ALDOT central testing lab, where the plastic moulds were stripped and the cylinders placed in a moist room for final curing. Compressive strength testing was conducted at 7 and 28 days on neoprene pads.

TEST RESULTS FROM PRODUCTION

Production of the CIP HPC for the Uphapee Creek Relief Bridge and Bulger Creek Bridge took place over an eight month period. As mentioned already, three different mix designs were utilized during production of the CIP HPC. Mix designs given in Tables 5.2 and 5.3 were used on the Uphapee Creek Relief Bridge substructure. The mix design given in Table 5.3 was used on the

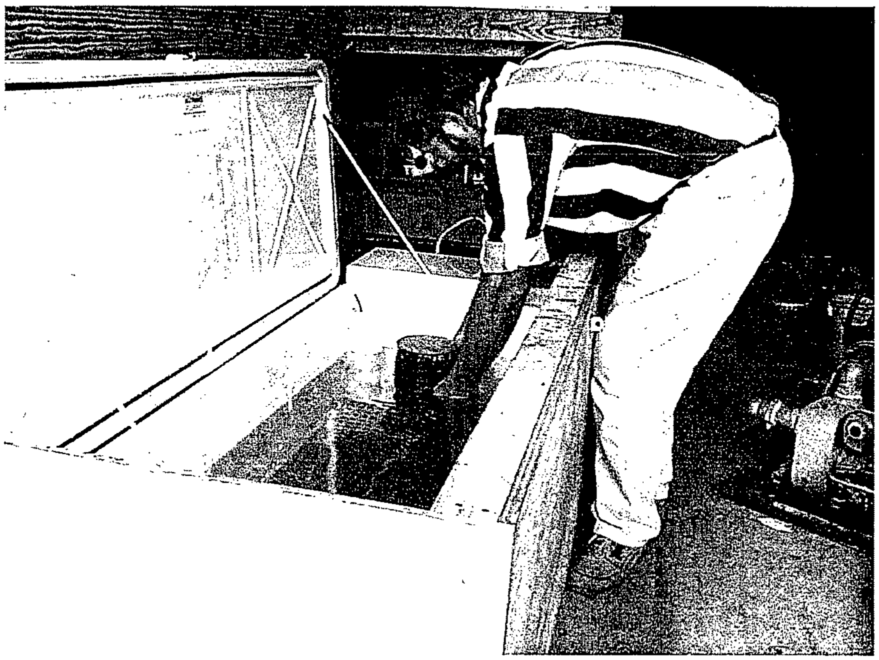


Figure 5.14. Cylinders Being Placed into the Water Bath for Initial Curing

Table 5.7. Results from Production of the CIP HPC Substructure at the Uphapee Creek Relief Bridge

Date	Location or Station Number	Mix Number	Compressive Strength (psi)	
			7-Day	28-Day
10/5/98	357+14	HPC-2	4670	5680
10/5/98	357+14	HPC-2	3950	4250
10/9/98	357+96	HPC-2	4900	6210
10/14/98	357+14	HPC-2	5250	6580
10/21/98	358+37	HPC-2	4710	6080
10/28/98	358+78	HPC-3	5100	6220
11/3/98	358+37	HPC-3	4990	6490
11/10/98	358+78	HPC-3	4940	6440
11/11/98	359+19	HPC-3	4970	6530
11/18/98	359+19	HPC-3	4790	6560
2/5/99	359+60	HPC-3	4650	6140
2/23/99	357+14	HPC-3	4120	5760
2/26/99	Webwalls	HPC-3	5240	6700
3/2/99	Webwalls	HPC-3	5040	6410
3/5/99	Webwalls	HPC-3	5440	6570

Table 5.8. Results from Production of the CIP HPC Substructure at the Bulgar Creek Bridge

Date	Location or Station Number	Mix Number	Compressive Strength (psi)	
			7-Day	28-Day
11/16/98	414+27	HPC-3	4960	6290
11/19/98	414+27	HPC-3	4720	6300
11/30/98	412+62	HPC-3	5230	6740
12/3/98	413+72	HPC-3	4840	6510
12/9/98	412+62	HPC-3	5420	6810
12/14/98	413+17	HPC-3	5620	7320
3/17/99	Webwalls	HPC-3	5200	7070
3/26/99	Webwalls	HPC-3	4980	6340

Table 5.9 Results from Production of the CIP HPC Substructure at the Uphapee Creek Bridge

Date	Location or Station Number	Mix Number	Compressive Strength (psi)	
			7 - Day	28 - Day
10/5/99	349+86	HPC - 4	5560	6730
10/6/99	351+01	HPC - 4	5680	6820
10/7/99	346+44	HPC - 4	5340	6500
10/8/99	346+44	HPC - 4	5380	6790
10/12/99	349+86	HPC - 4	5260	6500
10/14/99	351+01	HPC - 4	6200	7220
10/15/00	347+58	HPC - 4	5890	7040
10/18/00	352+14	HPC - 4	6350	7220
10/18/00	347+58	HPC – 4	5980	6940
10/19/00	348+72	HPC – 4	6400	7300
10/20/00	348+72	HPC - 4	5700	7200
10/22/00	353+28	HPC - 4	5840	7180
10/22/00	353+28	HPC - 4	6740	7175
10/25/00	353+28	HPC - 4	6010	7380
10/26/00	345+30	HPC - 4	6160	7680
10/27/00	348+72	HPC - 4	5620	7020
10/28/00	345+30	HPC - 4	5720	7460
10/28/00	347+58	HPC - 4	6300	7600
10/29/00	346+44	HPC - 4	5820	7190

Table 5.10. Results from Production of the CIP HPC Superstructure at the Uphappee Creek Relief Bridge

Date	Location or Station Number	Mix Number	Compressive Strength (psi)	
			7-Day	28-Day
4/27/99	Span 2	HPC-4	4480	6260
4/28/99	Span 4	HPC-4	6030	7200
4/29/99	Span 6	HPC-4	5010	6250
5/4/99	Span 1	HPC-4	5590	6820
5/5/99	Span 3	HPC-4	5420	6770
5/7/99	Span 5	HPC-4	5050	6520

Table 5.11. Results from Production of the CIP HPC Superstructure at the Bulgar Creek Bridge

Date	Location or Station Number	Mix Number	Compressive Strength (psi)	
			7-Day	28-Day
5/12/99	Span 1	HPC-4	5210	6740
5/12/99	Span 1	HPC-4	5460	6590
5/13/99	Span 3	HPC-4	5680	7170
5/13/99	Span 3	HPC-4	5320	6820
5/18/99	Span 2	HPC-4	5330	6520
5/18/99	Span 2	HPC-4	4960	6340

Table 5.12. Results from Production of CIP HPC Superstructure at the Uphapee Creek Bridge

Date	Location or Station Number	Mix Number	Compressive Strength (psi)	
			7 - Day	28 - Day
10/29/99	Webwalls	HPC - 4	6000	7340
11/3/00	Webwalls	HPC - 4	5730	6660
11/5/00	Webwalls	HPC - 4	5620	7100
11/9/00	Webwalls	HPC - 4	5400	7040
11/15/00	Span 5 Center	HPC - 4	6180	7500
11/15/00	Span 5 Center	HPC - 4	5690	7000
11/16/00	Span 6 Center	HPC - 4	6340	7840
11/16/00	Span 6 Center	HPC - 4	5700	7010
11/16/00	Webwalls	HPC - 4	5640	6940
11/18/00	Span 7 Center	HPC - 4	5000	6000
11/18/00	Span 7 Center	HPC - 4	5730	6700
11/18/00	Webwalls	HPC - 4	5020	5960
11//19/00	Span 5 North ¼	HPC - 4	6010	7100
11/22/00	Span 6 South ¼	HPC - 4	5780	6840
11/23/00	Span 6 North ¼	HPC - 4	5860	6970
11/24/00	Span 7 North ¼	HPC - 4	5620	7140
11/29/00	Span 7 South ¼	HPC - 4	5120	6510
11/30/00	Span 5 South ¼	HPC - 4	5860	7360
12/2/00	Span 4 Center	HPC - 4	5720	6900

Table 5.12. Continued

12/2/00	Span 4 Center	HPC - 4	5470	7300
12/6/00	Span 3 Center	HPC - 4	5760	8280
12/6/00	Span 3 Center	HPC - 4	6560	7760
12/7/00	Span 4 North ¼	HPC - 4	5640	7100
12/8/99	Span 4 South ¼	HPC - 4	5820	7070
12/14/99	Span 2 Center	HPC - 4	6600	8000
12/14/99	Span 2 Center	HPC - 4	6640	7950
12/15/99	Span 1 Center	HPC - 4	6570	8180
12/ 15/99	Span 1 Center	HPC - 4	6830	7540
12/16/99	Span 3 North ¼	HPC - 4	6600	8140
12/16/99	Span 3 South ¼	HPC - 4	6670	8200
12/20/99	Span 2 South ¼	HPC - 4	6320	7560
12/22/99	Span 2 North ¼	HPC - 4	5920	6960
12/23/99	Span 1 South ¼	HPC - 4	6240	7700
12/23/99	Span 1 North ¼	HPC - 4	6060	7790

Table 5.13. Strength Test Statistics for CIP HPC

Mix Number	7-Day Compressive Strength (psi)			28-Day Compressive Strength (psi)		
	n	Average	Std. Dev.	n	Average	Std. Dev.
HPC-2	5	4700	480	5	5760	900
HPC-3	18	5010	340	18	6510	350
HPC-4[1]	12	5300	400	12	6670	320
HPC-4[2]	53	5910	440	53	7220	510
HPC-4[3]	65	5800	490	65	7120	530

[1]All results from April and May 1999.
[2]All results from October through December 1999.
[3]All results.

Table 5.14. Compressive Strength Results Used for Approval

Mix Number	Compressive Strength (psi)	
	7-Day	28-Day
HPC-2[1]	5660	7250
HPC-2[1]	6190	7260
HPC-3[2]	6160	7490

1. Test cylinders made in the laboratory. Strengths are the average of three individual cylinder test results.
2. Test cylinders made at jobsite. Strengths are the average of two individual cylinder test results.

Bulger Creek substructure. The mix design given in Table 5.4 was used on the superstructures of the Uphapee Creek Relief Bridge and Bulger Creek Bridge, and on the substructure and superstructure of the Uphapee Creek Bridge. Results of compressive strength tests performed by the ALDOT during production of the HPC substructures on Uphapee Creek Relief Bridge, Bulger Creek Bridge, and Uphapee Creek Bridge are given in Tables 5.7, 5.8, and 5.9 respectively. Tables 5.10 and 5.11 give the results of compressive strength tests performed by ALDOT during production of the HPC superstructures for the Uphapee Creek Relief Bridge and Bulger Creek Bridge. Table 5.12 gives similar results for the Uphapee Creek Bridge. Individual cylinder test results, measured air content and slump corresponding to each set of test cylinders are given in Appendix A, Tables A.10 through A.15. A summary of the average 7-day and 28-day compressive strength results for mixes HPC-2, HPC-3, and HPC-4 are given in Table 5.13. Also given in Table 5.13 is the standard deviation for each mix along with the number "n" of test sets used to determine the average and standard deviation. Each test set result is the average of two cylinder tests.

COMPARISON OF SPECIFIED OVERDESIGN TO ACI RECOMMENDATIONS

Approval of the CIP HPC was based on mixes conducted in the laboratory by Blue Circle Materials and samples obtained in the field during a test pour. The compressive strength originally specified for mix approval was 7800 psi which was based on the required 28-day strength of 6000 psi plus an overdesign of 1800 psi. Compressive strength results of two laboratory trials of the HPC-2 mix (shown in Table 5.2) are given in Table 5.14. This mix design was submitted for approval and approved for production of substructure HPC even though the specified overdesign was not met. The HPC-2 mix was refined to the HPC-3 mix in Table 5.3. The only difference between these two mix designs was the addition of a retarding admixture. Approval of the mix HPC-3 was based on strengths of test cylinders made at a test pour conducted at the jobsite. Those strength results are given in Table 5.14. Because these cylinder test specimens or the HPC-3 mix were obtained form a ready-mix truck, under production conditions, the overdesign of 1800 psi from the project specifications was relaxed. The mix design was refined further. The HPC-4 mix design given in Table 5.4 was approved for production of CIP HPC decks based on the proposed addition of cementitious materials and experience with similar mixes. A laboratory trial mix was not made for this mix design. Comparisons of the specified overdesign to those given by ACI 318-95 (1995) are discussed in the following. The overdesign recommended by ACI 318-95 (1995) Table 5.3.2.2 when no previous data of a particular mix is available to establish a standard deviation is 1400 psi as compared to 1800 psi required for this project. Using the data from production of the CIP HPC given in Table 5.13, a required overdesign compressive strength was calculated based on the standard deviation and use of Equations (5-1) and (5-2) from ACI 318-95 (1995). ACI requires the average compressive strength f_{cr}' used as the basis for selection of concrete proportions to be the larger of:

$$f_{cr}' = f_c' + 1.34s \tag{5.1}$$

or

$$f_{cr}' = f_c' + 2.33s - 500 \tag{5.2}$$

where f_c' is the required 28-day compressive strength and s is the standard deviation. Use of equation (5.1) and (5.2) are only allowed by ACI when 15 or more strength test results are available from prior production of the mix. The compressive strength overdesign based on data for mix HPC-3 is 470 psi from Equation (5.1) and 320 psi from Equation (5.2). Using the standard deviation shown in Table 5.13 for all the HPC-4 mix results give an overdesign from Equation (5.1) of 710 psi and the overdesign from Equation (5.2) is 730 psi. The required overdesign value of 1800 psi specified in this project may be overly conservative based on comparison to the value of 1400 psi from ACI 318-95 (1995). The overdesign based on strength results from production is significantly lower than required by project specifications and by ACI 318-95 (1995) Table 5.3.2.2. This value would be appropriate for use after several projects involving this mix design were successfully completed and rigorous QC/QA programs are used.

SUMMARY

The procedures used to place and finish the HPC-4 mix on the cast-in-place bridge decks in this project were not much different than if a conventional concrete mix design was used. Refinements in the mix design as well as the use of an evaporation reduction compound helped overcome concerns over the workability and finishability of the "sticky" mix. The slab test pour served as a good practice run for the Contractor to demonstrate his ability to produce a finished bridge deck with an acceptable appearance. These refinements along with the slab test pour helped the Contractor produce a bridge deck that had the same appearance as a bridge deck using conventional concrete, but with superior strength and durability properties.

The use of wet curing produced virtually crack free bridge decks on the Uphapee Creek Relief Bridge and Bulger Creek Bridge. Visual inspection of the bridge decks approximately 90 days after they were poured showed only a few very minor shrinkage cracks in isolated locations. These small surface cracks were likely a result of the deck not being kept fully saturated during the curing period. Overall, these bridge decks were quite acceptable. However, the decks of the Uphapee Creek bridge exhibited regularly spaced transverse deck cracks within 90 days of the completion of the deck pours. Reasons for the differences in performance between the individual bridges are unclear.

CHAPTER SIX

PERFORMANCE OF THE HPC GIRDER MIX

INTRODUCTION

A testing plan was developed to measure the performance characteristics of the HPC girder concrete. Properties determined for the HPC girder concrete are given in Table 6.1. Samples were obtained from multiple concrete pours during the production of the HPC girders. Descriptions of each testing procedure along with the results are given in following sections. All strength tests were performed in addition to tests conducted as required by project specifications. The properties of the HPC girder mix were then compared with the properties of a standard concrete girder mix under controlled final curing conditions. Direct comparison testing between the two mixtures was conducted to determine whether the HPC mix used in production of the HPC girders had better performance properties than a standard concrete girder mix. Results of those comparisons are given at the end of this chapter.

FRESH CONCRETE PROPERTIES

The fresh concrete properties shown in Table 6.1 were measured by ACI certified personal from Sherman Prestressed Concrete during each of the pours

Table 6.1. Properties of the HPC Girder Concrete

Fresh Properties	
Slump (ASTM C 143)	
Air Content (ASTM C 231)	
Unit Weight (ASTM C 138)	

Hardened Properties	
Compressive Strength (AASHTO T 22)	Release 7 Days 28 Days 56 Days
Tensile Strength (AASHTO T 198)	Release 28 Days 56 Days
Modulus of Elasticity (ASTM C 469)	Release 7 Days 28 Days 56 Days
Freeze-thaw Durability (AASHTO T 161)	300 Cycles
Chloride Ion Permeability (AASHTO T 277)	56 Days
Creep (ASTM C 512)	90 Days
Shrinkage (ASTM C 157)	90 Days

throughout production of the HPC girders. Air content and slump samples were obtained from the first batch of concrete leaving the batch plant and every 50 cubic yards thereafter. Typically two sets of air content and slump measurements were made per pour during the production of the HPC girders. Measured air and slump results from production of the HPC girders were previously presented in Chapter 4 and shown in Appendix A.9. The average unit weight of the concrete was 149.7 lbs/ft^3.

COMPRESSIVE STRENGTH

Compressive strength properties of the HPC girders were obtained according to AASHTO T 22 (1997) "Standard Specification for Compressive Strength of Cylindrical Concrete Specimens." Test samples were obtained from multiple pours during production of the HPC girders. All compressive strength tests were conducted at Auburn University using a pad-cap system on 70 durometer neoprene pads. A majority of the compressive strength data was obtained using 4 x 8 in. Sure Cure™ match-cured cylinders that were air cured along with the member on the casting yard after release. Other data collected included strength tests of: standard 4 x 8 in. cylinders made in plastic molds cured under the tarp along with the member and then cured on the casting yard, Sure Cure™ match cured cylinders subjected to final curing indoors or in a limebath, and standard 4 x 8 in. cylinders made in plastic molds and cured according to ASTM C 31 (1996). Cylinders ready for curing under the tarp along with the member are shown in Figure 6.1.

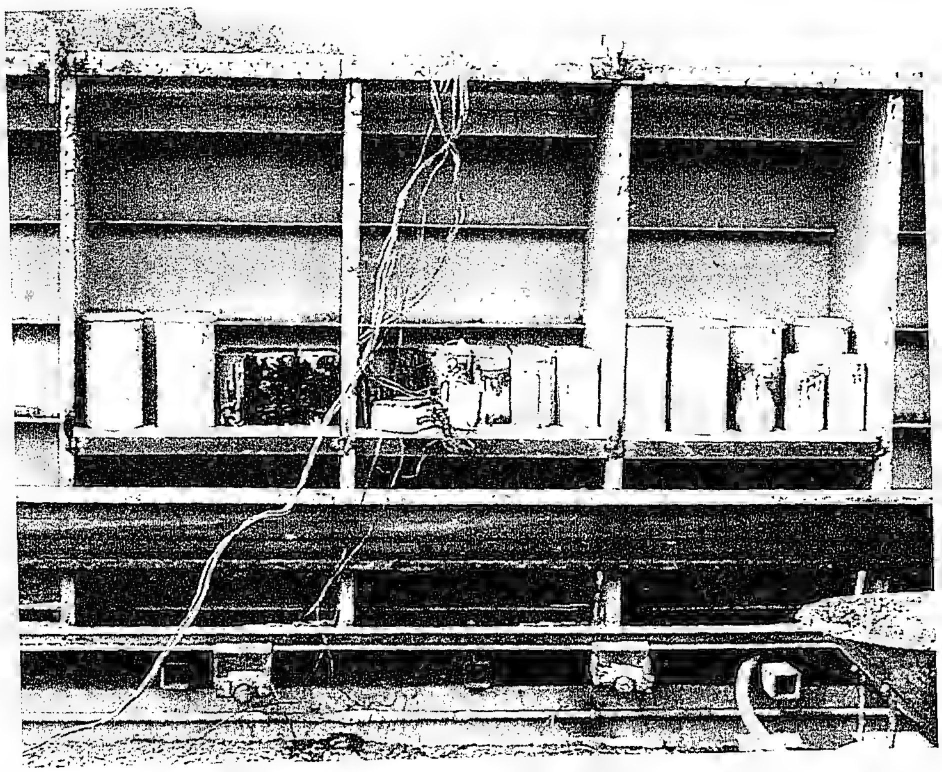


Figure 6.1. Cylinders Being Cured Along with the Member

Cylinders were tested at release, 7, 28, and 56 days to determine the compressive strength properties of the HPC girder mix. Measured compressive strengths ranged from 7760 psi at release to over 12,000 psi at 56 days. Compressive strength results for all the different initial and final curing conditions are given in Table 6.2. In Table 6.2, pour numbers 1 through 12 represent the mix design utilizing #67 limestone, and pours 13 through 18 represent the mix design utilizing #7 limestone. Discussion on the effects of initial and final curing condition on compressive strength properties are presented in Chapter 8.

TENSILE STRENGTH

Tensile strength properties of the HPC girders were obtained according to AASHTO T 198 (1997) "Standard Specification for Splitting Tensile Strength of Cylindrical Concrete Specimens." Tests were conducted on 4 x 8 in. Sure Cure™ match-cured cylinders and standard 4 x 8 in. cylinders made in plastic molds cured under the tarp along with the member. After release, final curing of the specimens was air curing either on the casting yard or indoors. Test samples were obtained from multiple pours during production of the HPC girders. Splitting tensile strength tests were conducted at release, 28, and 56 days at Auburn University. Measured splitting tensile strengths ranged from 590 psi at release to over 800 psi at 56 days. The results are given in Table 6.3. In Table 6.3, pour numbers 1 through 12 represent the mix design utilizing #67 limestone, and pours 13 through 18 represent the mix design utilizing #7 limestone. The average 56-day splitting tensile strength of the 4 x 8 in. Sure Cure™ match-cured

Table 6.2. Compressive Strength Test Results from Production Pouts of the HPC Girders Utilizing Various Initial and Final Curing Conditions

Pour Number	Curing Method[2]	Compressive Strength (psi)[1]			
		Release	7-Day	28-Day	56-Day
1	(1)	8940	9780	10050	11280
1	(4)	8100	9130	9750	10100
1	(5)	-	-	10130	11020
2	(4)	8080	8680	9270	-
3	(1)	-	-	-	10750
3	(4)	8320	9530	10600	12290
4	(4)	-	8690	10300	10810
5	(1)	-	-	11000	10850
5	(4)	7060	9470	10460	10240
6	(1)	-	-	-	9270
6	(4)	7760	8870	10520	10410
7	(4)	7720	9030	-	-
8	(2)	8950	-	10480	10150
8	(4)	7480	8910	9890	10360
9	(1)	-	-	-	9990
9	(4)	7520	8790	9470	10540
10	(1)	-	-	11090	10350
10	(4)	8200	9250	9910	10380
11	(1)	-	-	10550	10250
11	(2)	-	-	10410	10230
11	(3)	-	-	10650	-
11	(4)	-	9390	9930	-
12	(1)	-	-	9950	10150
12	(3)	-	-	10050	11240
12	(4)	-	8780	8990	-
13	(1)	-	-	11270	11350
13	(4)	8990	10130	10960	11580
14	(1)	-	-	10760	11060
14	(3)	-	-	12070	12640
14	(4)	8870	10180	10920	11850
15	(1)	-	11110	12300	12230
15	(4)	8730	9810	11600	11620
16	(1)	-	-	11620	11540
16	(2)	-	-	10630	11540
16	(3)	-	-	13030	13650
16	(4)	8440	9770	11380	11400
17	(1)	-	12280	11880	12190
17	(4)	9390	10300	11080	11540
18	(1)	9950	-	11320	12370
18	(4)	9750	10920	11290	12010

1. All cylinder tests performed at Auburn University using 70 durometer neoprene pads. Refer to Appendix A, Tables A.14 through A.16 for individual cylinder test results.
2. (1) - 4 x 8 in. Sure Cure™ cylinders cured on the casting yard.
(2) - 4 x 8 in. Sure Cure™ cylinders cured indoors.
(3) - 4 x 8 in. Sure Cure™ cylinders cured in a limebath.
(4) - 4 x 8 in. standard cylinders made in plastic molds and cured under the tarp then cured on the casting yard.
(5) - 4 x 8 in. cylinders cured according to ASTM C 31.

cylinders for pours 1 through 12 was 650 psi, and the average for pours 13 through 18 was 725 psi.

The ACI Committee 363 (1992) "State-of-the-Art Report on High Strength Concrete" recommends the following equation for estimating the splitting tensile strength when the compressive strength is between 3000 and 12,000 psi :

$$f_{sp}' = 7.4 \sqrt{f_c'} \text{ (psi)} \tag{6.1}$$

AASHTO (1996) limits the tensile stress in the precompressed tensile zone of a prestressed concrete girder to the following:

$$f_t = 6\sqrt{f_c'} \text{ (psi)} \tag{6.2}$$

Comparison of the splitting tensile strength results given in Table 6.3 in which a corresponding compressive strength result was obtained are compared to the values from the ACI 363 (1992) and AASHTO (1996) equations. These comparisons are given in Figure 6.2. A majority of the split cylinder tensile strength results fell between the values from Equation 6.1 and 6.2 as shown in Figure 6.2. The AASHTO (1996) tensile stress limit provides a reasonable lower bound to the measured tensile strength.

Research conducted at the University of Texas (1996) found that both the ACI 363 (1992) and AASHTO (1996) equations significantly underestimate the tensile strength of the concrete containing fly ash at 56 days. That research indicated that HPC could exhibit a very high flexural strength to compressive strength ratio, much higher than what is normally assumed for normal strength concrete. The flexural strength of concrete is dependent upon the shape and texture of the coarse aggregate used, and the results in the Figure 6.2 show that

Table 6.3. Splitting Tensile Strength Results from Production of the HPC Girders Utilizing Various Initial and Final Curing Conditions

Pour Number	Curing Method[2]	Tensile Strength (psi)[1]		
		Release	28-Day	56-Day
1	(1)	590	650	-
3	(1)	-	-	700
4	(3)	-	-	800
5	(1)	-	-	700
6	(1)	-	-	660
8	(2)	650	700	740
9	(1)	-	-	550
9	(1)	-	-	600
10	(1)	-	740	600
10	(3)	-	650	600
13	(1)	-	600	650
13	(1)	-	-	700
14	(1)	-	-	650
15	(1)	600	700	700
15	(1)	-	-	650
16	(1)	-	650	840
17	(1)	-	600	-
17	(1)	640	700	800
18	(1)	-	720	810

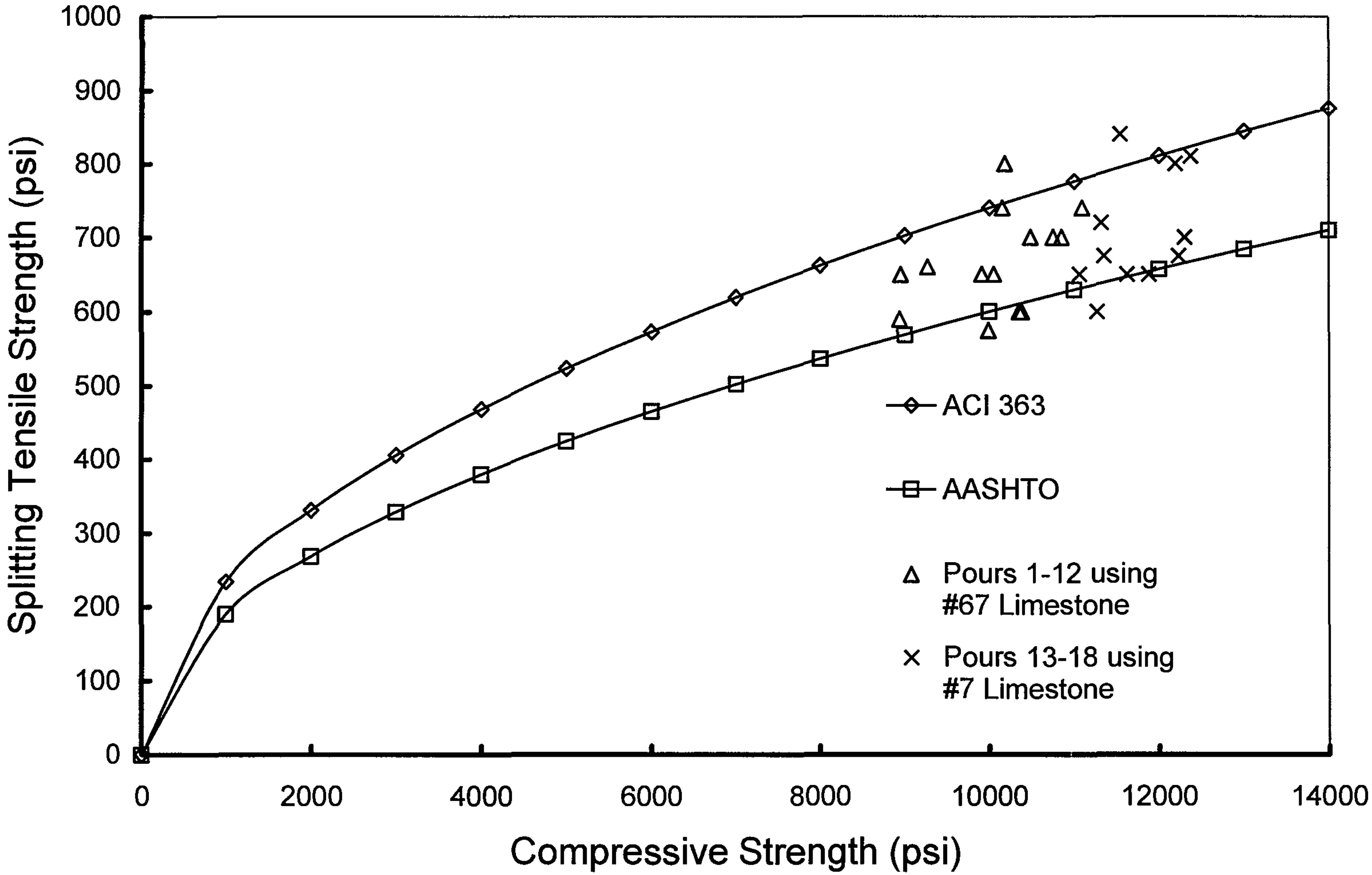


Figure 6.2. Comparison of Splitting Tensile Strengths from All Production Pours to ACI 363 and AASHTO Equations

Table 6.4. Modulus of Elasticity Results from Production of the HPC Girders

Pour Number	Curing Method[2]	Modulus of Elasticity (psi)[1]			
		Release	7-Day	28-Day	56-Day
1	(1)	5,900,000	6,600,000	6,000,000	5,900,000
3	(1)	-	-	-	6,000,000
4	(1)	-	-	-	4,300,000
4	(3)	-	-	-	6,150,000
5	(1)	-	-	-	4,400,000
6	(1)	-	-	-	5,350,000
8	(1)	-	-	5,600,000	5,600,000
9	(2)	5,400,000	5,600,000	5,450,000	5,900,000
10	(1)	-	-	5,300,000	5,400,000
10	(3)	-	-	5,500,000	5,050,000
13	(1)	-	-	5,200,000	6,000,000
14	(1)	-	-	-	5,800,000
15	(1)	5,250,000	-	5,800,000	5,300,000
16	(1)	-	-	6,300,000	5,800,000
17	(1)	5,300,000	-	7,100,000	7,000,000
18	(1)	-	-	6,500,000	6,900,000

1. Results are for one cylinder test performed at Auburn University.
2. (1) - 4 x 8 in. Sure Cure™ cylinders cured on the casting yard.
   (2) - 4 x 8 in. Sure Cure™ cylinders cured indoors.
   (3) - 4 x 8 in. standard plastic molded cylinders cured under the tarp and cured on the casting yard.

the use of an increased allowable tensile stress in girder design should be examined for each particular case.

MODULUS OF ELASTICITY

Modulus of elasticity of the HPC girder concrete was measured according to ASTM C 469 (1996) "Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression." Figure 6.3 shows the setup for measuring the modulus of elasticity. Tests were conducted on 4 x 8 in. Sure Cure™ match-cured cylinders and 4 x 8 in. plastic molded cylinders cured under the tarp along with the member. Test samples were obtained from multiple pours during production of the HPC girders. The specimens were air cured on the casting yard along with the member or indoors after release.

Modulus of elasticity was measured at release, 7, 28, and 56 days. Results ranged from 5,250,000 psi at release to over 7,000,000 psi at later ages. Results are given in Table 6.4. In Table 6.4, pour numbers 1 through 12 represent the mix design utilizing #67 limestone, and pours 13 through 18 represent the mix design utilizing #7 limestone.

An empirical relationship given by the ACI 318, Section 8.5 (1995) states:

$$E_c = 33\, w^{1.5} \sqrt{f_c'} \quad \text{(psi)} \qquad (6.3)$$

where w = true unit weight of the mix in pounds per cubic foot. The ACI Committee 363 (1992) "State-of-the-Art Report on High Strength Concrete"

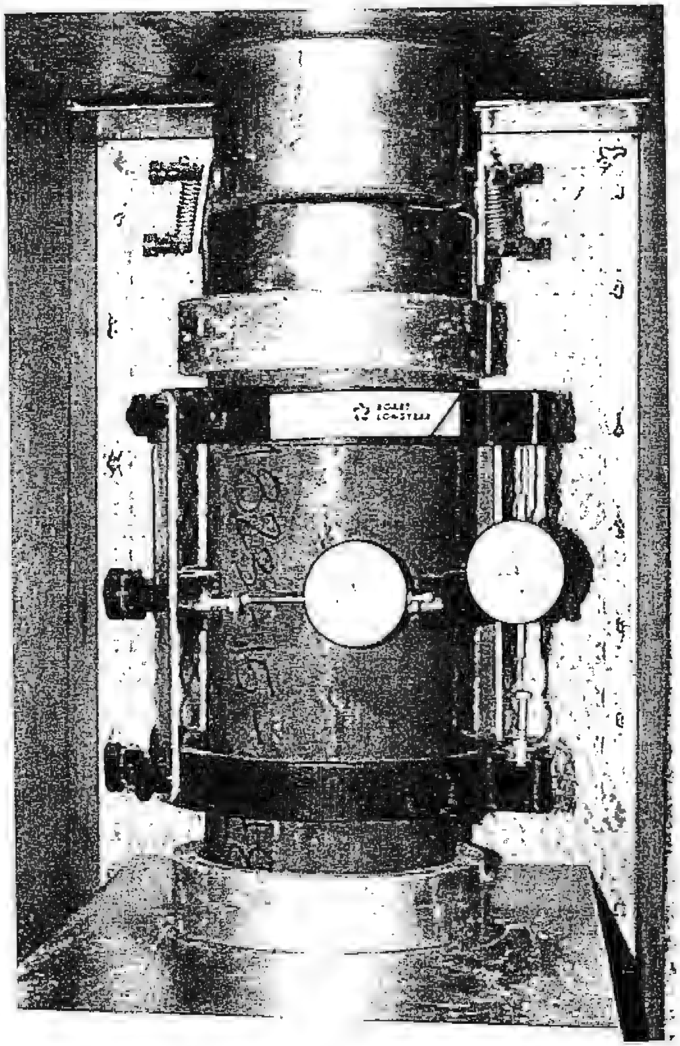


Figure 6.3. Modulus of Elasticity Test Setup

recommends the following equation for compressive strengths between 3000 and 12,000 psi:

$$E_c = 40,000 \sqrt{f_c'} + 1.0 \times 10^6 \text{ (psi)} \qquad (6.4)$$

A comparison of values from these equations to values measured during production of the HPC girders is shown in Figures 6.4. The concrete unit weight used in Equation 6.3 was 149.7 lbs/ft^3. A majority of the test data fell between the two equations as shown in Figure 6.4.

FREEZE-THAW DURABILITY

Freeze-Thaw durability properties of the HPC girders were measured according to AASHTO T 161 (1997) "Standard Specification for Resistance of Concrete to Rapid Freezing and Thawing." Samples were 3 x 4 x 16 in. cast in steel molds and initially cured according to ASTM C 31 (1996). Samples were stripped at release and cured in a limebath for 14 days. At 14 days, the specimen's initial fundamental transverse frequency was recorded using a sonometer, then the specimen was put into the freeze-thaw cabinet for repeated freezing and thawing cycles. The sonometer and freeze-thaw apparatus are shown in Figure 6.5. The test was conducted according to Procedure A from AASHTO T 161 (1997) in which the specimens were surrounded in 1/32 in. to 1/8 in. of water throughout the freeze-thaw cycle. The specimens went through a complete freeze-thaw cycle every 3 to 4 hours. A freeze-thaw cycle consists of lowering the temperature of the specimen from 40°F to 0°F and raising it from 0°F to 40°F. Samples were tested approximately every 30 to 36 cycles in the

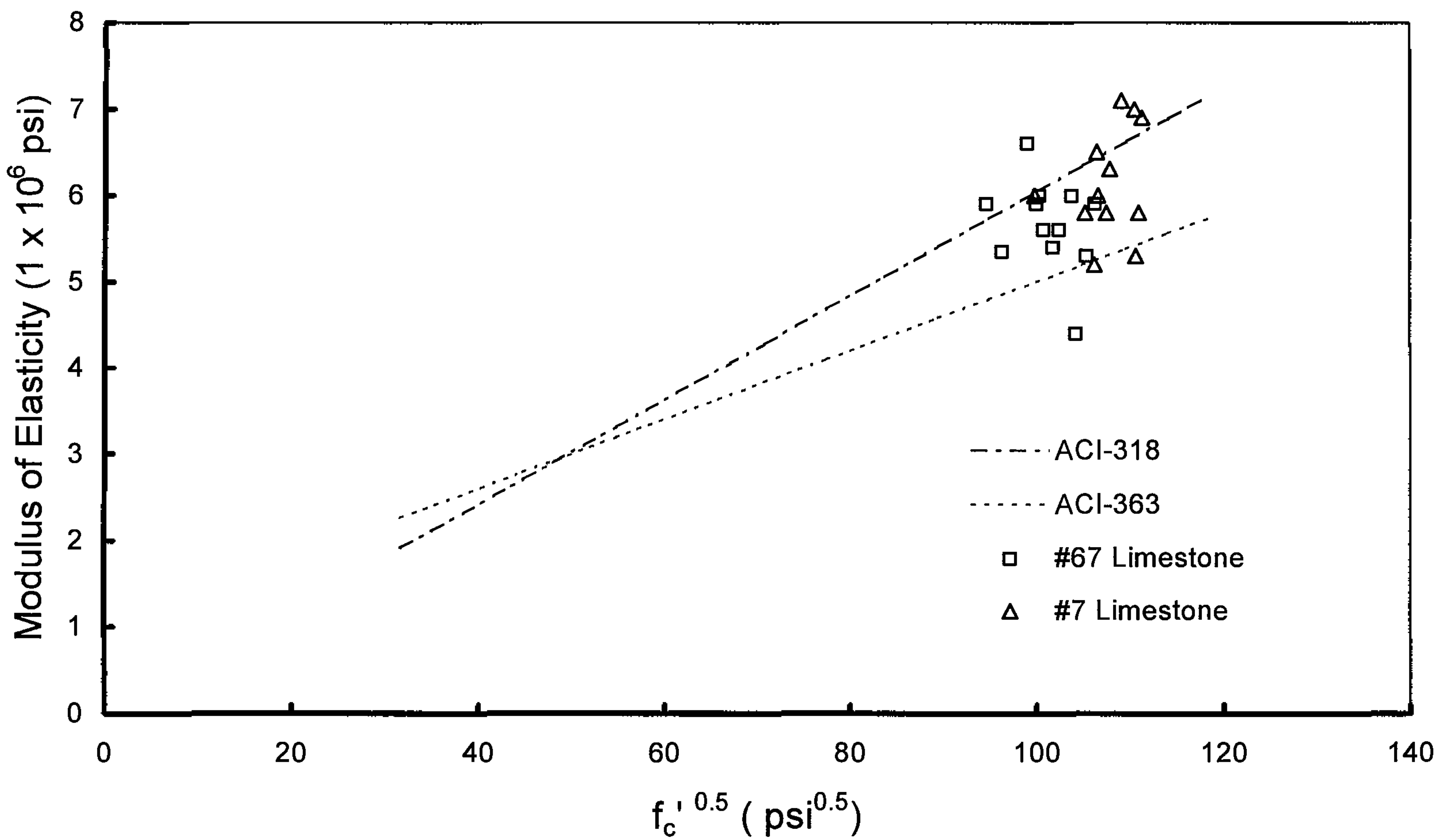
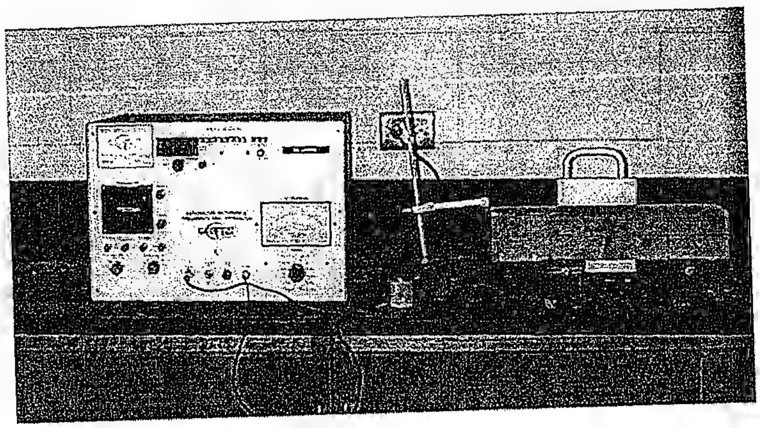


Figure 6.4. Modulus of Elasticity from 4 x 8 in. Sure Cure™ Cylinders



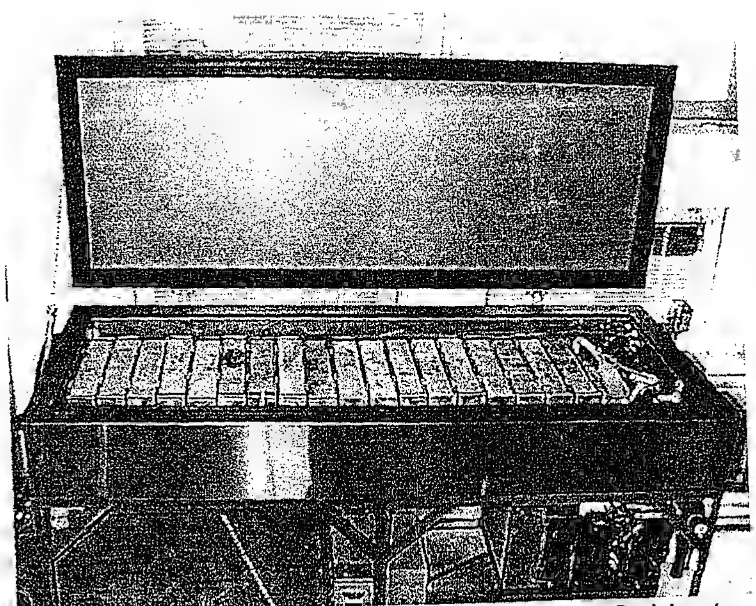


Figure 6.5. Sonometer and Freeze-Thaw Cabinet

thawed state for fundamental transverse frequency. Tests were conducted for 300 cycles. At 300 cycles, the samples were removed from the freeze-thaw cabinet and tested for the final fundamental transverse frequency. Resistance to freeze-thaw is given by the Relative Dynamic Modulus of Elasticity, or Durability Factor (DF). The equation used to calculate the DF is shown below where n is the fundamental transverse frequency (Hz) at 0 cycles and n_1 is the fundamental transverse frequency (Hz) at 300 cycles.

$$DF = (n_1^2) / (n^2) \text{ X } 100 \qquad (6.5)$$

Results of the freeze-thaw testing are given in Table 6.5. Plots of Durability Factor versus Number of Cycles for each test specimen are given in Figures 6.6 through 6.8. The average Durability Factor of the three pours tested was 97.9%. In the definition that Goodspeed et al. (1996) developed for HPC in highway structures, different grades of performance characteristics for high performance structural concrete were defined. Concrete with a Durability Factor above 80% after 300 cycles receives the highest performance grade. The guidelines set forth by Goodspeed et al. (1996) identifies a set of concrete performance characteristics sufficient to estimate long-term concrete durability for highway structures. The reason for the high measured Durability Factor for the girder concrete was the proper use of air entrainment and pozzolans in the concrete mix design.

Table 6.5. Freeze-thaw Durability Results from Production of the HPC Girders

Pour Number	Initial Frequency (Hz)	Final Frequency (Hz)	Durability Factor (%)
1	2500	2450	96.0
1	2460	2450	99.2
1	2470	2440	97.6
			Average = 97.6
10	2530	2510	98.4
10	2530	2490	96.9
10	2540	2520	98.4
			Average = 97.9
16	2580	2560	98.5
16	2590	2560	97.7
16	2580	2560	98.5
			Average = 98.2

Tests were conducted for 300 freeze-thaw cycles with three samples per mix.

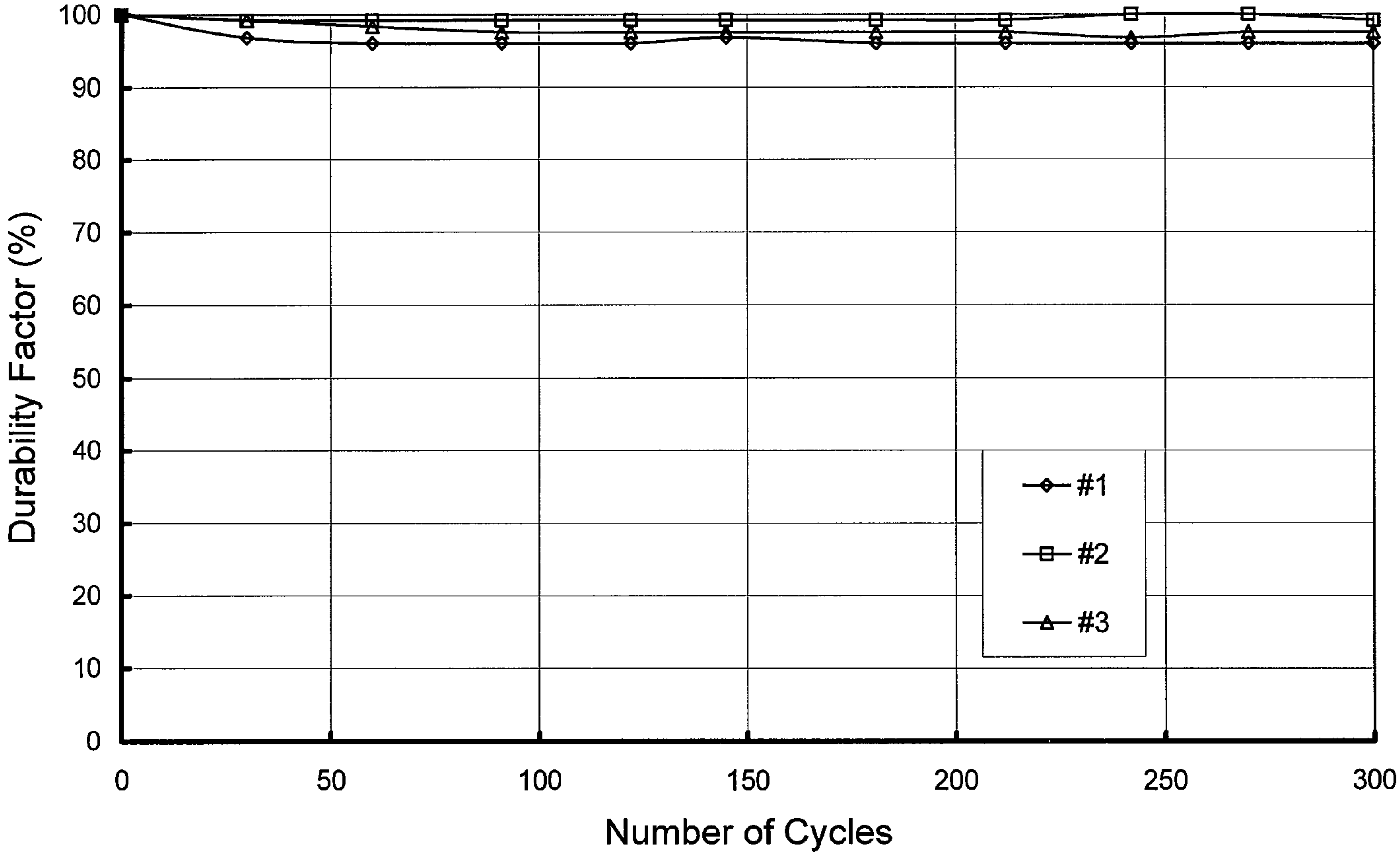


Figure 6.6. Durability Factor for 300 Cycles for Pour 1

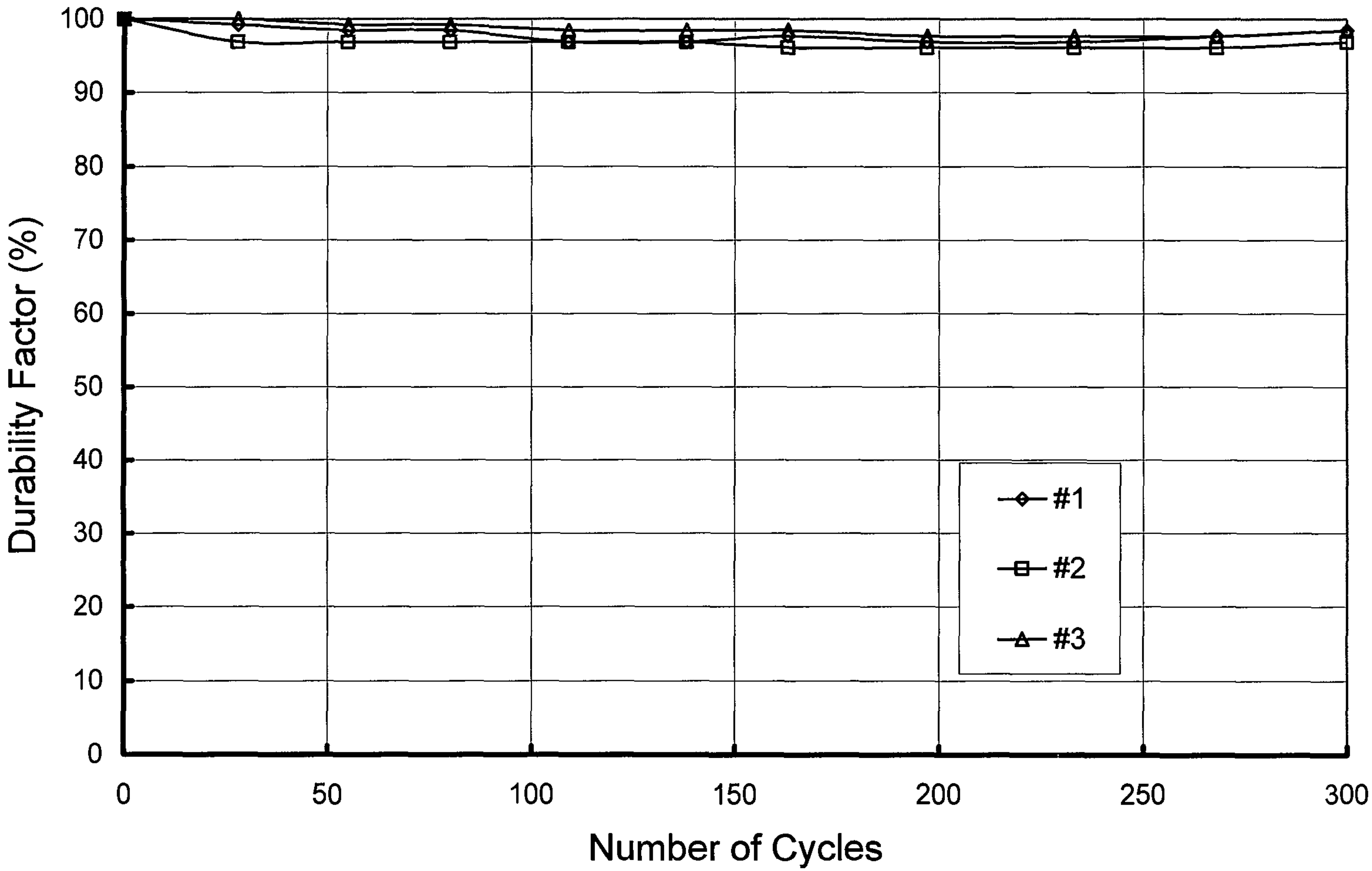


Figure 6.7. Durability Factor for 300 Cycles for Pour 10

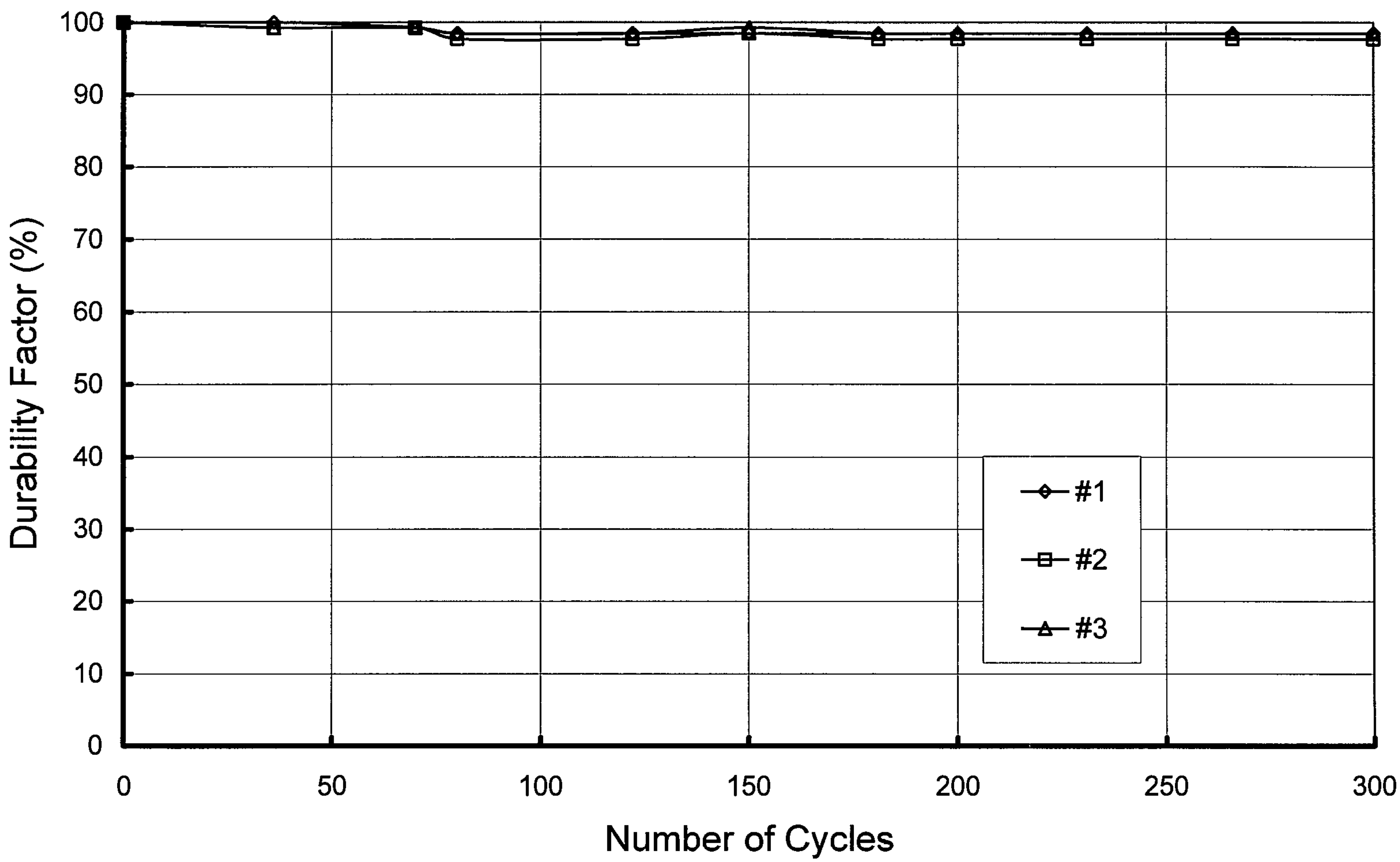


Figure 6.8. Durability Factor for 300 Cycles for Pour 16

CHLORIDE ION PERMEABILITY

Permeability properties of the HPC girders was measured according to AASHTO T 277 (1997) "Standard Specification for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration." Conducting the chloride ion test consists of monitoring the amount of electrical current that passes thorough a 2 in. thick slice of a 4 in. nominal diameter specimen during a 6 hour period. A 60 Volt current is maintained across the ends of the specimen, one of which is immersed in a sodium chloride solution, the other in a sodium hydroxide solution. The total charge passed is measured in coulombs. The amount of coulombs passed during the 6 hour long test has been found to be related to the resistance of the specimen to chloride ion penetration. A brief summary of the testing procedure is given below:

1. Using a water-cooled diamond saw, cut a 2 ± 0.125 in. slice from a 4 x 8 in. test cylinder.
2. Apply a rapid setting epoxy coating to side surface of the specimen and allow specimen to cure for approximately 18 hours.
3. Place specimen in sealed vacuum desiccator with both ends of the specimen exposed and run vacuum at 1 mm Hg for 3 hours.
4. With vacuum still running, drain de-aerated water into desiccator fully submerging the specimen and run vacuum for an additional hour.
5. Stop vacuum and allow air to reenter the desiccator.
6. Soak specimen under water for 18 ± 2 hours.

7. Remove specimen from water, and place on voltage cell. Apply sealant to achieve a seal between the specimen and voltage cell. Repeat process for other voltage cell. Allow specimen to cure according to manufacturers specifications without allowing the loss of moisture to occur from the voltage cell.
8. Fill one side of the voltage cell with the 3.0 percent NaCl solution and connect the negative terminal of the power supply. Fill the other side with 0.3 normal NaOH solution and connect to the positive terminal of the power supply.
9. Turn power supply on, set to 60.0 ± 0.1 V, and record initial current reading. Record current every 30 minutes. Terminate test after 6 hours.

An automatic data processing machine was used to continuously display the coulomb value throughout the test. This apparatus is shown in Figure 6.9 along with the voltage cells and specimens.

Tests were conducted at 56 days on 4 x 8 in. Sure Cure™ match-cured cylinders and standard 4 x 8 in. cylinders made in plastic molds cured under the tarp along with the member. Final curing of the specimens was either on the casting yard or indoors. Also, a set of cylinders made and cured according to ASTM C 31 (1996) were tested. Test samples were obtained from multiple pours during production of the HPC girders and tested at Auburn University. Values obtained for the total charge passed are based upon test samples with diameters

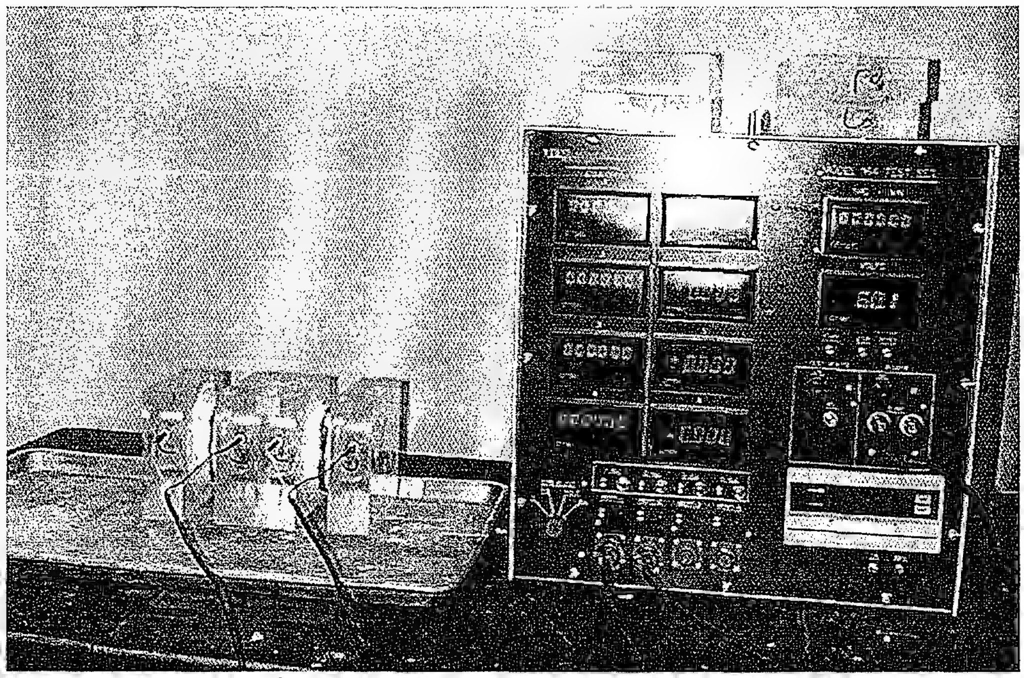


Figure 6.9. Rapid Chloride Ion Test Unit

of 3.75 inches. For specimen diameters other than 3.75 in., the value for total charge passed is adjusted by the following:

$$Q_{FINAL} = Q_x \times (3.75 / x)^2 \qquad (6.6)$$

where Q_{FINAL} = charge passed in coulombs through a 3.75 in. diameter specimen, x = diameter of the nonstandard specimen, and Q_x = charge passed in coulombs through x diameter specimen. The diameter of the specimens tested in this research where approximately 4.0 in. and where adjusted using Equation 6.6. A summary of the test results are given in Table 6.6. Coulomb values ranged from 1920 to over 5700. The high coulomb values obtained from pour 8 resulted from Sure Cure™ match-cured cylinders that were cured at temperatures in excess of 200 degrees Fahrenheit due to an equipment malfunction. A majority of the test samples had coulomb values between 2000 and 4000 which is classified as moderate permeability by AASHTO T 277 (1997) standards. The HPC girder mixes that were tested contained 15% fly ash. These moderate permeability values appear reasonable considering the amount of pozzolan that was incorporated into the mix was not high.

Excluding the erratic results of the Sure Cure™ match-cured cylinders in pour 8, a comparison between the permeability of Sure Cure™ match-cured cylinders and cylinders cured under the tarp along with the member yielded similar results. The average permeability at 56 days for all the Sure Cure™ match-cured cylinders was 2460 coulombs as compared to 2520 coulombs for cylinders cured under the tarp along with the member. Also, a slight improvement in the permeability characteristics was observed for the Sure

Table 6.6. Rapid Chloride Ion Penetration Results from Production of the HPC Girders

Pour Number	Curing Method[2]	Charge Passed (Coulombs)	AASHTO T 277 Rating[1]
1	(1)	2140	Moderate
1	(1)	2870	Moderate
1	(4)	1920	Low
1	(4)	2280	Moderate
5	(1)	3130	Moderate
5	(3)	2290	Moderate
8	(1)	5610	High
8	(1)	5730	High
8	(3)	2710	Moderate
8	(3)	2720	Moderate
10	(2)	2530	Moderate
10	(2)	2600	Moderate
10	(3)	2690	Moderate
10	(3)	2730	Moderate
13	(1)	2220	Moderate
13	(1)	2120	Moderate
13	(3)	2470	Moderate
13	(3)	2530	Moderate
15	(1)	2400	Moderate
15	(1)	2210	Moderate
15	(3)	2340	Moderate
15	(3)	2370	Moderate
17	(1)	2340	Moderate
17	(3)	2370	Moderate

1. Refer to Table 1 in AASHTO T 277-93
2. (1) - 4 x 8 in. Sure Cure™ cylinder cured on the casting yard.
   (2) - 4 x 8 in. Sure Cure™ cylinders cured indoors.
   (3) - 4 x 8 in. Standard plastic molded cylinders cured under the tarp and cured on the casting yard.
   (4) - 4 x 8 in. cylinders cured according to ASTM C 31.

Cure™ match-cured cylinders of pours 13 through 18 in which a #7 limestone coarse aggregate was used instead of a #67 limestone coarse aggregate. The average permeability of the Sure Cure™ match-cured cylinders of pours 1 through 12 was 2650 coulombs as compared to 2260 for pours 13 through 18.

CREEP

Creep properties of the HPC girders were measured according to the procedure described in ASTM C 512 (1996) "Standard Test Method for Creep of Concrete in Compression." The only modification to this procedure was the test specimens were loaded at 24 hours instead of the prescribed 28 days. This was done to more realistically model the loading condition of the prestressed girders. This choice was made because the creep data is also used in a companion investigation of camber and prestress losses for the HPC girders.

A suitable apparatus was constructed in which to load the test specimens. The creep frame used is shown in Figure 6.10. The sequence used to conduct the creep test is discussed in the following. Test specimens were 4 x 8 in. Sure Cure™ match-cured cylinders which were brought to the laboratory at Auburn University from the prestress yard at the time of release of the prestressing force. Test samples were capped with a high strength sulfur capping compound to ensure an even transfer of load between the specimens and the steel loading saddles as well as between each test specimen. Companion cylinders were broken at the time of loading to give an accurate indication of the compressive strength of the concrete. Four specimens were placed in the creep frame and loaded through the use of a hydraulic ram to approximately 45% of the measured

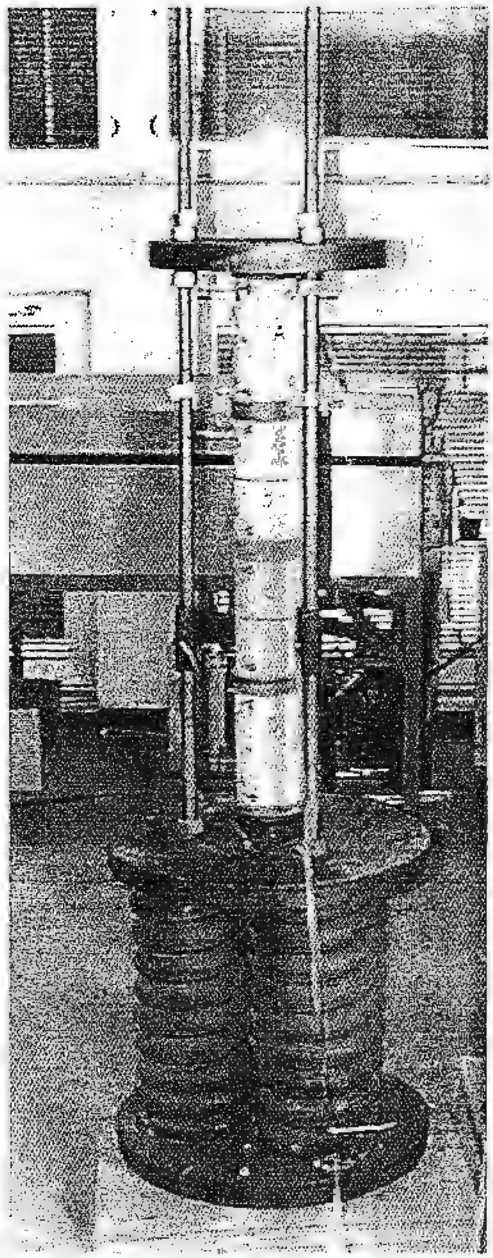


Figure 6.10. Creep Frame Used in Testing the HPC Girder Mix

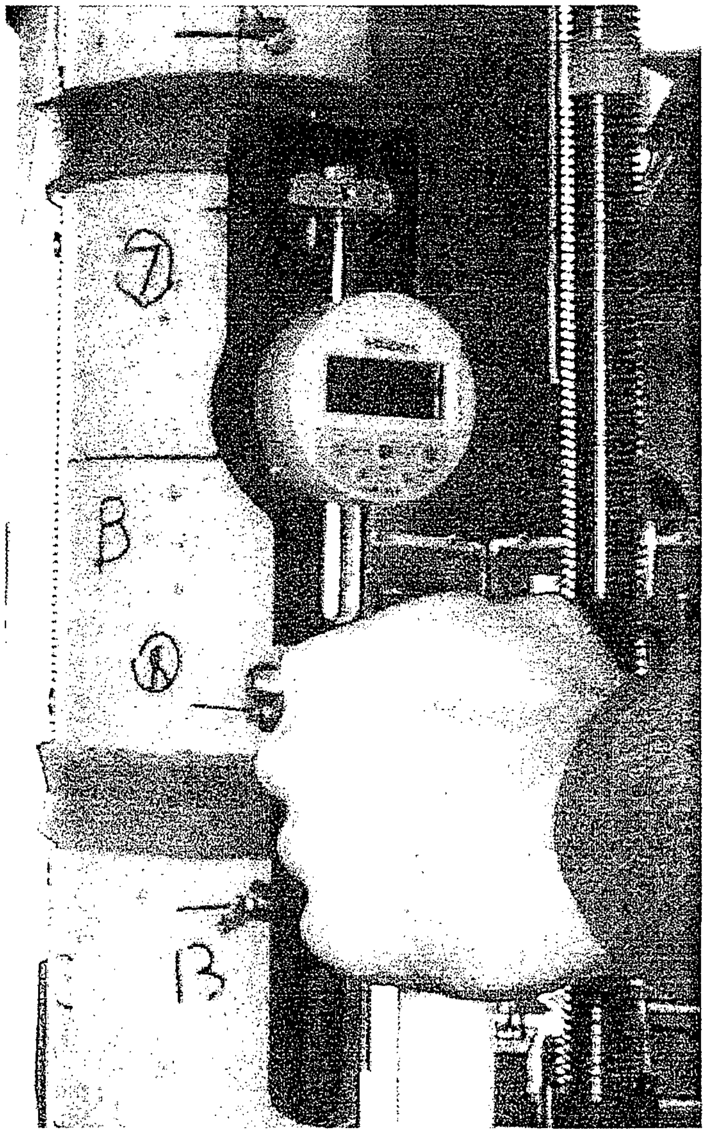


Figure 6.11. Demec Gauge Used to Measure Creep Specimens

compressive strength of the concrete. The load was sustained by three springs that were situated between the bottom two plates. As the specimen's length decreased due to creep and shrinkage effects, the load was maintained by the elongation of the springs. The amount of load lost was determined from strain gages mounted on the treaded rods in the longitudinal direction. The load was maintained between 44.5% and 43.9% of the initial compressive strength of the concrete.

Strain measurements were made using a Demec mechanical strain gage. The Demec mechanical strain gage used is shown in Figure 6.11. To use the Demec mechanical strain gage, a series of 2 measuring points were epoxyed on each test specimen at 120-degree intervals. Each set of points had approximately a 6 in. gage length and were placed parallel to the longitudinal axis at each of the 120-degree intervals so that three readings could be taken from each side of all the specimens. Figure 6.12 shows the Demec point layout on the creep specimens.

Creep results presented in this research are specific creep values. Specific creep is defined as the creep strain divided by the applied stress on the specimen. The creep strain is the total strain minus the initial elastic strain due to the applied load, the drying shrinkage strain, and thermal strains. Specific creep results for a loading period up to 98 days are given in Table 6.7. The measured specific creep at 90 days of loading under approximately 44% of the initial compressive strength was 0.20 microstrain/psi. A plot of specific creep versus time is shown in Figure 6.13.

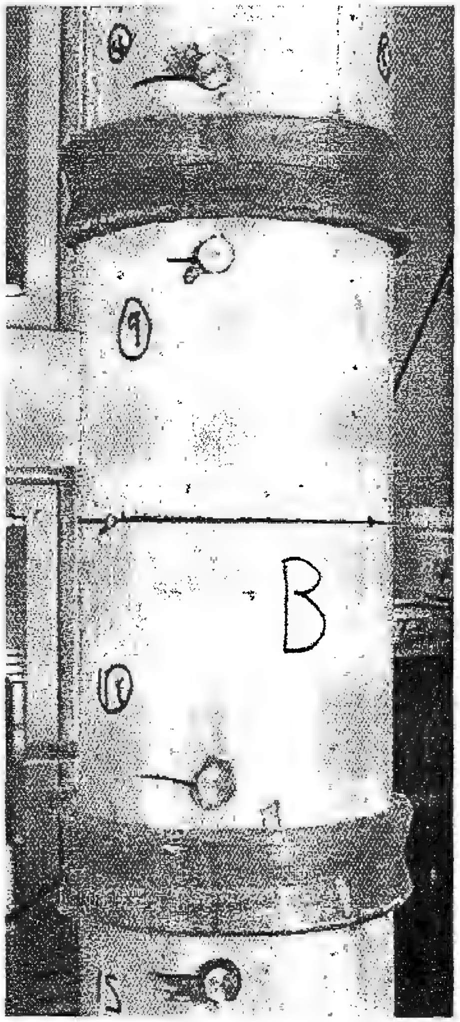


Figure 6.12. Demec Points on Creep Specimen

Table 6.7. Specific Creep of HPC Girder Mix

Age (Days)	Specific Creep (Microstrain/psi)
Immediately before loading	0
Immediately after loading	0
1	0.08
2	0.11
3	0.10
4	0.13
5	0.05
6	0.08
7	0.09
16	0.08
22	0.14
30	0.16
62	0.20
98	0.20

Creep specimens loaded to approximately 44% of the measured compressive strength at approximately 24 hours. The specific creep values shown take into account the effects of the initial elastic strain due to the applied load, drying shrinkage, and thermal effects.

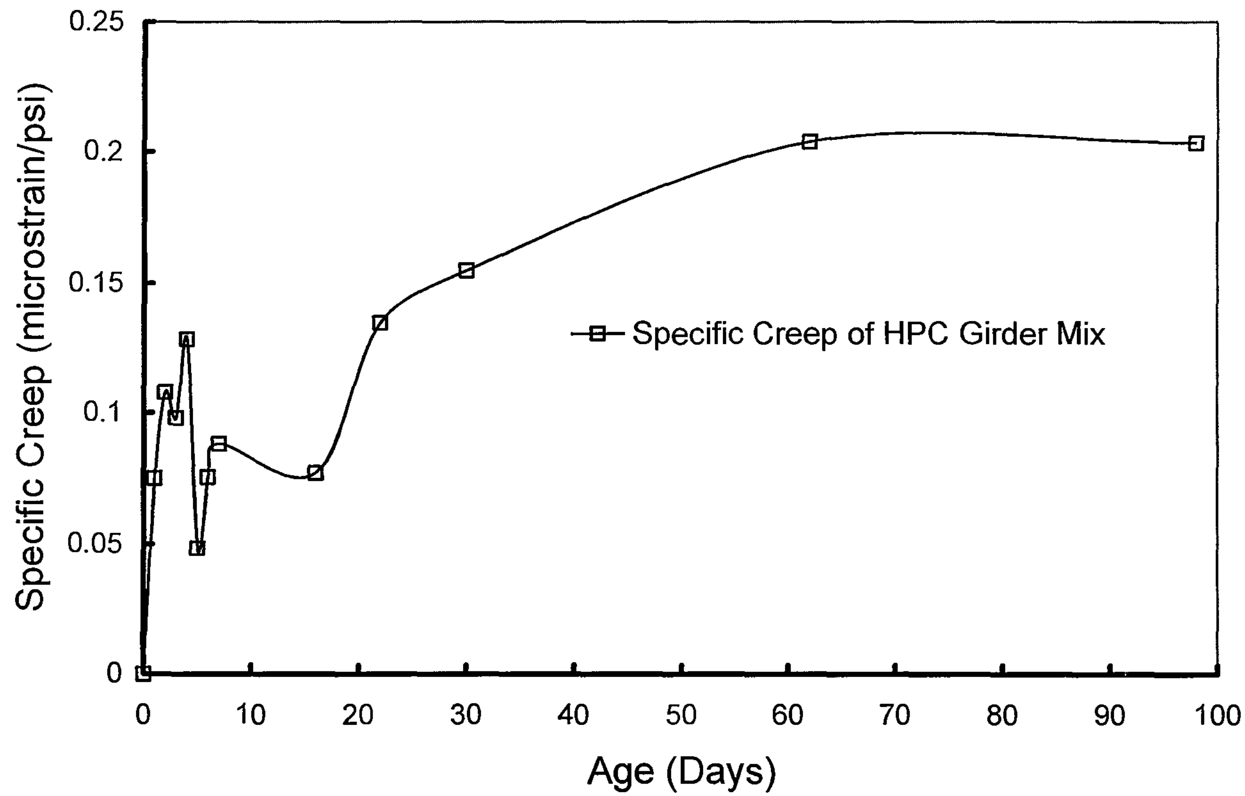


Figure 6.13. Specific Creep up to 90 Days for the HPC Girder Mix Loaded to Approximately 44% of f_{ci}'

In the definition that Goodspeed et al. (1996) developed for HPC in highway structures, grades of performance characteristics for high performance structural concrete are defined. Concrete with specific creep values below 0.21/psi are given the highest performance grade rating, grade 4.

Concrete with specific creep values between 0.21/psi and 0.31/psi are given the next to highest grade, grade 3. These grade definitions are based on the specific creep at 180 days for specimen loaded at an age of 28-days. The specific creep results shown in Figure 6.13 are for specimens loaded at an age of 1 day which is more severe. Based on the results presented in Figure 6.13 it is reasonable to conclude that the HPC girder concrete would easily met the requirements for grade 3 and probably for grade 4.

A report by ACI Committee 209 (1997) recommends predicting a creep coefficient in steam-cured concrete subjected to air drying by the following:

$$v_t = (t^{0.60} \ / \ 10 + t^{0.60}) * v_u \qquad (6.7)$$

where $v_u = 2.35 * \gamma_c$

The variable γ_c represents the product of applicable correction factors. This correction factor is a function of the ambient relative humidity, average thickness of the member, measured slump, fine aggregate percentage, and air content of the concrete. The creep coefficient at 90 days predicted by Equation 6.7 and is 1.69. The specific creep is found by the following equation:

$$\delta_t = v_t \ / \ E_{ci} \qquad (6.8)$$

where v_t was given in Equation 6.7, and E_{ci} is the initial modulus of elasticity of the concrete which is known to be 5,900,000 psi. Therefore, from equation 6.8

the predicted specific creep after 90 days of loading is 0.29 microstrain/psi as compared to 0.20 microstrain/psi found through experimentation. The fact that a higher value is obtained from Equation 6.7 seems reasonable since general equations of this type often produce conservative results.

SHRINKAGE

Shrinkage properties of the HPC girder concrete were obtained according to ASTM C 157 (1996) "Standard Specification for Length Change of Hardened Hydraulic-Cement Mortar and Concrete" as well as by a procedure previously used by Farrington et al. (1996).

For the ASTM C 157 procedure, samples were cast in 3 x 3 x 12 in. steel molds and initially cured according to ASTM C 31 (1996). Specimens were stripped at the time of release of the girders and the specimen lengths were measured. The apparatus used to measure the length of the specimens is shown in Figure 6.14. One shrinkage sample was then air dried for a period of 90 days, while another was moist cured for 28 days, then air dried for a period up to 90 days after release. Specimens were measured for change in length at various times during the drying period. Drying shrinkage at 90 days was 280 microstrain for the specimen immediately air dried, and 140 microstrain for the specimen moist cured for 28 days, then air dried. A plot of the shrinkage versus time up to 90 days is given for both samples in Figure 6.15.

The procedure used by Farrington et al. (1996) to obtain shrinkage values used specimens in the same manner as those used in creep testing with Demec points affixed to each specimen. For this research, the specimens were 4 x 8 in.

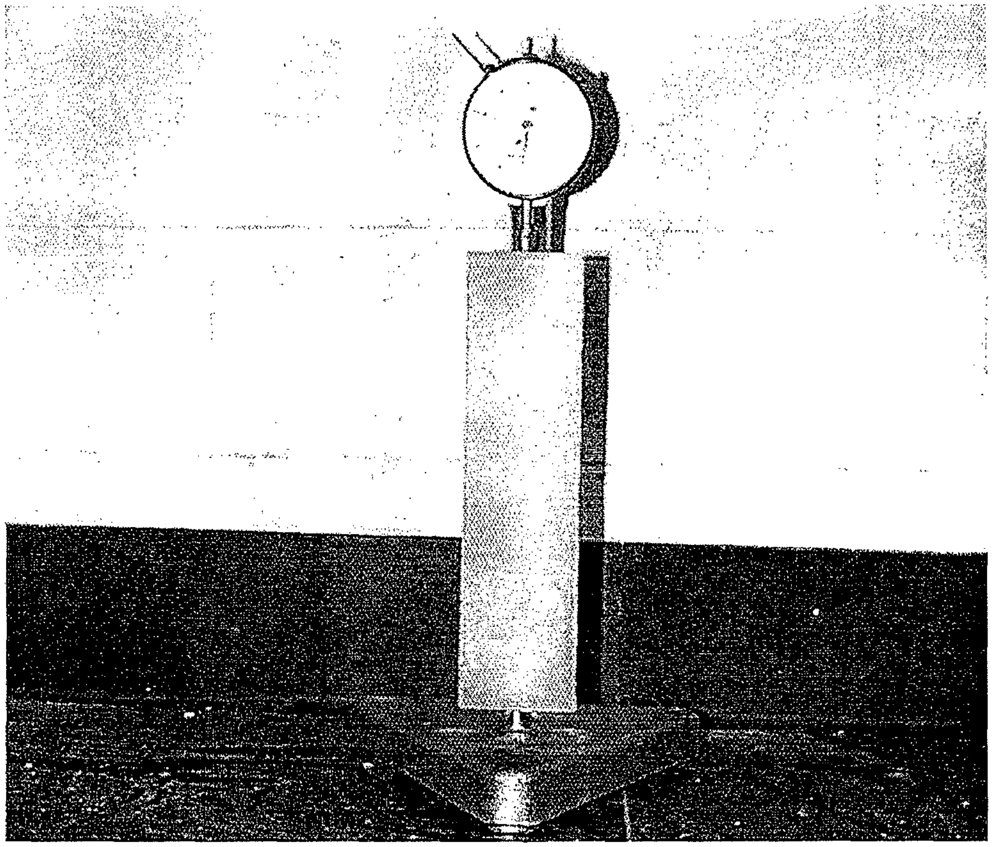


Figure 6.14. Shrinkage Test Apparatus

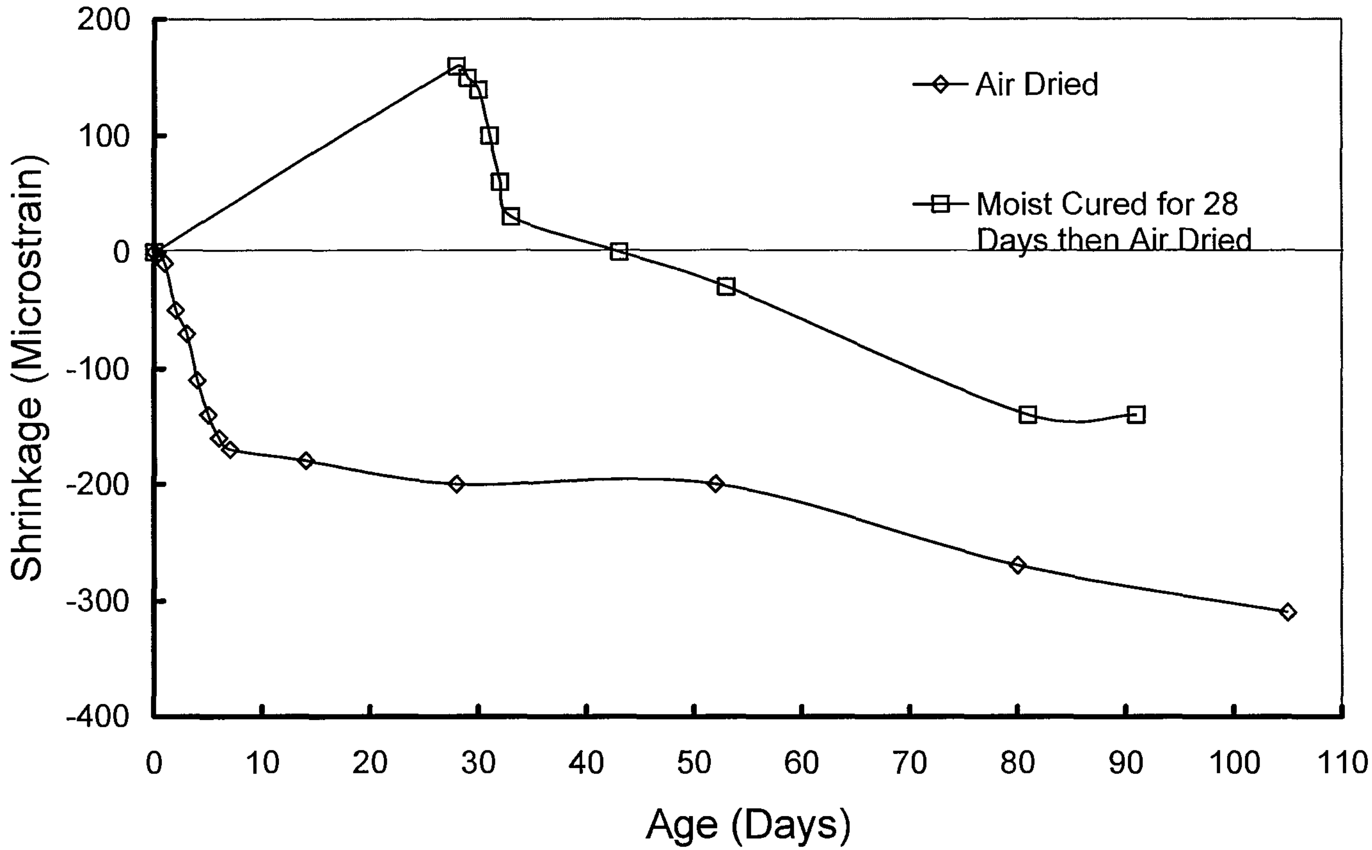


Figure 6.15. Drying Shrinkage After 90 Days for the HPC Girder Mix

Sure Cure™ match-cured cylinders that experienced the same shrinkage, temperature and humidity effects as the creep specimens, but were not put under any load during testing. The specimens had Demec points affixed in a manner similar to that used for the creep specimens. Strains resulting from drying shrinkage and temperature effects were measured using the same Demec mechanical strain gage system as described previously for the creep specimens. Drying shrinkage at 90 days was 290 microstrain. A plot of the shrinkage versus time is given in Figure 6.16.

A report by ACI Committee 209 (1997) recommends the prediction of unrestrained shrinkage in steam-cured concrete subjected to air drying by the following:

$$(\varepsilon_{sh})_t = (t / 55 + t) * (\varepsilon_{sh})_u \qquad (6.9)$$

where t is the drying period and $(\varepsilon_{sh})_u = 780 \times 10^{-6} * \gamma_{sh}$

The variable γ_{sh} represents the product of applicable correction factors. This correction factor is a function of the ambient relative humidity, average thickness of the member, measured slump, fine aggregate percentage, cement content, and air content of the concrete. The predicted drying shrinkage value from Equation 6.9 at 90 days is 310 microstrain.

A summary of the drying shrinkage results is presented Table 6.8. Performance grades defixed by Goodspeed et al. (1996) are based on Procedure B of Table 6.8 at 180 days. Shrinkage under 400 microstrain defines the best category of performance, grade 3. The HPC girder concrete would easily meet the requirements of grade 3.

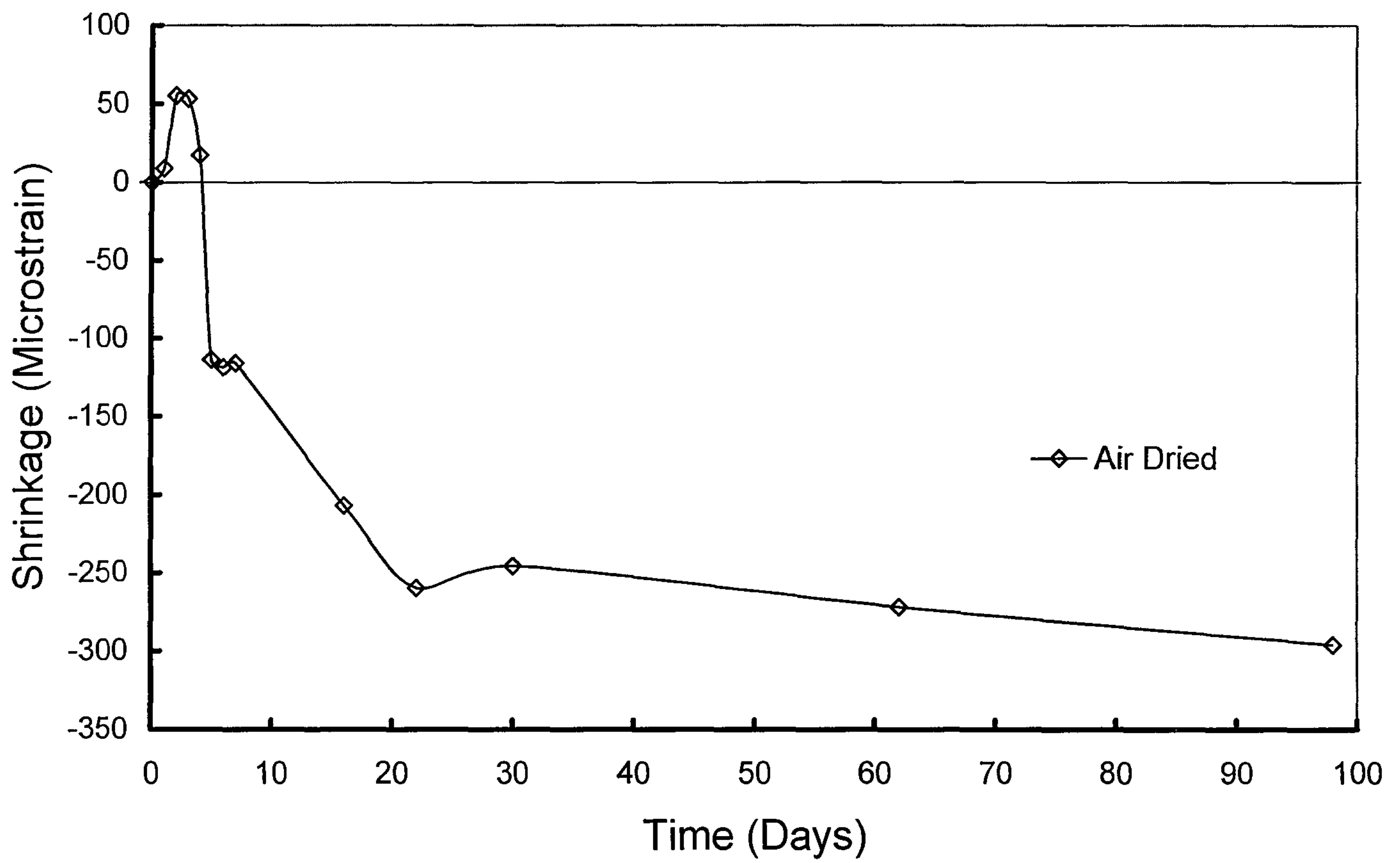


Figure 6.16. Drying Shrinkage After 90 Days for the HPC Girder Mix Using Sure Cure™ Cylinders with Demec Points

Table 6.8. Shrinkage Values after 90 Days for HPC Girder Concrete

Testing Procedure	Shrinkage (microstrain)
Procedure A	280
Procedure B	140
Procedure C	290
ACI 209 Predicted Shrinkage	310

Procedure A – Test conducted according to ASTM C 157. Specimens were air dried for 90 days.

Procedure B – Test conducted according to ASTM C 157. Specimens were moist cured for 28 days, then air dried to 90 days.

Procedure C – Test conducted according to procedure used by Farrington et al. (1996). 4 x 8 in. Sure Cure™ cylinders with attached Demec points were air dried for 90 days.

ACI 209 – Predicted drying shrinkage values at 90 days according to ACI 209R-92

COMPARISON OF HPC AND A STANDARD GIRDER MIX

Properties of the HPC girder mix were compared with properties of a standard concrete girder mix. The properties being compared are shown in Table 6.9. Direct comparisons were made to determine whether the HPC mix used in production of the girders has better strength and durability properties than a standard girder concrete mix.

Specimens used for comparison in strength tests, the rapid chloride ion test, and the creep test were 4 x 8 in. Sure Cure™ match-cured cylinders. Freeze-thaw and shrinkage specimens were obtained as previously discussed in this chapter. Final curing of the specimens was conducted indoors under a controlled environment to eliminate effects of seasonal weather changes on the final curing of the specimens since the samples were obtained during different times of the year.

The standard concrete mix design that was chosen for comparison with the HPC mix is given in Table 6.10. This mix design represented a typical concrete mix that would be used by Sherman Prestressed Concrete to produce standard concrete girders for the ALDOT. The initial curing temperature history of the standard girder mix is shown in Figure 6.17. Samples for strength, rapid chloride ion permeability, creep, and shrinkage tests of the HPC mix were obtained from a special pour of BT-72 girders which closely resembled the placing and curing procedures used during production of the HPC girders. The special pour was required to obtain test specimens from the HPC mix utilizing #67 limestone after actual production of the HPC girders was changed to the mix

Table 6.9. Comparison of HPC and Standard Concrete Properties for Precast Prestressed Girders

Hardened Properties

Compressive Strength (AASHTO T 22)	Release 28 Days 56 Days
Tensile Strength (AASHTO T 198)	Release 28 Days
Modulus of Elasticity (ASTM C 469)	Release 28 Days 56 Days
Freeze-thaw Durability (AASHTO T 161)	300 Cycles
Chloride Ion Permeability (AASHTO T 277)	56 Days 91 Days
Creep (ASTM C 512)	90 Days
Shrinkage (ASTM C 157)	90 Days

4 x 8 in. Sure Cure™ match-cured cylinders were used on all strength, chloride ion permeability, creep and shrinkage tests.

Final curing of cylinders was in laboratory at 70-74 degrees Fahrenheit.

Table 6.10. Mix Proportions Used in Standard Concrete Girder Mix

Material	Source	Cubic Yard Proportions
Type III Cement	Citadel	752 lbs
#67 Limestone	Sherman	2025 lbs
#100 Sand	Sherman	1100 lbs
Water	---	240 lbs
Superplasticizer	Rheobuild® 1000	14 oz/cwt
Water Reducer	Pozzolith® 220-N	3 oz/cwt
Air Entrainment	MB Micro-Air®	5 oz

Water/Cement Ratio = 0.32

using #7 limestone. Freeze-thaw durability and ASTM shrinkage specimens for the HPC mix were obtained during actual production of girders using the HPC mix with #67 limestone coarse aggregate. The initial curing temperature history for the special pour of the HPC girder mix is shown in Figure 6.18. Batch ticket information and fresh properties of both mixes are given in Table 6.11.

RESULTS OF THE COMPARISON BETWEEN THE HPC GIRDER MIX AND A STANDARD CONCRETE GIRDER MIX

The results from direct comparison testing between the HPC girder mix and the standard concrete girder mix are given in Table 6.12. Compressive strength of the HPC girder mix was higher at all ages, particularly at release, which was approximately 24 hours. Values obtained for the tensile strength and modulus of elasticity varied, with the HPC girder mix being slightly higher than the standard concrete girder mix at most ages. Comparison of both the HPC and standard girder concrete splitting tensile strength values to the ACI 363 (1992) and AASHTO (1996) empirical values are given in Figure 6.19. Comparison of both the HPC and standard girder concrete modulus of elasticity values to the ACI 318 (1995) and ACI 363 (1992) empirical values are given in Figure 6.20.

The average Freeze-thaw Durability factor for the HPC mix was 97.6% as compared to 95.1% for the standard girder mix after 300 cycles. The individual specimen results are given in Table 6.13. Both of these values are considered superior based on the definition that Goodspeed et al. (1996) developed for HPC in highway structures. Concrete with a Durability Factor above 80% after 300

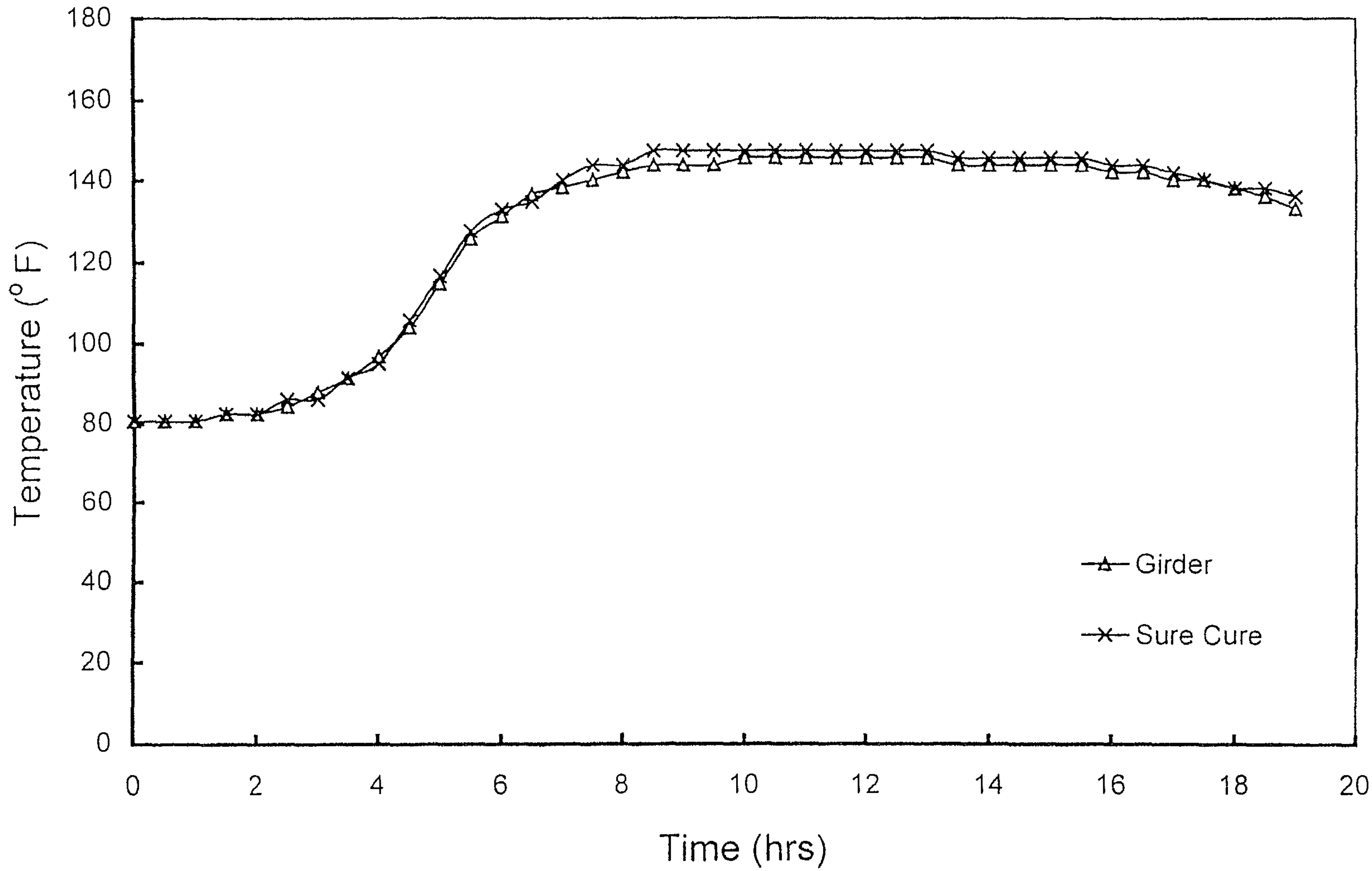


Figure 6:17. Temperature History of the Standard Girder Concrete Mix

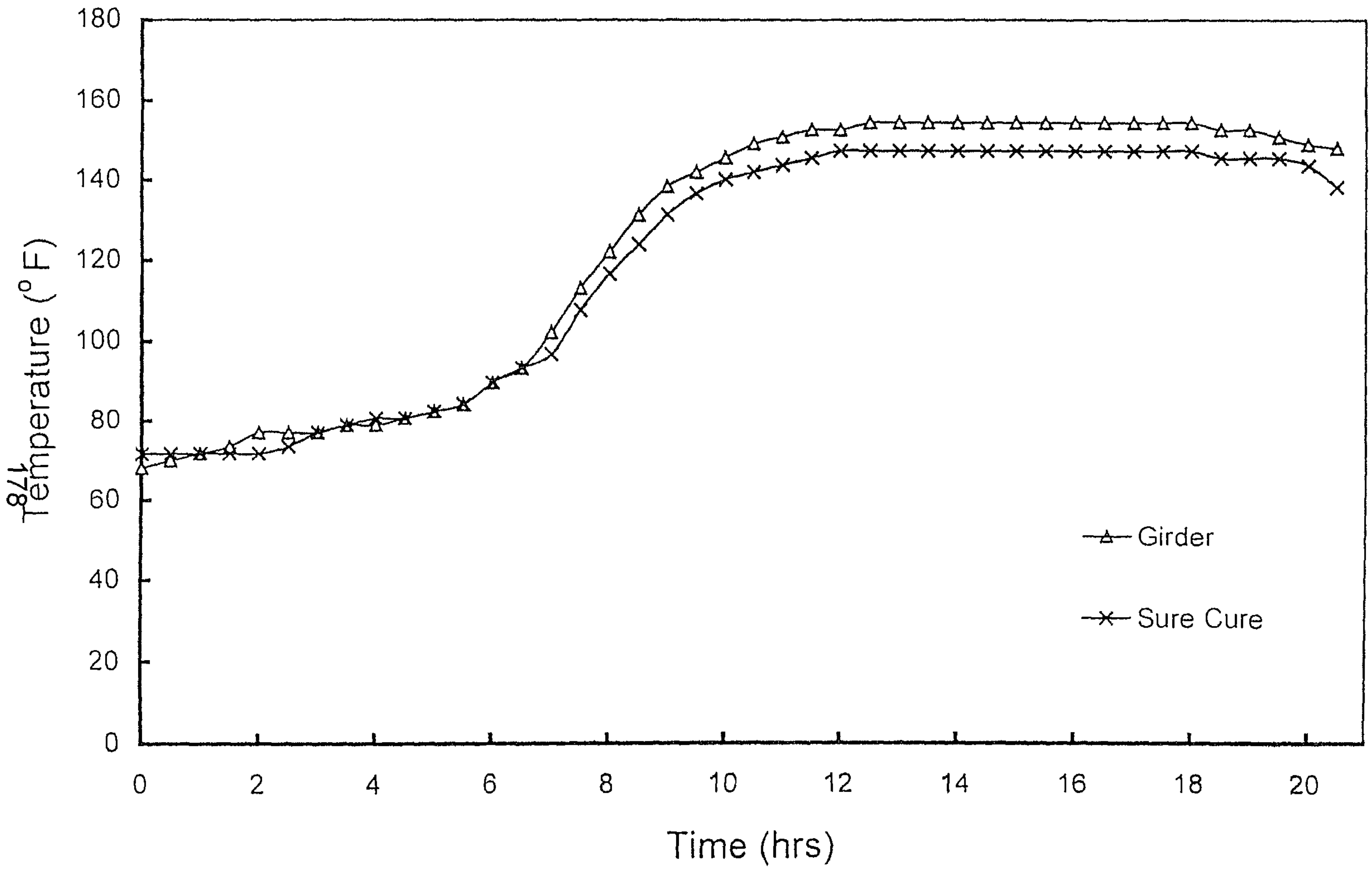


Figure 6.18. Temperature History of the HPC Girder Mix

Table 6.11. Batch Ticket Information and Fresh Concrete Properties

	HPC	Standard Concrete
Fresh Properties		
Slump (in.)	8.0	5.5
Air Content (%)	4.2	3.8
Placement Temperature (°F)	71	78
Total Yards	61	44
Number of Batches	15	11
Average Mix Proportions (lbs/yd^3)		
Type III Cement	751	752
Type C Fly Ash	132	0
#67 Limestone	1863	2028
#89 Sand	385	0
#100 Sand	755	1108
Water	256	244
Admixtures (oz/cwt)		
Rheobuild® 1000	14	14
Pozzolith® 220-N	0	2.9
Delvo®	1.5	0
Admixtures (oz)		
Micro-Air®	4	5
Average w/cm ratio	0.289	.324

Mix proportions are dry weights.

Table 6.12. Comparison of HPC and Standard Girder Concrete Properties[1]

Hardened Properties		HPC	Standard Concrete
Compressive Strength (psi)	Release	9150	7070
	28-Day	11340	10360
	56-Day	11140	10210
Tensile Strength (psi)	Release	770	610
	28-Day	850	870
Modulus of Elasticity (psi)	Release	5,900,000	5,250,000
	28-Day	6,300,000	5,900,000
	56-Day	6,000,000	5,900,000
Freeze-Thaw Durability (Durability Factor, %)	300 Cycles	97.6%	95.1%
Chloride Ion Permeability (Coulombs)	56-Day	2130	2790
	91-Day	1850	3080
Creep (microstrain/psi)	90-Day	0.20	0.54
Shrinkage (microstrain)	90-Day / Air[2]	280	350
	90-Day / Moist[3]	140	250
	90-Day / Cyl.[4]	290	430

1. See Appendix A, Table A.19 for individual cylinder test results.
2. Specimens were air dried for 90 days.
3. Specimens were moist cured for 28 days, then air dried to 90 days.
4. Procedure used by Farrington et al. (1996). Specimens air dried for 90 days.

4 x 8 in. Sure Cure™ match-cured cylinders were used where applicable. Final cure was indoors at 70-74 degrees Fahrenheit.

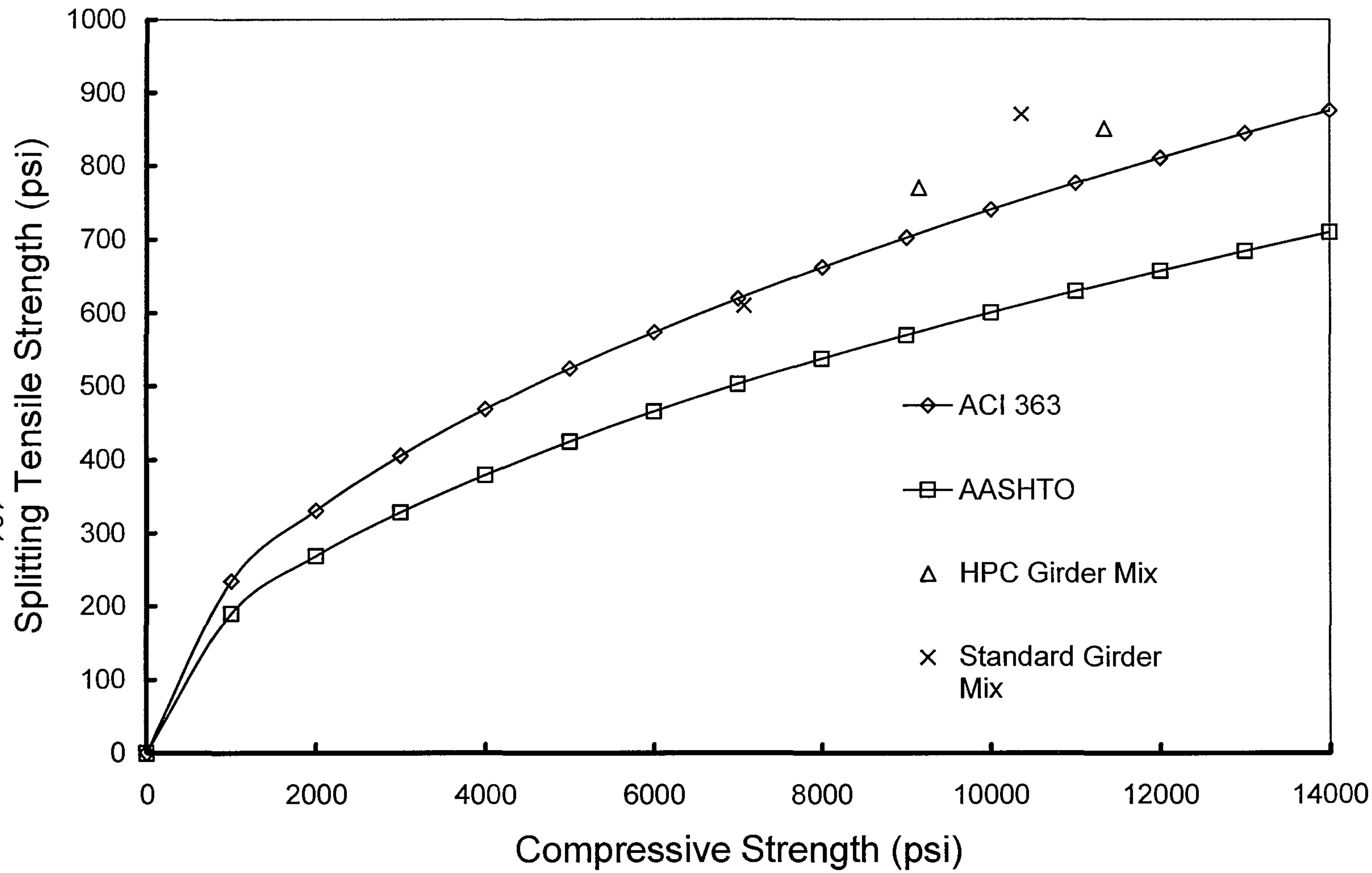


Figure 6.19. Comparison of Splitting Tensile Strength to ACI 363 and AASHTO Values

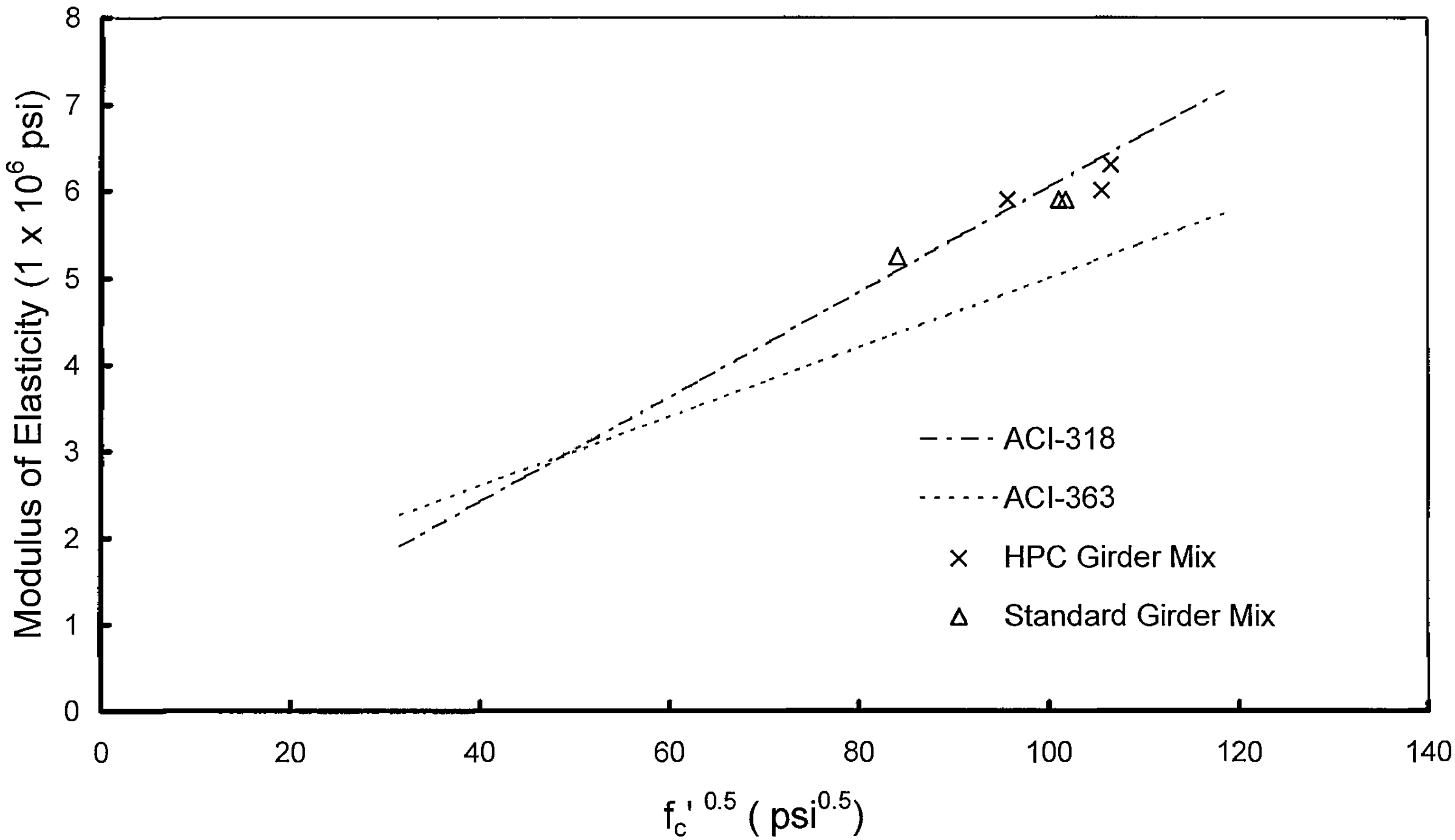


Figure 6.20. Modulus of Elasticity Results for HPC and Standard Concrete 4 x 8 in. Sure Cure™ Cylinders

Table 6.13. Freeze-Thaw Durability for the HPC Girder Mix and a Standard Concrete Girder Mix

Mix	Initial Frequency (Hz)	Final Frequency (Hz)	Durability Factor (%)
HPC	2500	2450	96.0
	2460	2450	99.2
	2470	2440	97.6
			Average = 97.6
Standard Mix	2520	2450	94.5
	2540	2490	96.1
	2540	2470	94.6
			Average = 95.1

Tests were conducted for 300 freeze-thaw cycles with three samples per mix.

cycles meets the requirements of the highest performance grade by those definitions. Plots of Durability Factor versus Number of Cycles for HPC and standard girder mix are given in Figures 6.21 and 6.22, respectively.

Rapid chloride ion permeability of the HPC girder mix was lower than that of the standard concrete girder mix. The permeability values obtained for the HPC mix were 2130 coulombs at 56 days versus 2790 coulombs for the standard mix, and 1850 coulombs at 91 days versus 3080 coulombs for the standard mix. At 56 days both mixes fell within the AASHTO T 277 (1997) rating of moderate. At 91 days, the permeability of the standard mix was still considered moderate, while the permeability of the HPC mix improved to a rating of low by AASHTO T 277 (1997) standards. The lower permeability results obtained by the HPC mix can be attributed to the use of pozzolans in the mix design, as opposed to the standard mix which used only cement.

Values obtained at 90 days for specific creep were 0.20 microstrain/psi for the HPC mix and 0.54 microstrain/psi for the standard concrete mix. A plot of specific creep versus time for HPC girder mix was given previously in Figure 6.13. Figure 6.23 shows a comparison of specific creep versus time for the standard girder mix and HPC girder mix up to 90 days. As previously mentioned, the specific creep value of the HPC girder mix places it in the highest performance grade category, while the value achieved by the standard girder mix is slightly higher than the 0.52 to 0.41 microstrain/psi range needed to be placed in the lowest performance grade category as defined by Goodspeed et al. (1996).

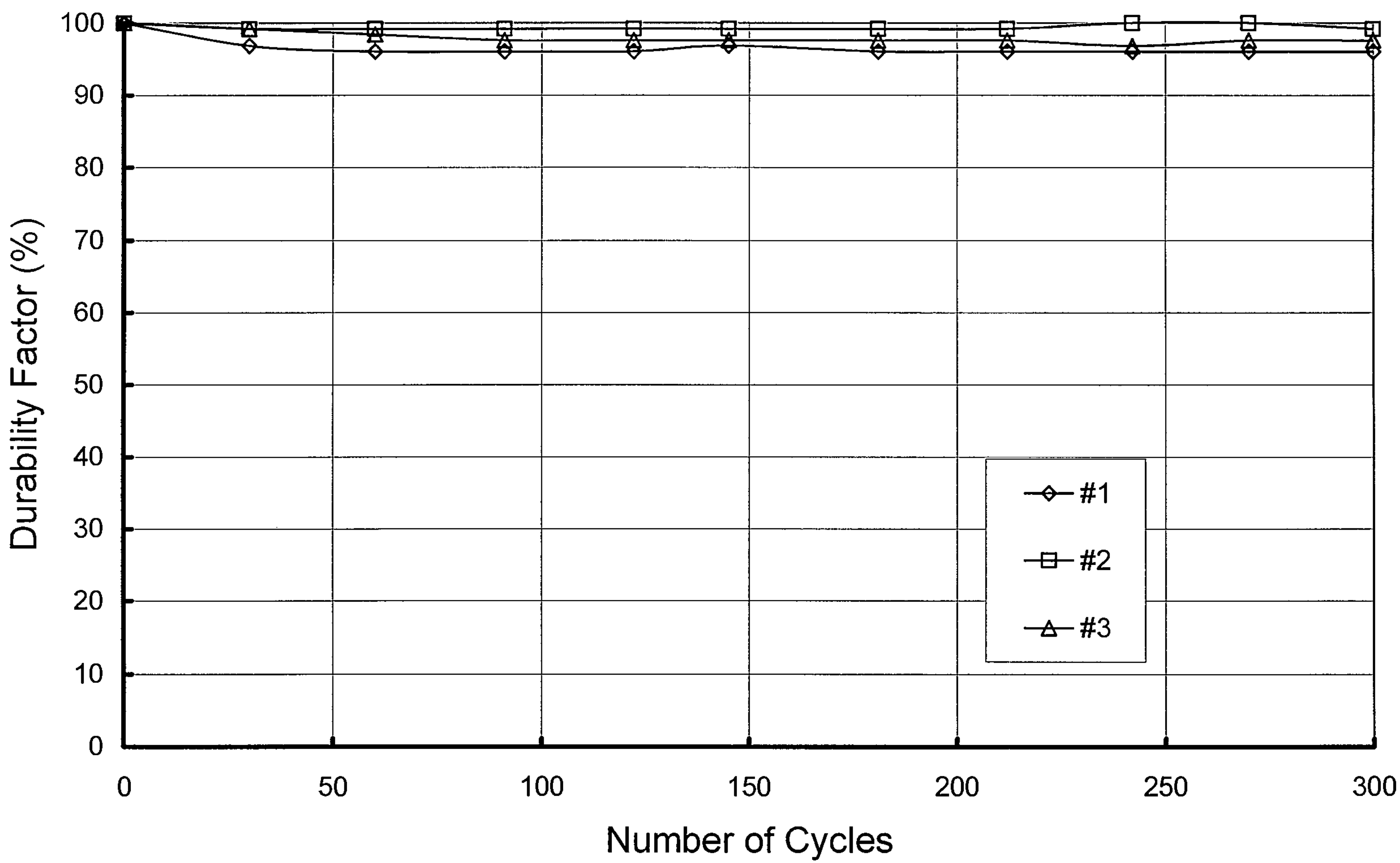


Figure 6.21. Durability Factor for 300 Cycles for the HPC Girder MIx

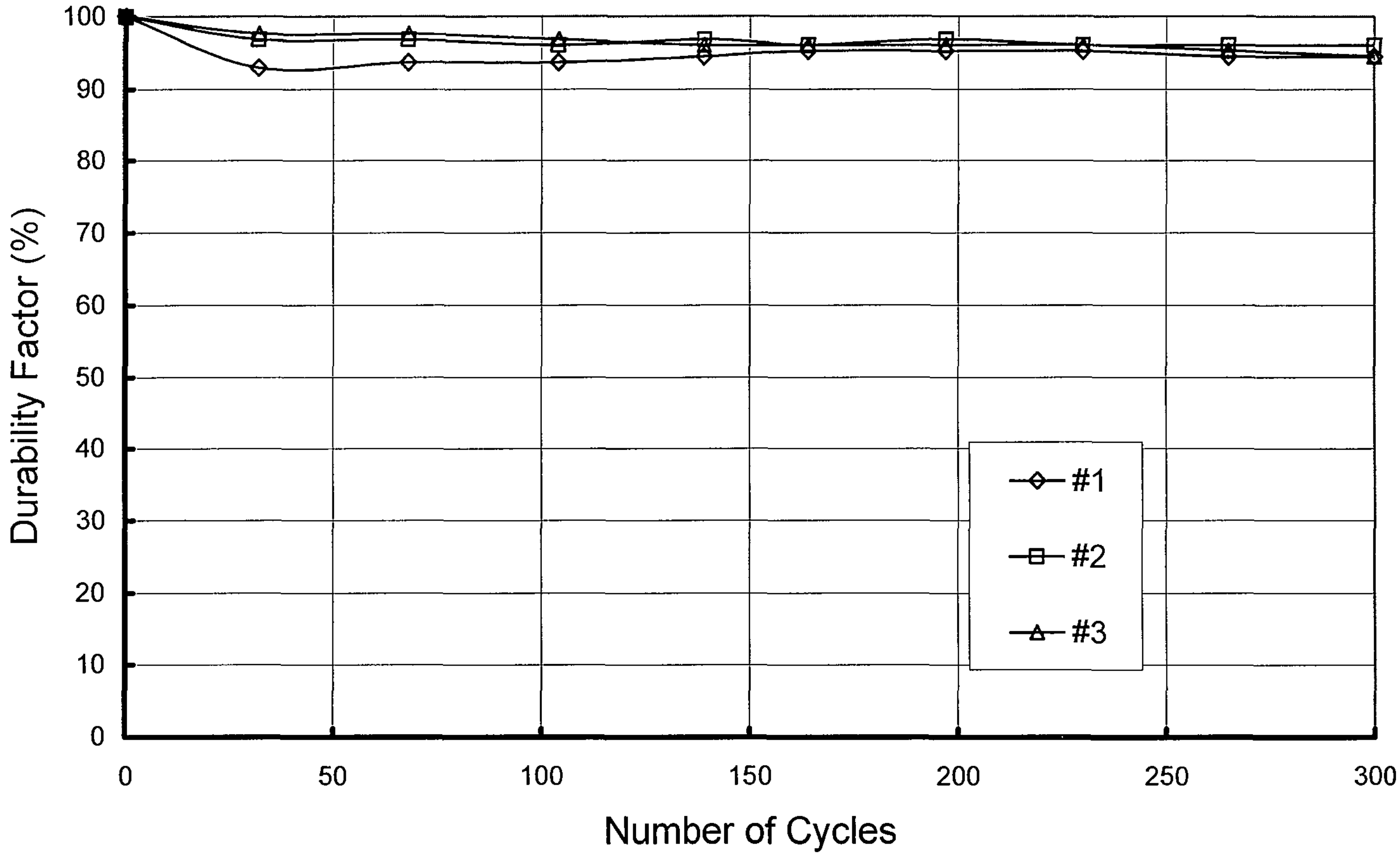


Figure 6.22. Durability Factor for 300 Cycles for the Standard Girder Concrete Mix

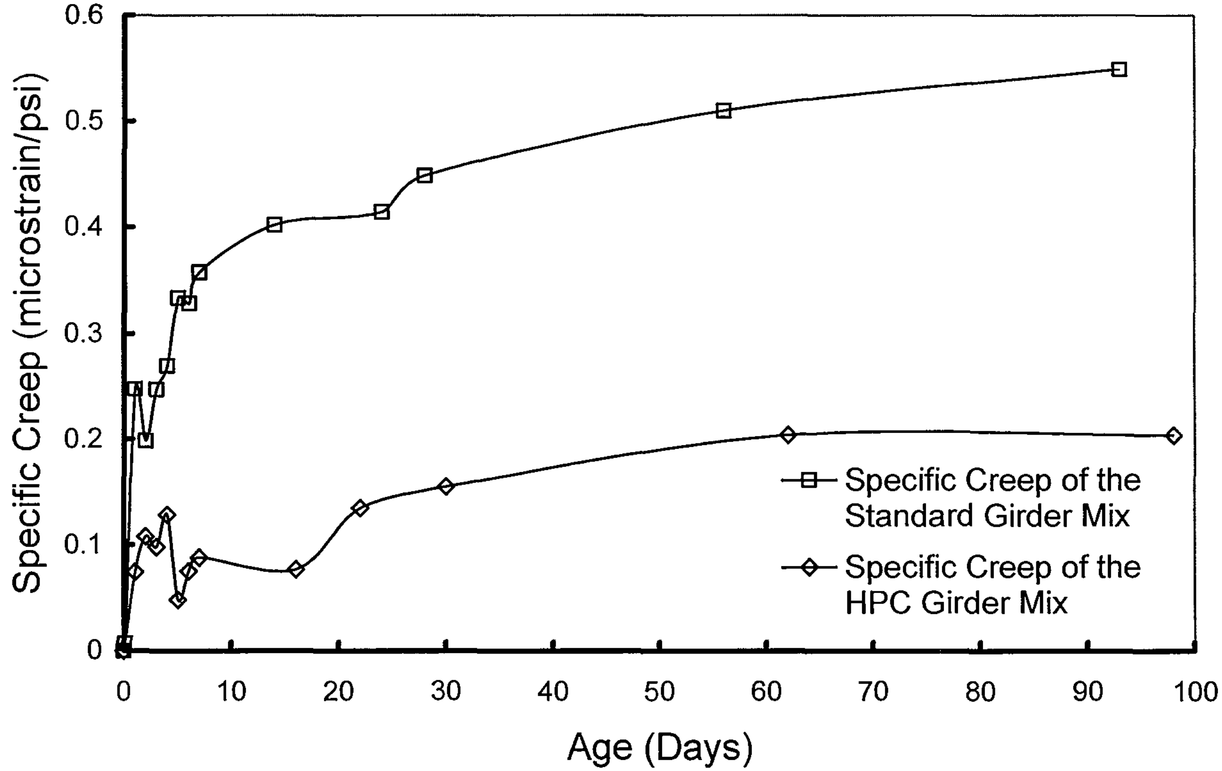


Figure 6.23. Comparison of Specific Creep up to 90 Days for the Standard Girder Mix and the HPC Girder Mix

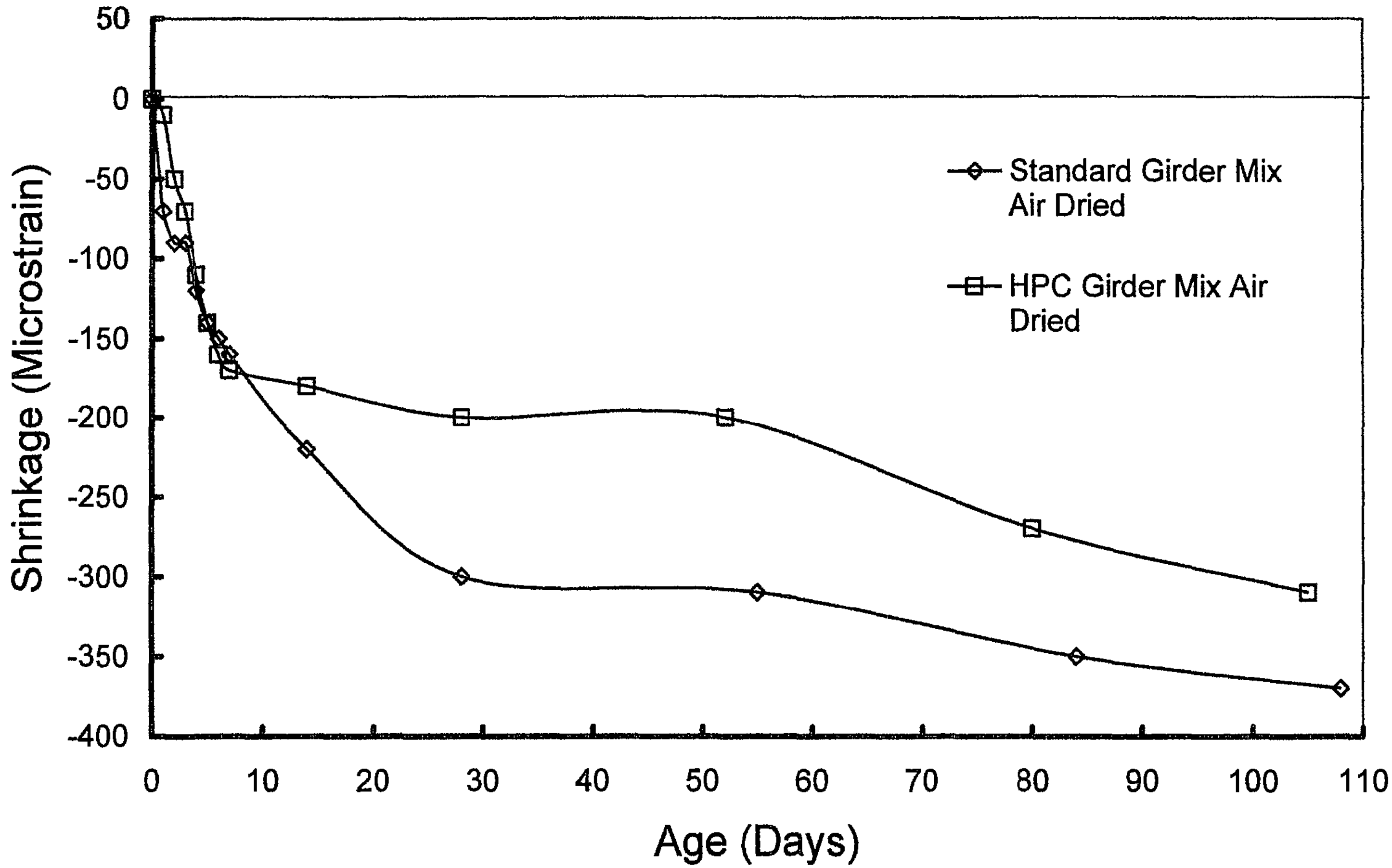


Figure 6.24. Comparison of the Drying Shrinkage after 90 Days for the Standard Girder Mix and the HPC Girder Mix Subjected to Immediate Air Drying Using ASTM Procedures

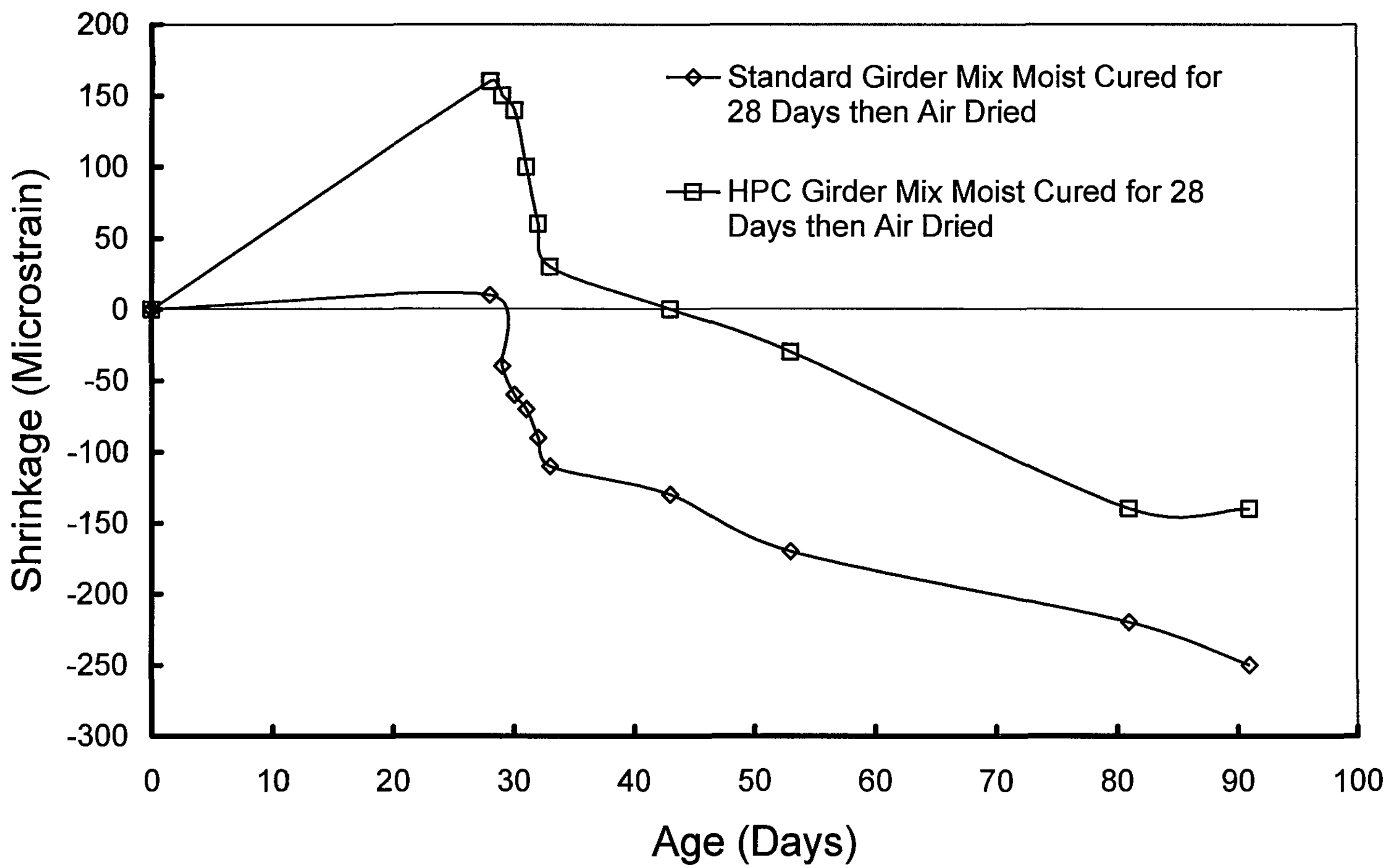


Figure 6.25. Comparison of the Drying Shrinkage after 90 Days for the Standard Girder Mix and the HPC Girder Mix Subjected to Moist Curing for 28 Days then Air Dried Using ASTM Procedures

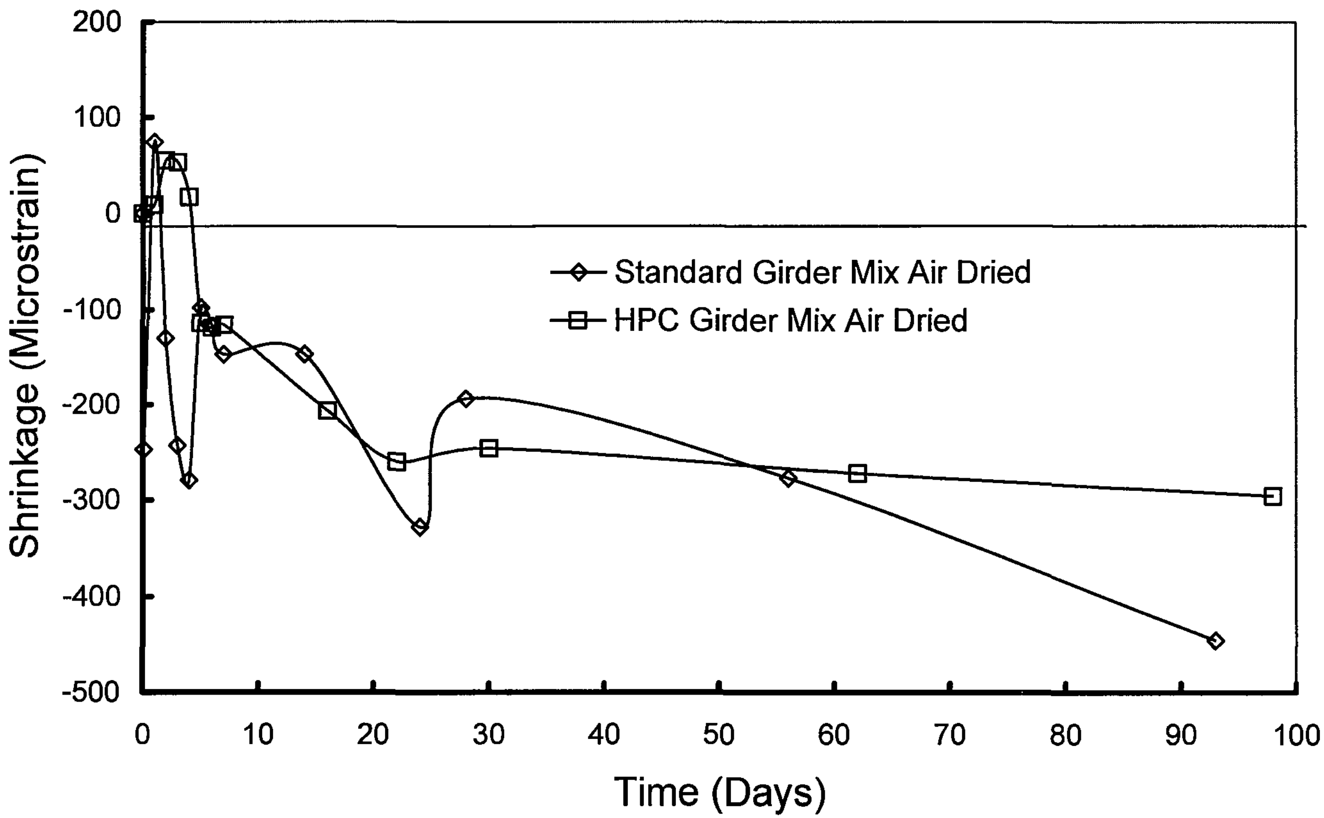


Figure 6.26. Comparison of Drying Shrinkage After 90 Days for the Standard Girder Mix and the HPC Mix Using Sure Cure™ Cylinders with Demec Points

The lower specific creep value obtained by the HPC can be attributed to the *w/cm* and the use of fly ash in the mix design.

Shrinkage values obtained at 90 days according to ASTM C 157 (1996) procedures in which the samples were subjected immediately to air drying were 280 microstrain for the HPC mix and 350 microstrain for the standard concrete mix. A plot of shrinkage versus time for the standard girder mix and HPC girder mix immediately air cured and tested according to ASTM procedures is given in Figure 6.24. Shrinkage values for specimens that were moist cured for 28 days then subjected to air drying were 140 microstrain for the HPC mix and 250 microstrain for the standard concrete mix. A plot of shrinkage versus time for the standard girder mix and HPC girder mix for specimens that were moist cured for 28 days then air cured and tested according to ASTM procedures is given in Figure 6.25. Shrinkage values obtained according to the procedure used by Farrington et al. (1996) in which Sure Cure™ cylinders with Demec points attached to the sides were measured for shrinkage yielded 290 microstrain for the HPC mix and 430 microstrain for the standard concrete mix. Figure 6.26 shows a plot of shrinkage versus time for the standard girder mix and HPC girder mix by the procedure used by Farrington et al. (1996).

IS HPC THE PROPER CHOICE?

The question that ultimately arises is whether HPC is a better choice than conventional concrete. Depending on the viewpoint taken, the use of HPC in the production of precast prestressed girders can be argued in many ways. From the standpoint of mechanical properties of a particular concrete, HPC girder concrete should be considered superior to standard girder concrete. Results from Table 6.12 clearly show that the HPC outperformed the standard girder concrete mix in virtually every category. The performance grade criteria used by Goodspeed et al. (1996) to rate HPC was given in Figure 2.1. A comparison of the performance grades received by the standard girder mix to those of the HPC utilizing #67 limestone and #7 limestone is given in Table 6.14.

From a designers' point of view, the use of high strength HPC in the design of precast prestressed bridge girders allows for longer spans using fewer girders of the same size, or the use of a shallower section spanning the same distance. This results in materials and labor cost savings. Also, the use of longer girder spans in bridge construction saves on the number of foundations needed to span a certain distance. For the HPC bridge project reported here, the additional material and production costs of using HPC did not outweigh the savings resulting from using fewer girders with longer spans and fewer foundations.

The use of cementitious replacement materials especially fly ash, typically provides a cost saving over a cement only mix in obtaining a given strength particularly at later ages, while giving the mix superior strength and durability properties as compared to a mix containing only cement. This is evident by the results given in Table 6.12. Often associated with the use of HPC is the need for experience using admixtures and pozzolans, trial mix designs, better quality control procedures and modern batching equipment that would add to the cost of producing HPC girders. The use of HPC in precast prestressed girders can be beneficial if the engineer, precast producer, and contractor all have knowledge of the associated advantages and disadvantages of HPC and are able to face these challenges in an efficient manner.

Table 6.14. Comparison of the Performance Grades[1] Received by the Standard Concrete Mix and HPC Mix

		Standard Concrete		HPC - #67 Limestone[2]		HPC - #7 Limestone[3]	
Performance Characteristic		Average	Grade	Average	Grade	Average	Grade
Compressive Strength (psi)	56-Day	10210	3	10420	3	11750	3
Tensile Strength (psi)	28-Day	870	n.a.[4]	740	n.a.[4]	660	n.a.[4]
Modulus of Elasticity (psi)	56-Day	5,900,000	1	5,450,000	1	6,100,000	2
Freeze-Thaw Durability (Durability Factor, %)	300 Cycles	95.1	2	97.8	2	98.2	2
Chloride Ion Permeability (Coulombs)	56-Day	2790	1	2580	1	2270	1
Creep (microstrain/psi)	90-Day	0.54	n.a.[5]	0.20	4	n.a.[6]	n.a.[6]
Shrinkage (microstrain)[7]	90-Day / Air 90-Day / Moist 90-Day / Cyl.	350 250 430	3 3 2	280 140 290	3 3 3	n.a.[6]	n.a.[6]

Averages of compressive strength, tensile strength, modulus of elasticity and chloride ion permeability based on 4 x 8 in. Sure Cure™ match cured cylinders cured on the casting yard or indoors.

1. Based on Goodspeed et al. (1996) and Table 1 shown in Figure 2.1.
2. Based on Production pours 1 through 12 and data from Table 6.12.
3. Based on Production pours 13 through 18.
4. Performance grade category not given for splitting tensile strength.
5. Value outside of range needed to be given a performance grade.
6. Data not available.
7. Refer to Table 6.12 for description of each testing procedure.

CHAPTER SEVEN

PERFORMANCE OF THE CAST-IN-PLACE HPC MIX

INTRODUCTION

A testing plan was developed to measure the performance characteristics of the CIP HPC. Properties determined for the CIP HPC are given in Table 7.1. Samples were obtained from four different concrete pours during the production of the CIP HPC on the Uphapee Creek Relief Bridge. All strength test results presented in this chapter were determined in addition to the tests performed by ALDOT as required by project specifications. A set of test data was obtained from concrete poured into a webwall on the Uphapee Creek Relief Bridge, and three sets of data were obtained from concrete placed on the Uphapee Creek Relief Bridge superstructure. The mix designs tested were previously presented in Tables 5.3 and 5.4. Descriptions of each testing procedure along with the results are given in following sections. The properties of the CIP HPC were then compared with the properties of a standard CIP concrete mix. Direct comparison testing between the two mixes was conducted to determine whether the HPC had better performance properties than a standard concrete mix. Results of those comparisons are given at the end of this chapter.

Table 7.1. Properties of the CIP HPC Concrete

Fresh Properties	
Slump (ASTM C 143)	
Air Content (ASTM C 231)	
Temperature (ASTM C 1064)	
Unit Weight (ASTM C 138)	

Hardened Properties	
Compressive Strength (AASHTO T 22)	7 Days 28 Days 56 Days 91 Days
Tensile Strength (AASHTO T 198)	7 Days 28 Days 56 Days 91 Days
Modulus of Elasticity (ASTM C 469)	7 Days 28 Days 56 Days 91 Days
Freeze-thaw Durability (AASHTO T 161)	300 Cycles
Chloride Ion Permeability (AASHTO T 277)	56 Days 91 Days
Shrinkage (ASTM C 157)	90 Days
Abrasion Resistance (ASTM C 944)	56 Days

FRESH CONCRETE PROPERTIES

The fresh concrete properties shown in Table 7.1 were measured by ACI certified personal from the ALDOT during each of the pours throughout production of the CIP HPC. Air content, concrete temperature, unit weight and slump measurements were made from each truckload of concrete arriving from the batch plant. Air content, concrete temperature and slump measurement results for the samples obtained in this research are given Table 7.2. Batch ticket information for each mix is given in Appendix A, Table A.20. The average unit weight of the concrete was approximately 145 lbs/ft^3.

COMPRESSIVE STRENGTH

Compressive strength properties of the CIP HPC were obtained according to procedures in AASHTO T 22 (1997) "Standard Specification for Compressive Strength of Cylindrical Concrete Specimens." All compressive strength tests were conducted at Auburn University on standard 6 x 12 in. cylinders made in plastic molds and tested on 70 durometer neoprene pads. Test cylinders were made according to ASTM C 31 (1996) and stored in a wooden box at the bridge site during the first 24 hours, then transported back to Auburn and placed in a moist room for final curing. The temperature and humidity in the moist room was 70-74°F and 90-95%, respectively.

Cylinders were tested at 7, 28, 56 and 91 days to determine the compressive strength properties of the CIP HPC. Compressive strength ranged from 5170 psi at 7 days to 8630 psi at 91 days. Compressive strength results for

Table 7.2. Fresh Concrete Properties Measured During Production of the CIP HPC

Sample Number[1]	Mix Number[2]	Fresh Properties		
		Air Content (%)	Slump (in.)	Temperature (°F)
1	HPC-3	6.5	7.0	76
2	HPC-4	3.6	4.5	81
3	HPC-4	4.7	5.25	76
4	HPC-4	3.9	4.0	81

1. 1 – Webwalls located on Uphapee Creek Relief Bridge
   2 – Deck Span 4 located on Uphapee Creek Relief Bridge
   3 – Deck Span 6 located on Uphapee Creek Relief Bridge
   4 – Deck Span 3 located on Uphapee Creek Relief Bridge
2. Batch ticket information from each mix is given in Appendix A, Table A.20.

all ages are given in Table 7.3. Significant compressive strength gains were obtained from 28 to 91 days. Average compressive strength results of all the test sets averaged 14% higher at 91 days than 28 days. This continued strength gain after 28 days is attributed to the 20% Type C fly ash used in the mix. Conclusions on the long-term effects of pozzolans are presented in Chapter Nine.

TENSILE STRENGTH

Tensile strength properties of the CIP HPC were obtained according to AASHTO T 198 (1997) "Standard Specification for Splitting Tensile Strength of Cylindrical Concrete Specimens." Tests were conducted at Auburn University on standard 6 x 12 in. cylinders made in plastic molds and cured in the same manner as the compressive strength cylinders. Splitting tensile strength tests were conducted at 7, 28, 56, and 91 days. Strengths ranged from 350 psi at 7 days to 560 psi at later ages. The average 56-day splitting tensile strength was 490 psi. Tensile strength results for all ages are given in Table 7.4.

The ACI Committee 363 (1992) "State-of-the-Art Report on High Strength Concrete" recommends the following equation for compressive strengths between 3000 and 12,000 psi for the splitting tensile of strength of concrete:

$$f_{sp}' = 7.4 \sqrt{f_c'} \text{ (psi)} \qquad (7.1)$$

ACI 318-95 (1995) Section 11.2 Commentary states for normal weight concrete, the splitting tensile strength is approximately equal to:

$$f_{sp}' = 6.7\sqrt{f_c'} \text{ (psi)} \qquad (7.2)$$

Table 7.3. Compressive Strength Test Results from Production of the CIP HPC

Sample Number[2]	Mix Number	Compressive Strength (psi)[1]			
		7-Day	28-Day	56-Day	91-Day
1	HPC-3	5330	6490	7400	8100
2	HPC-4	5810	7440	8220	8630
3	HPC-4	5280	7220	7440	7870
4	HPC-4	5170	6450	6940	7370

Results are the average of two cylinders for sample 1 and the average of three cylinders for samples 2,3 and 4. All cylinders were tested on 70 durometer neoprene pads at Auburn University.

1. Refer to Appendix A, Table A.21 for individual cylinder test results.
2. 1 – Webwalls located on Uphapee Creek Relief Bridge
   2 – Deck Span 4 located on Uphapee Creek Relief Bridge
   3 – Deck Span 6 located on Uphapee Creek Relief Bridge
   4 – Deck Span 3 located on Uphapee Creek Relief Bridge

Table 7.4. Splitting Tensile Strength Results from Production of the CIP HPC

Sample Number[2]	Mix Number	Tensile Strength (psi)[1]			
		7-Day	28-Day	56-Day	91-Day
1	HPC-3	400	440	460	520
2	HPC-4	440	530	520	490
3	HPC-4	410	530	490	560
4	HPC-4	350	470	490	430

Results are the average of two cylinders tested at Auburn University.

1. Refer to Appendix A, Table A.22 for individual cylinder test results.
2. 1 – Webwalls located on Uphapee Creek Relief Bridge
   2 – Deck Span 4 located on Uphapee Creek Relief Bridge
   3 – Deck Span 6 located on Uphapee Creek Relief Bridge
   4 – Deck Span 3 located on Uphapee Creek Relief Bridge

Comparison of these results and those predicted by the ACI 363 (1992) and ACI 318-95 (1995) equations are shown in Figure 7.1. Results obtained in this research fell below the predicted values given by ACI 363 (1992) and ACI 318-95 (1995). Low tensile strengths may be due to the size and shape of coarse aggregate used in the concrete mix design. The #57 coarse aggregate used in the CIP HPC contained flat and elongated particles which possibly lead to reduced aggregate-paste bond strength.

MODULUS OF ELASTICITY

Modulus of elasticity properties of the CIP HPC were obtained according to ASTM C 469 (1996) "Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression." Tests were conducted at Auburn University on standard 6 x 12 in. cylinders made in plastic molds and cured in the same manner as the compressive strength test cylinders. Modulus of elasticity was measured at 7, 28, 56 and 91 days. Results ranged from a 4,050,000 psi at 7 days to over 7,000,000 psi at later ages. Modulus of Elasticity results for all ages are given in Table 7.5.

An empirical relationship given by ACI 318-95 Building Code (1995), Section 8.5 Formula states:

$$E_c = 33\ w^{1.5} \sqrt{f_c'} \quad \text{(psi)} \tag{7.3}$$

where w = true unit weight of the mix in pounds per cubic foot. The ACI Committee 363 (1992) State-of-the-Art Report on High Strength Concrete

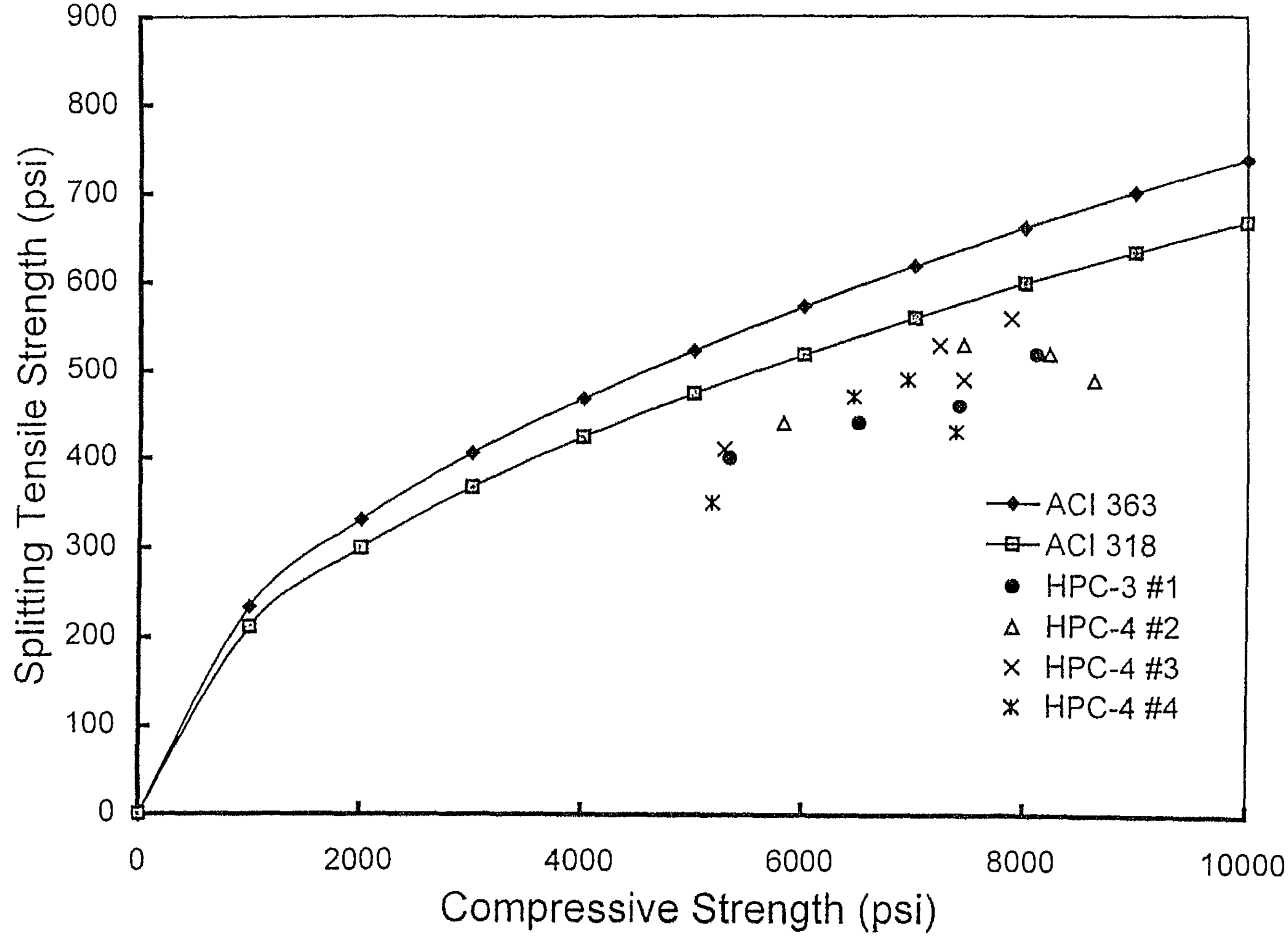


Figure 7.1. Splitting Tensile Strength Results for CIP HPC

recommends the following equation for compressive strengths between 3000 and 12,000 psi:

$$E_c = 40{,}000 \sqrt{f_c'} + 1.0 \times 10^6 \text{ (psi)} \qquad (7.4)$$

Comparison of the ACI 363 (1992) and ACI 318 (1995) equations to values obtained during production of the CIP HPC is given in Figure 7.2. The unit weight used in Equation 7.3 was 145 lbs/ft^3. A majority of the test results were above the ACI 363 (1992) and ACI 318 (1995) predicted values.

FREEZE-THAW DURABILITY

Freeze-Thaw durability properties of the CIP HPC were measured according to AASHTO T 161 (1997) "Standard Specification for Resistance of Concrete to Rapid Freezing and Thawing." Samples were 3 x 4 x 16 in. cast in steel molds and initially cured according to ASTM C 31 (1996). Samples were stripped after 24 hours and cured in a lime bath for 14 days. At 14 days, the specimen's initial fundamental transverse frequency was recorded using a sonometer, then the specimen was put into the freeze-thaw cabinet for repeated freezing and thawing cycles. The sonometer and freeze-thaw apparatus was previously shown in Chapter 6. The test was conducted according to Procedure A from AASHTO T 161 in which the specimens were surrounded in 1/32 in. to 1/8 in. of water throughout the freeze-thaw cycle. The specimens went through a complete freeze-thaw cycle every 3-4 hours. A freeze-thaw cycle consists of lowering the temperature of the specimen from 40°F to 0°F and raising it from 0°F to 40°F. Samples were tested approximately every 30 to 36 cycles in the

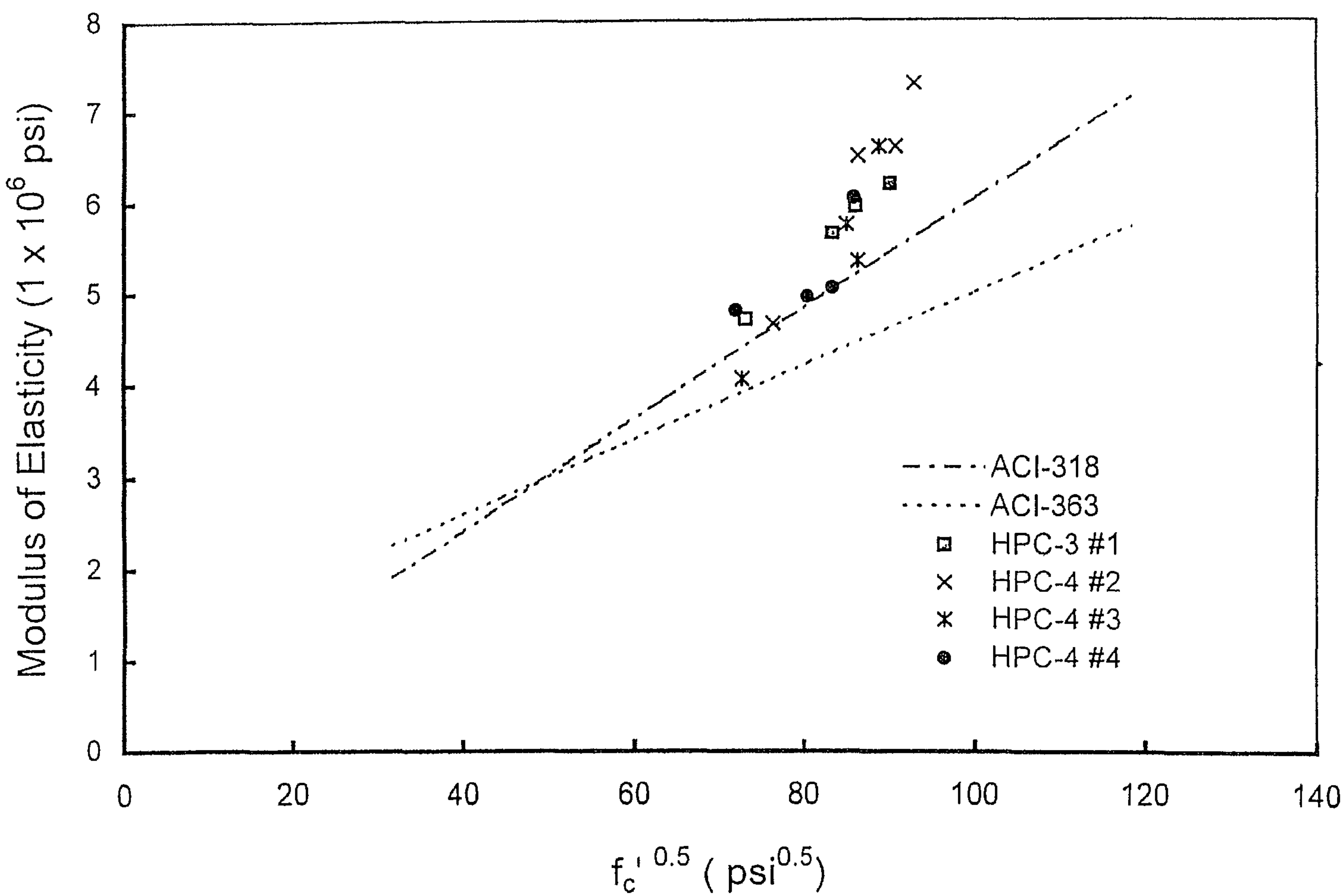


Figure 7.2. Modulus of Elasticity Results for CIP HPC

Table 7.5. Modulus of Elasticity Test Results from Production of the CIP HPC

Sample Number[2]	Mix Number	Modulus of Elasticity (psi)[1]			
		7-Day	28-Day	56-Day	91-Day
1	HPC-3	4,700,000	5,650,000	5,950,000	6,200,000
2	HPC-4	4,650,000	6,500,000	6,600,000	7,300,000
3	HPC-4	4,050,000	5,750,000	5,350,000	6,600,000
4	HPC-4	4,800,000	4,950,000	5,050,000	6,050,000

1. Result of one cylinder tested at Auburn University.
2. 1 – Webwalls located on Uphapee Creek Relief Bridge
   2 – Deck Span 4 located on Uphapee Creek Relief Bridge
   3 – Deck Span 6 located on Uphapee Creek Relief Bridge
   4 – Deck Span 3 located on Uphapee Creek Relief Bridge

thawed state for fundamental transverse frequency. Tests were conducted for 300 cycles. At 300 cycles, the samples were removed from the freeze-thaw cabinet and tested for the final fundamental transverse frequency. Resistance to freeze-thaw is given by the Relative Dynamic Modulus of Elasticity, or Durability Factor (DF). The equation used to calculate the DF is shown below where n is the fundamental transverse frequency (Hz) at 0 cycles and n_1 is the fundamental transverse frequency (Hz) at 300 cycles.

$$DF = (n_1^2) / (n^2) \text{ X } 100 \tag{7.5}$$

Results of the freeze-thaw testing are given in Table 7.6. Plots of Durability Factor vs. Number of Cycles for each test specimen are given in Figures 7.3 – 7.6. The average Durability Factor from the four pours tested was 89.9%. In the definition that Goodspeed et al. (1996) developed for HPC in highway structures, different grades of performance characteristics are identified for high performance structural concrete. Concrete with a Durability Factor above 80% after 300 cycles has the highest performance grade. The guidelines set forth by Goodspeed et al. (1996) identifies a set of concrete performance characteristics sufficient to estimate long-term concrete durability for highway structures. The measured Durability Factors indicate the HPC mixes have excellent long-term resistance to freeze-thaw damage.

CHLORIDE ION PERMEABILITY

Permeability properties of the CIP HPC was measured according to AASHTO T 277 (1997) "Standard Specification for Electrical Indication of

Table 7.6. Freeze-Thaw Durability Results from Production of the CIP HPC

Sample Number[1]	Initial Frequency (Hz)	Final Frequency (Hz)	Durability Factor (%)
1	2270	2090	84.8
	2280	2130	87.3
	2280	2140	88.1
			Average = 86.7
2	2340	2230	90.8
	2350	2170	85.3
	2350	2250	91.7
			Average = 89.3
3	2270	2170	91.4
	2270	2160	90.5
	2260	2200	94.8
			Average = 92.2
4	2320	2230	92.4
	2320	2220	91.6
	2330	2220	90.8
			Average = 91.6

Tests were conducted for 300 freeze-thaw cycles with three samples per mix.

1. 1 – Webwalls located on Uphapee Creek Relief Bridge
   2 – Deck Span 4 located on Uphapee Creek Relief Bridge
   3 – Deck Span 6 located on Uphapee Creek Relief Bridge
   4 – Deck Span 3 located on Uphapee Creek Relief Bridge

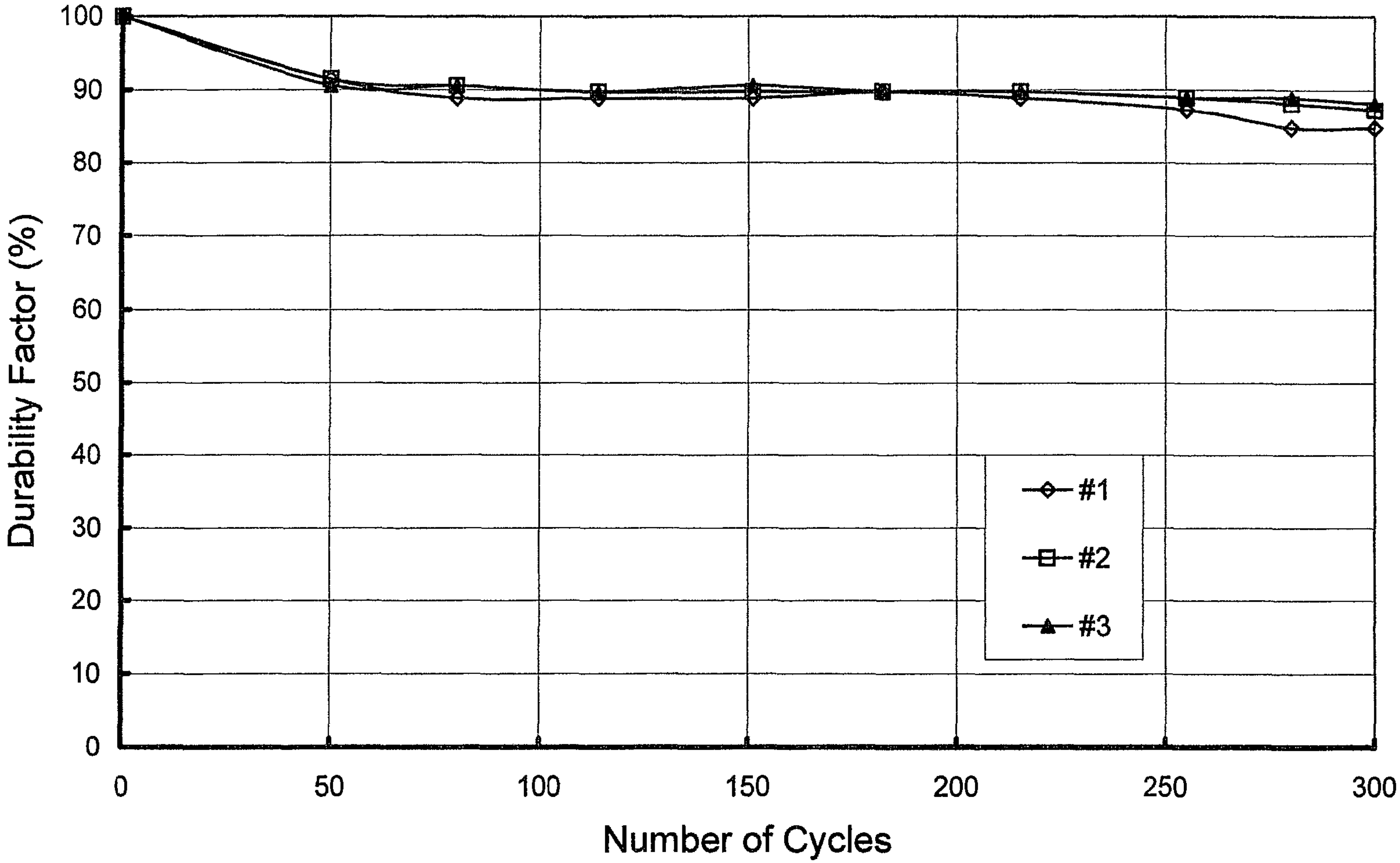


Figure 7.3. Durability Factor for 300 cycles for HPC – 3 #1

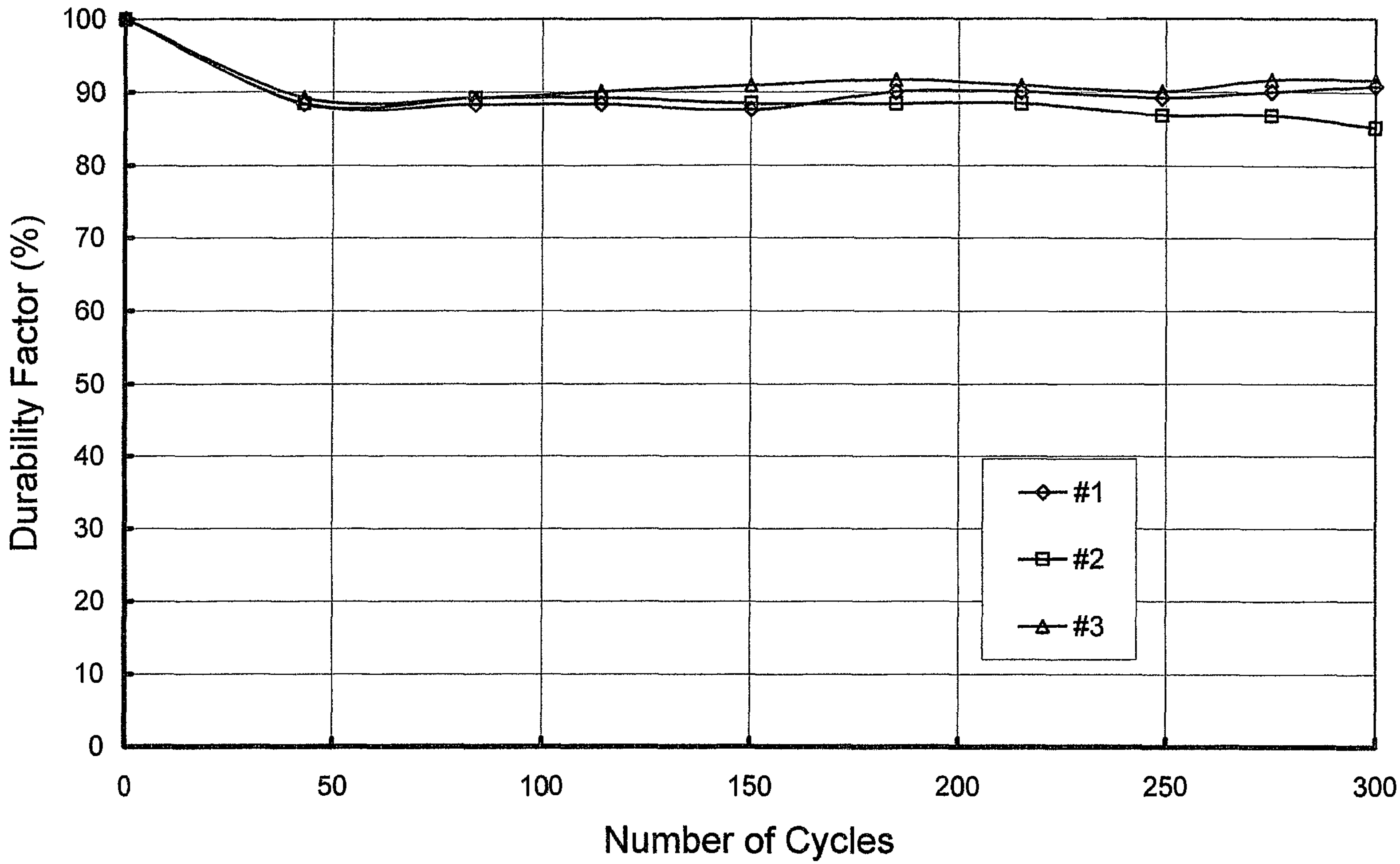


Figure 7.4. Durability Factor for 300 Cycles for HPC – 4 #2

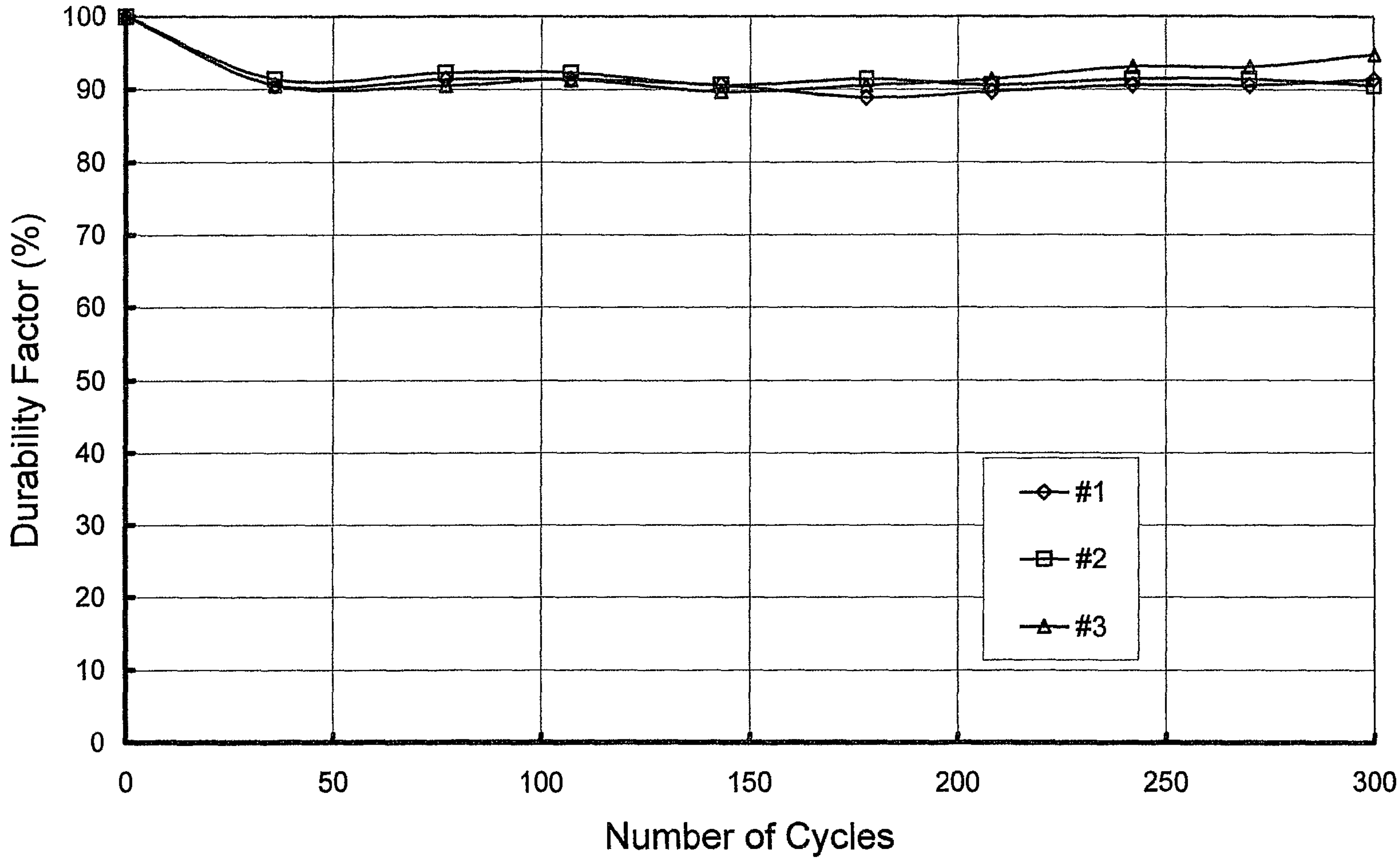


Figure 7.5. Durability Factor for 300 Cycles for HPC – 4 #3

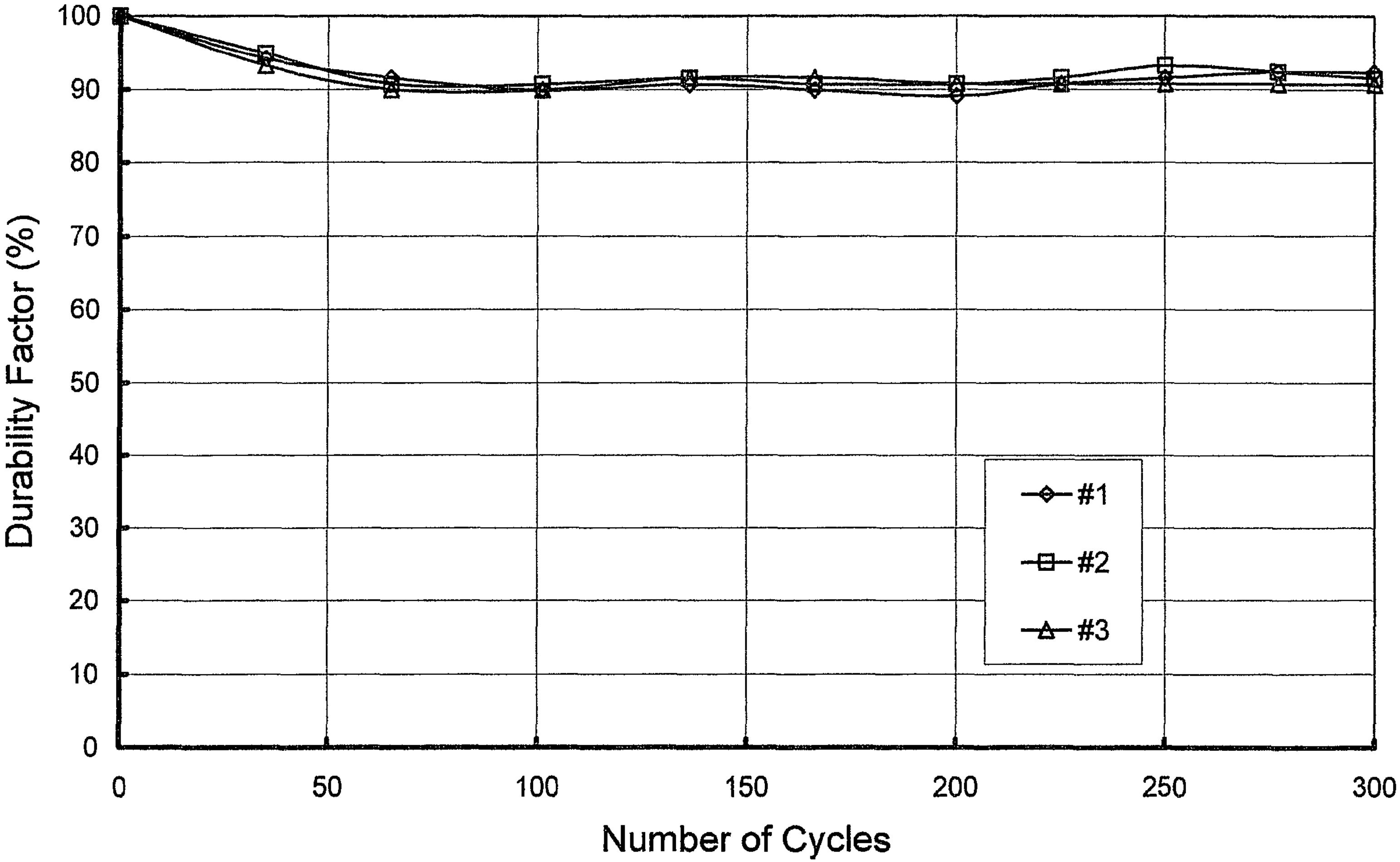


Figure 7.6. Durability Factor for 300 Cycles for HPC – 4 #4

Concrete's Ability to Resist Chloride Ion Penetration." Conducting the chloride ion test consists of monitoring the amount of electrical current that passes thorough a 2 in. thick slice (4 in.) nominal diameter specimen during a 6 hour period. A 60 Volt current is maintained across the ends of the specimen, one of which is immersed in a sodium chloride solution, the other in a sodium hydroxide solution. The total charge passed is measured in coulombs. The amount of coulombs passed during the 6 hour long test has been found to be related to the resistance of the specimen to chloride ion penetration. A brief summary of the testing procedure can be found in Chapter 6.

Test samples were standard 4 x 8 in. cylinders made in plastic molds and cured in the same manner as the compressive strength test specimens. Tests were conducted at 56 and 91 days at Auburn University. Tests were conducted at 91 days to study the long-term effects of the use of pozzolans on the permeability of concrete. Values obtained for the total charge passed are based upon test samples with diameters of 3.75 inches. For specimen diameters other than 3.75 in., the value for total charge passed is adjusted by the following:

$$Q_{FINAL} = Q_x \text{ x } (3.75 / x)^2 \qquad (7.6)$$

where Q_{FINAL} = charge passed in coulombs through a 3.75 in. diameter specimen, x = diameter of the nonstandard specimen, and Q_x = charge passed in coulombs through x diameter specimen. The diameter of the specimens tested in this research where approximately 4.0 in. and where adjusted by Equation 7.6.

Table 7.7. Rapid Chloride Ion Penetration Results from Production of the CIP HPC Mix

Sample Number[1]	Mix Number	Test Age (Days)	Charge Passed (coulombs)	AASHTO T 277 Rating[2]
1	HPC-3	56	3010	Moderate
1	HPC-3	56	2940	Moderate
1	HPC-3	91	2760	Moderate
1	HPC-3	91	2710	Moderate
2	HPC-4	56	2870	Moderate
2	HPC-4	56	2800	Moderate
2	HPC-4	91	2010	Moderate
2	HPC-4	91	1980	Low
3	HPC-4	56	2680	Moderate
3	HPC-4	56	2850	Moderate
3	HPC-4	91	1990	Low
3	HPC-4	91	1930	Low
4	HPC-4	56	3000	Moderate
4	HPC-4	56	3040	Moderate
4	HPC-4	91	2110	Moderate
4	HPC-4	91	2060	Moderate

1. 1 – Webwalls located on Uphapee Creek Relief Bridge
   2 – Deck Span 4 located on Uphapee Creek Relief Bridge
   3 – Deck Span 6 located on Uphapee Creek Relief Bridge
   4 – Deck Span 3 located on Uphapee Creek Relief Bridge
2. Refer to Table 1 in AASHTO T 277-93

A summary of the test results are given in Table 7.7. Values ranged from 3000 coulombs at 56 days to under 2000 coulombs at 91 days. A majority of the test results were between 2000 and 4000 coulombs which is classified as moderate permeability by AASHTO T 277 (1997) standards. The average values at 56 days and 91 days for the HPC-4 mixes were 2870 coulombs and 2010 coulombs, respectively. These are close to the 2460 coulombs at 56 days found for the match-cured cylinders of the HPC girder concrete. The CIP HPC contained a higher percentage of Type C fly ash (70% compared to 15%), but also had a higher *w/cm* (approximately 0.36 compared to 0.27).

A reduction in the permeability of the concrete was observed from 56 to 91 days. The reason for these results is believed to be the continued hydration due to the use of fly ash in the mix design. Fly ash reacts with calcium hydroxide which is a by-product of the hydration of Portland cement. This results in a reduction in the permeability at later ages. Similar results were observed in research conducted by Kuzmar (1999) in which a significant reduction in the permeability of concrete containing fly ash was observed from early age testing to tests conducted beyond 90 days.

SHRINKAGE

Shrinkage properties of the CIP HPC were obtained according to ASTM C 157 (1996) "Standard Specification for Length Change of Hardened Hydraulic-Cement Mortar and Concrete." Samples were cast in 3 x 3 x 12 in. steel molds and initially cured according to ASTM C 31 (1996). Specimens were stripped from the steel molds approximately 24 hours after being cast and their length was recorded. The apparatus used to measure the length of the specimens was previously shown in Chapter 6. Shrinkage samples were then cured in a lime bath for either 7 or 28 days. Samples wet cured for 7 days were chosen to simulate the actual wet curing time the bridge deck received in the field, while samples wet cured for 28 days were chosen because it was recommended by ASTM C 157 (1996) as well as by Goodspeed et al. (1996). At 7 and 28 days, samples were removed from the lime bath and measured for their change in length. Samples were then air dried for a period of 90 days. Specimens were measured for change in length at various times during the drying period. Drying shrinkage results for all ages are given in Table 7.8. Plots of shrinkage versus time are given in Figures 7.7 to 7.10.

A report by ACI Committee 209 (1997) recommends the prediction of unrestrained shrinkage in moist cured concrete by the following:

$$(\varepsilon_{sh})_t = (t / 35 + t) * (\varepsilon_{sh})_u \qquad (7.7)$$

where t is the drying period and $(\varepsilon_{sh})_u = 780 \times 10^{-6} * \gamma_{sh}$

The variable γ_{sh} represents the product of applicable correction factors. This correction factor is a function of the ambient relative humidity, average thickness of the member, measured slump, fine aggregate percentage, cement content, and air content of the concrete. The average calculated drying shrinkage value using the properties of the four mixes tested and Equation 7.7 was 320 microstrain after 90 days. A comparison between the calculated value and those

Table 7.8. Shrinkage Values for CIP HPC Mix After 90 Days

Sample Number[1]	Test Procedure[2]	Shrinkage (Microstrain)
1	B	260
Calculated Value, Equation 7.7		340
2	A	470
2	B	280
Calculated Value, Equation 7.7		310
3	A	480
3	B	330
Calculated Value, Equation 7.7		320
4	A	250
Calculate Value, Equation 7.7		300

1. 1 – Webwalls located on Uphapee Creek Relief Bridge
   2 – Deck Span 4 located on Uphapee Creek Relief Bridge
   3 – Deck Span 6 located on Uphapee Creek Relief Bridge
   4 – Deck Span 3 located on Uphapee Creek Relief Bridge

2. Procedure A – Test conducted according to ASTM C 157, except specimens were moist cured for 7 days, then air dried.

   Procedure B – Test conducted according to ASTM C 157, specimens were moist cured for 28 days, then air dried.

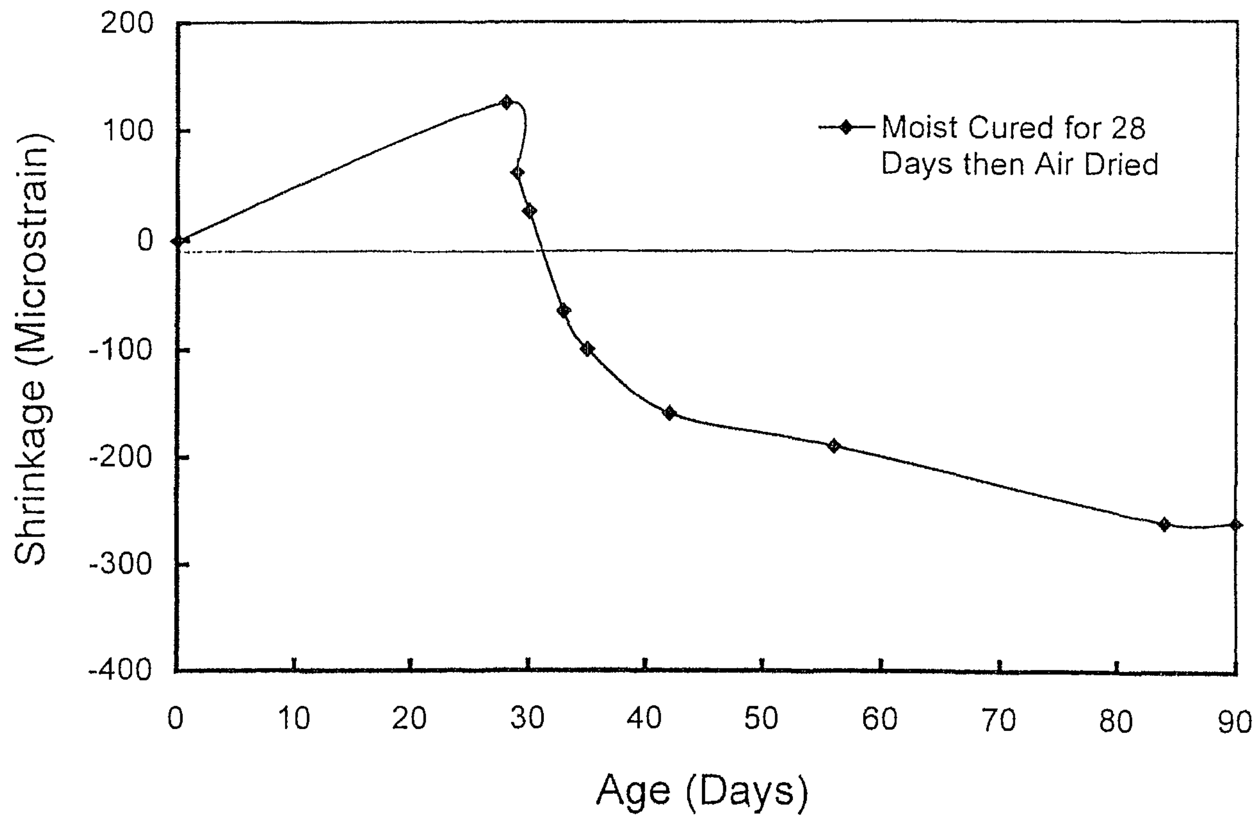


Figure 7.7. Drying Shrinkage of HPC-3 #1

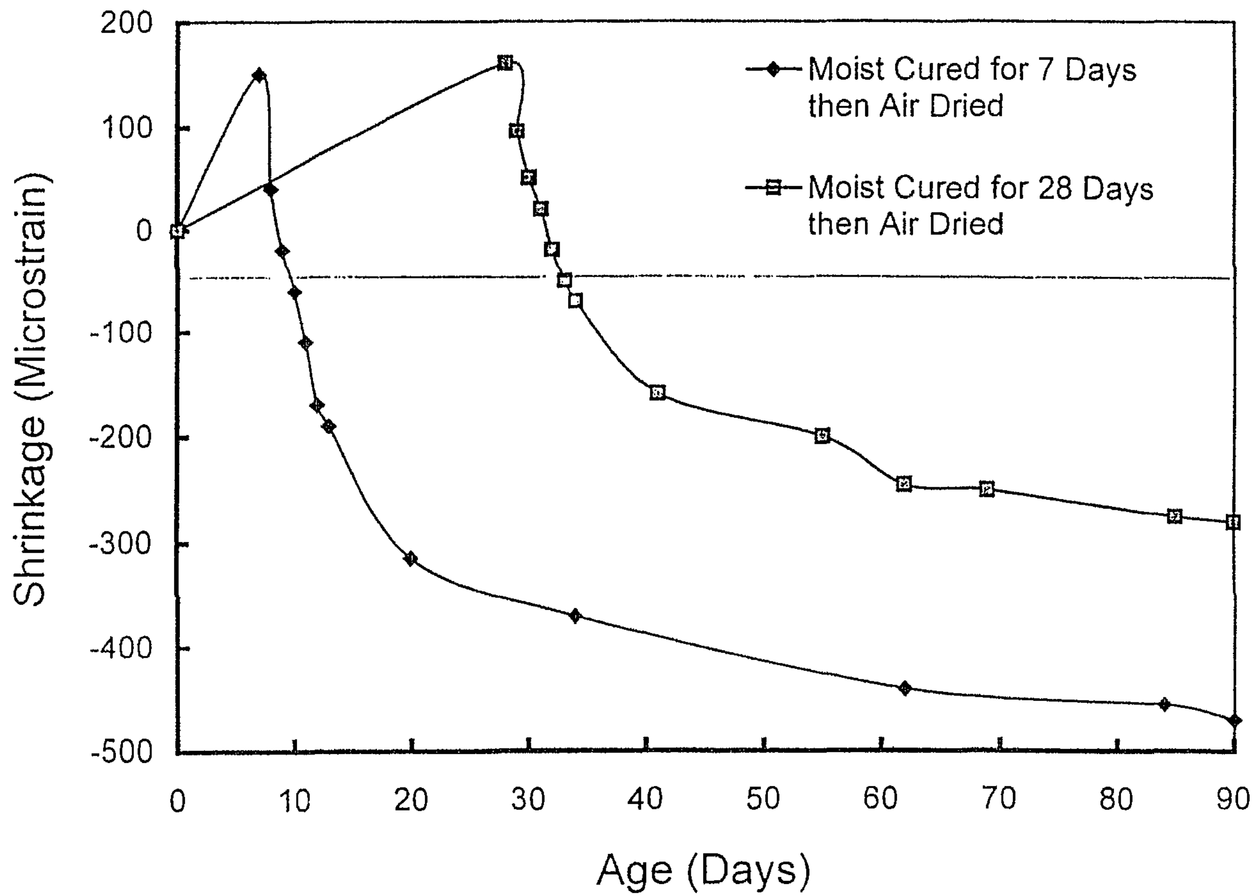


Figure 7.8. Drying Shrinkage of CIP HPC-4 #2

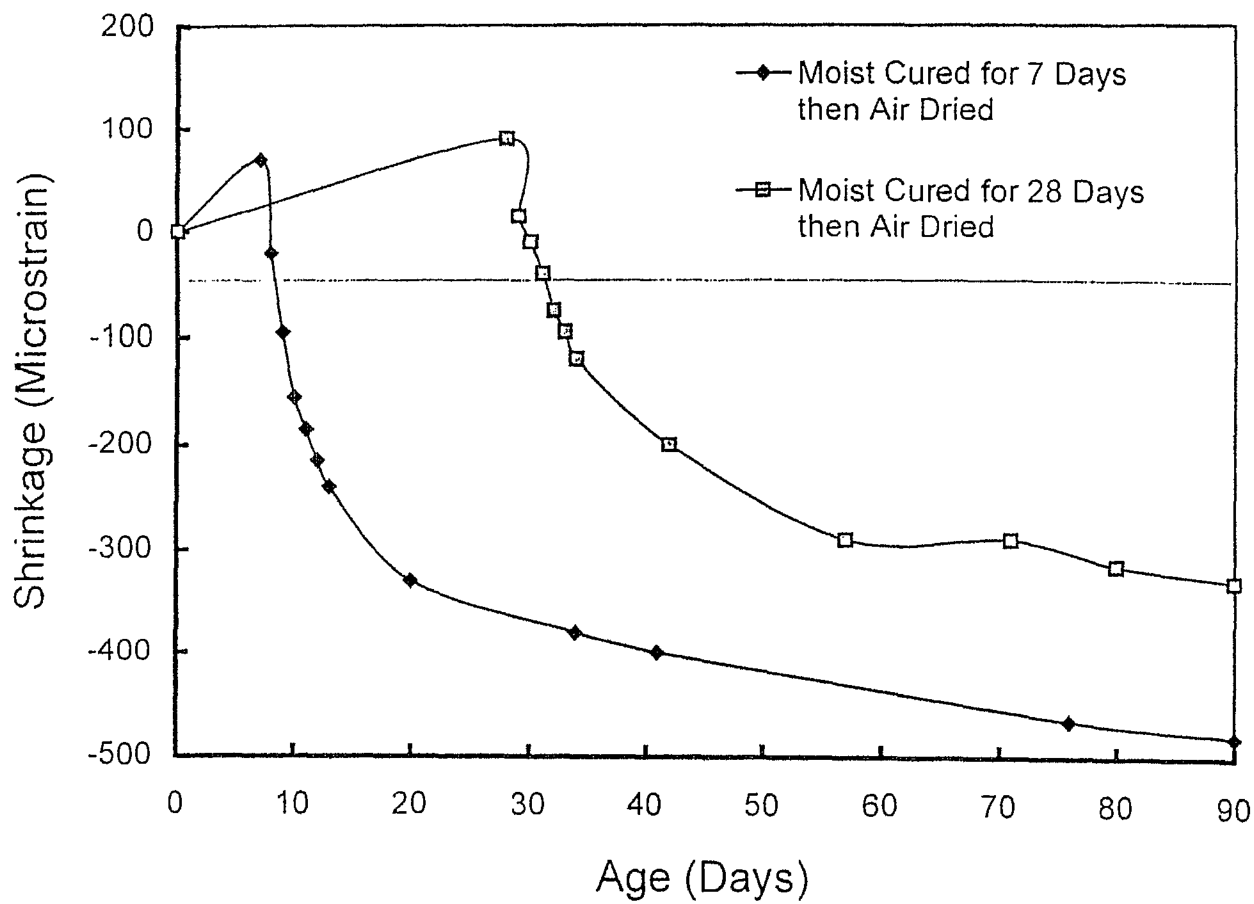


Figure 7.9. Drying Shrinkage of CIP HPC-4 #3

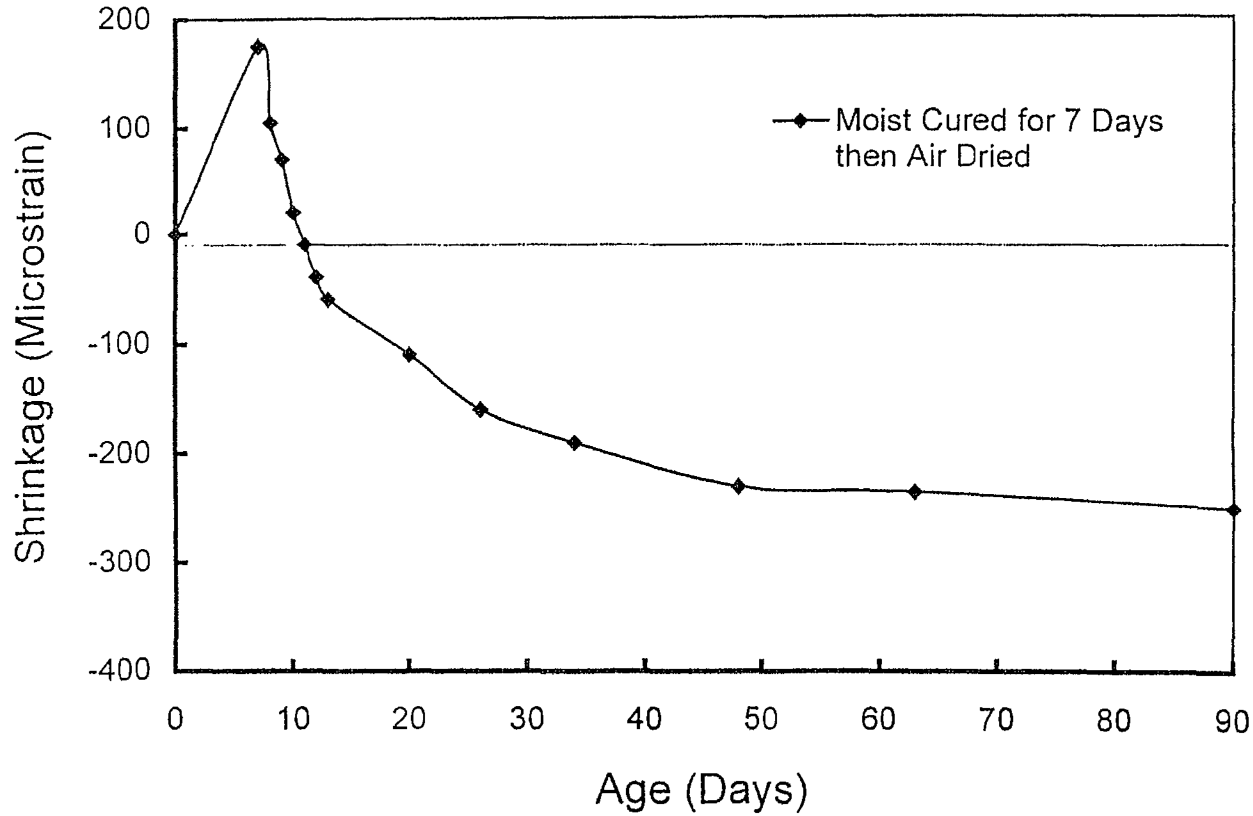


Figure 7.10. Drying Shrinkage of CIP HPC-4 #4

of samples that were moist cured for 7 days then subjected to air drying to 90 days showed the value calculated using ACI 209 (1997) is a lower estimate of the drying shrinkage. Comparison of the calculated value to samples moist cured for 28 days then air dried to 90 days showed better correlation.

ABRASION RESISTANCE

The abrasion resistance of the CIP HPC was determined according to ASTM C 944 (1996) "Standard Test Method for Abrasion Resistance of Concrete or Mortar Surfaces by the Rotating-Cutter Method." A sample was cast in a 6 x 12 x 2 in. wooden form and finished according to the same procedures used on the actual bridge deck. Upon initial set of the concrete, the sample was covered with water soaked burlap for the first 24 hours, then transported back to the laboratory for moist curing. The sample was moist cured for 7 days to simulate the wet curing the bridge deck received during the first 7 days after it was poured. After wet curing, the sample was air cured until it was tested at 56 days. The abrasion resistance test sample is shown in Figure 7.11.

Abrasion resistance testing was conducted by the Louisiana Transportation Research Center. Tests were conducted at 56 days according to ASTM C 944 (1996) procedures. Testing was conducted on three areas of the specimen for two minutes each. At each location, a 22 lbf cutter at 200 rpm was applied for a two minute abrasion period. A rotating-cutter drill press that was used to perform abrasion resistance tests is shown in Figure 7.12. Results presented in this research are for the amount of mass loss per unit area of the

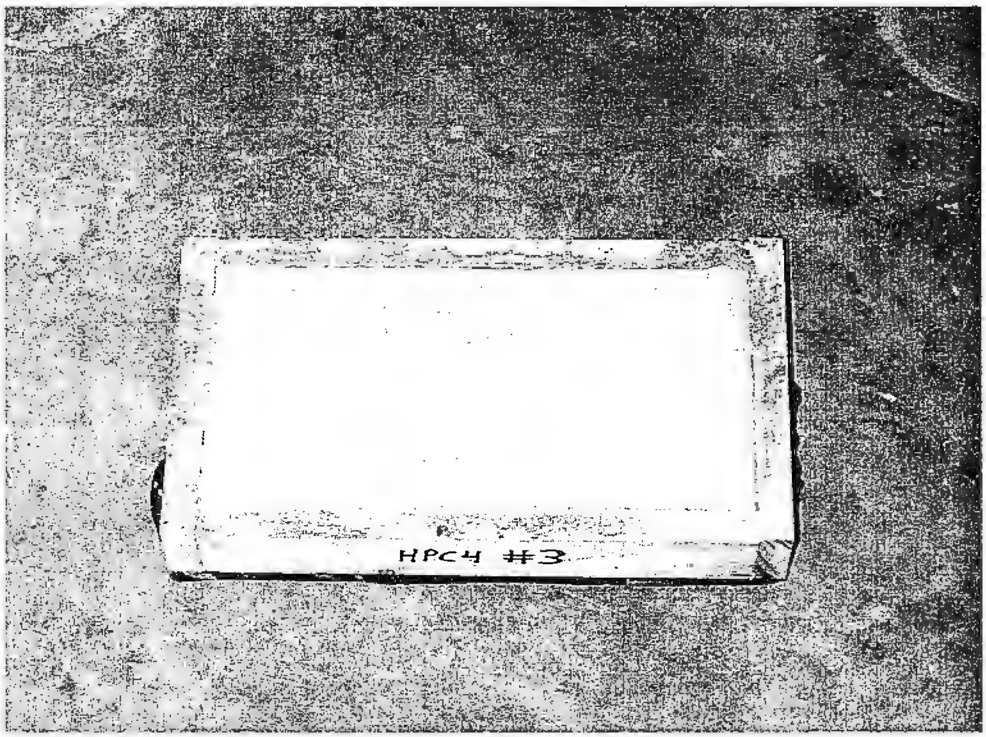


Figure 7.11. The Abrasion Resistance Test Sample

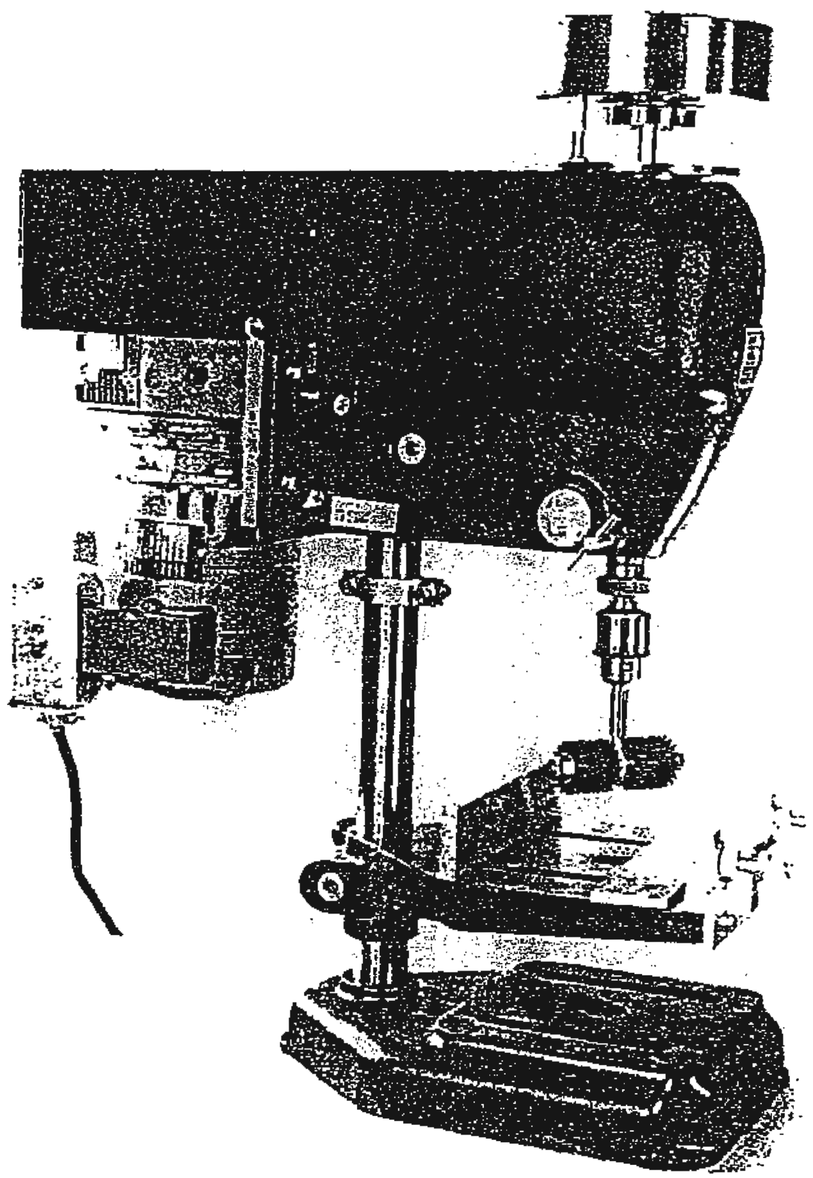


Figure 7.12. Rotating-Cutter Drill Press

specimen. The test result was a loss of 0.00195 oz/in.^{2} (0.00858 grams/cm^2) of abraded surface at 56 days.

Limited test data is available on abrasion resistance results. A comparison between the results obtained in this research and those from a research project in Louisiana on air entrained concrete containing 600 lbs/yd^3 of cement, limestone aggregate and a 0.45 water/cement ratio showed results obtained in this research performed very well. The results of the Louisiana research had an average abrasion loss of 0.00521 oz/in.^{2} (0.0229 grams/cm^2) at 28 days as compared to a loss of 0.00195 oz/in.^{2} (0.00858 grams/cm^2) of abraded surface at 56 days in this research. The concrete in this research averaged 62.6% less mass per unit area loss during abrasion testing than the concrete tested in the Louisiana research. The compressive strength of the concrete tested for abrasion resistance in this research was 6940 psi at 56 days as compared to 4500 psi at 28 days for the concrete tested in the Louisiana research. Compressive strength of concrete is a major factor in the measured abrasion resistance of the concrete. Also, a high quality paste and strong aggregates are important in producing abrasion resistant concrete.

COMPARISON OF HPC AND A STANDARD CONCRETE MIX

Properties of the CIP HPC mix were compared with properties of a standard concrete CIP mix. The properties being compared are shown in Table 7.9. Direct comparisons between the two mixes were made to determine whether the concrete used in production of the CIP HPC superstructures has

Table 7.9. Comparison of HPC and Standard Concrete Properties for the CIP Concrete

Hardened Properties	
Compressive Strength (AASHTO T 22)	28 Days 56 Days 91 Days
Tensile Strength (AASHTO T 198)	28 Days 56 Days
Modulus of Elasticity (ASTM C 469)	28 Days 56 Days
Freeze-thaw Durability (AASHTO T 161)	300 Cycles
Chloride Ion Permeability (AASHTO T 277)	56 Days 91 Days
Shrinkage (ASTM C 157)	90 Days

better strength and durability properties than a standard CIP concrete mix typically used in Alabama.

Specimens used for comparison in strength tests were standard 6 x 12 in. cylinders made in plastic molds. Final curing of the specimens was in a moist room at 70-74°F and 90-95% humidity. Strength tests were conducted in the same manner described already in this chapter. Rapid chloride ion tests were conducted using standard 4 x 8 in. cylinders made in plastic molds as previously described above. Freeze-thaw and shrinkage specimens were also obtained as previously discussed in this chapter.

The standard CIP concrete mix design that was chosen for comparison with the HPC mix is given in Table 7.10. This mix design was used for the CIP superstructure on the 2nd Avenue Bridge in Opelika, Alabama and represented a typical concrete mix that would be used on bridge decks in the state of Alabama. Significant differences in this mix as compared to the HPC mix are that it used gravel instead of limestone, it did not contain any pozzolanic materials, and it had a design strength of 4500 psi as compared to 6000 psi for the HPC mix.

Properties of the HPC mix used for comparison were obtained during casting of the superstructure on the Uphapee Creek Relief Bridge. The HPC-4 mix design of the concrete obtained for comparison was previously given in Table 5.4. Samples chosen for comparison with the standard concrete mix were already presented in this chapter. Sample number three was chosen for

Table 7.10. Mix Proportions Used in the Standard CIP Concrete Mix

Material	Source	Cubic Yard Proportions
Type I Cement	Blue Circle Atlanta	658 lbs
#57 Gravel	Martin Marietta Pinkston	1974 lbs
Sand	Martin Marietta Pinkston	972 lbs
Water	---	291 lbs
Water Reducer	MB Pozzolith® 100-XR	3 oz/cwt
Air Entrainment	MB AE® 90	3 oz

Water/Cement Ratio = 0.44

comparison with the standard CIP mix because it was very representative of the properties obtained during testing of the CIP HPC.

RESULTS OF THE COMPARISON BETWEEN HPC AND A STANDARD CONCRETE MIX

The results from direct comparison testing between the CIP HPC and the standard CIP concrete mix are given in Table 7.11. Compressive strength of the CIP HPC mix was higher at all ages. These comparisons are shown in Figure 7.13. The HPC mix showed continued strength gains beyond 56 days as opposed to the standard concrete mix that did not gain strength beyond 56 days. Continued strength gains of the HPC can be attributed to the pozzolan incorporated into the mix. Values obtained for the splitting tensile strength and modulus of elasticity outperformed the standard concrete mix at all ages. Comparison of both the CIP HPC and standard concrete mix splitting tensile strength values to the ACI 363 (1992) and ACI 318 (1995) empirical values are given in Figure 7.14. Comparison of both the CIP HPC and standard concrete mixes modulus of elasticity values to the ACI 318 (1995) and ACI 363 (1992) empirical values are given in Figure 7.15.

Freeze-thaw durability of the CIP HPC mix was 92.2% as compared to 82.7% for the standard mix after 300 cycles. The individual specimen results are given in Table 7.12. The freeze-thaw performance of both the HPC and standard concrete mix are considered superior based on the definition that Goodspeed et al. (1996) developed for HPC in highway structures. Plots of Durability Factor

Table 7.11. Comparison of HPC and Standard CIP Concrete Properties[1]

Hardened Properties		HPC	Standard Concrete
Compressive Strength (psi)	28-Day 56-Day 91-Day	7220 7440 7870	5040 5750 5600
Tensile Strength (psi)	28-Day 56-Day	530 490	300 400
Modulus of Elasticity (psi)	28-Day 56-Day	5,750,000 5,350,000	4,700,000 4,700,000
Freeze-thaw Durability (Durability Factor, %)	300 Cycles	92.2	82.7
Chloride Ion Permeability (coulombs)	56-Day 91-Day	2770 1960	5570 5540
Shrinkage (microstrain)	90-Day / Moist[2]	330	310

Tests were conducted on standard 6 x 12 in. cylinders made in plastic molds were applicable. Specimens were moist cured until tested.

1. Refer to Appendix A, Table A.23 for individual cylinder test results.
2. Specimens were moist cured for 28 days, then air dried to 90 days.

Table 7.12. Freeze-Thaw Durability for the CIP HPC Mix and a Standard Concrete Mix

Mix	Initial Frequency (Hz)	Final Frequency (Hz)	Durability Factor (%)
HPC	2270	2170	91.4
	2270	2160	90.5
	2260	2200	94.8
			Average = 92.2
Standard Mix	2330	2120	82.8
	2350	2130	82.2
	2280	2080	83.2
			Average = 82.7

Tests were conducted for 300 freeze-thaw cycles with three samples per mix.

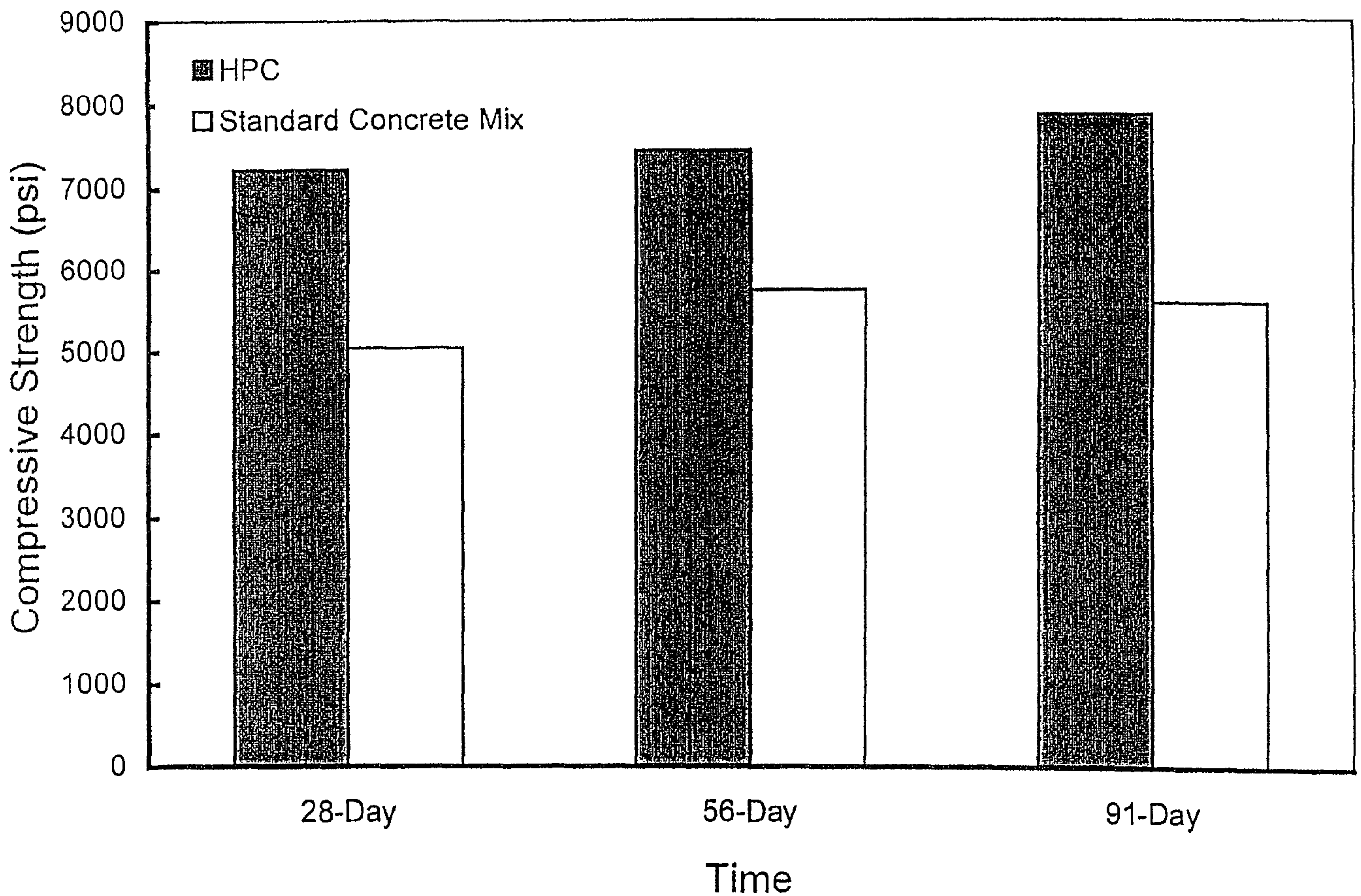


Figure 7.13. Comparison of Compressive Strength for the HPC and the Standard Concrete Mix at 28, 56 and 91 Days

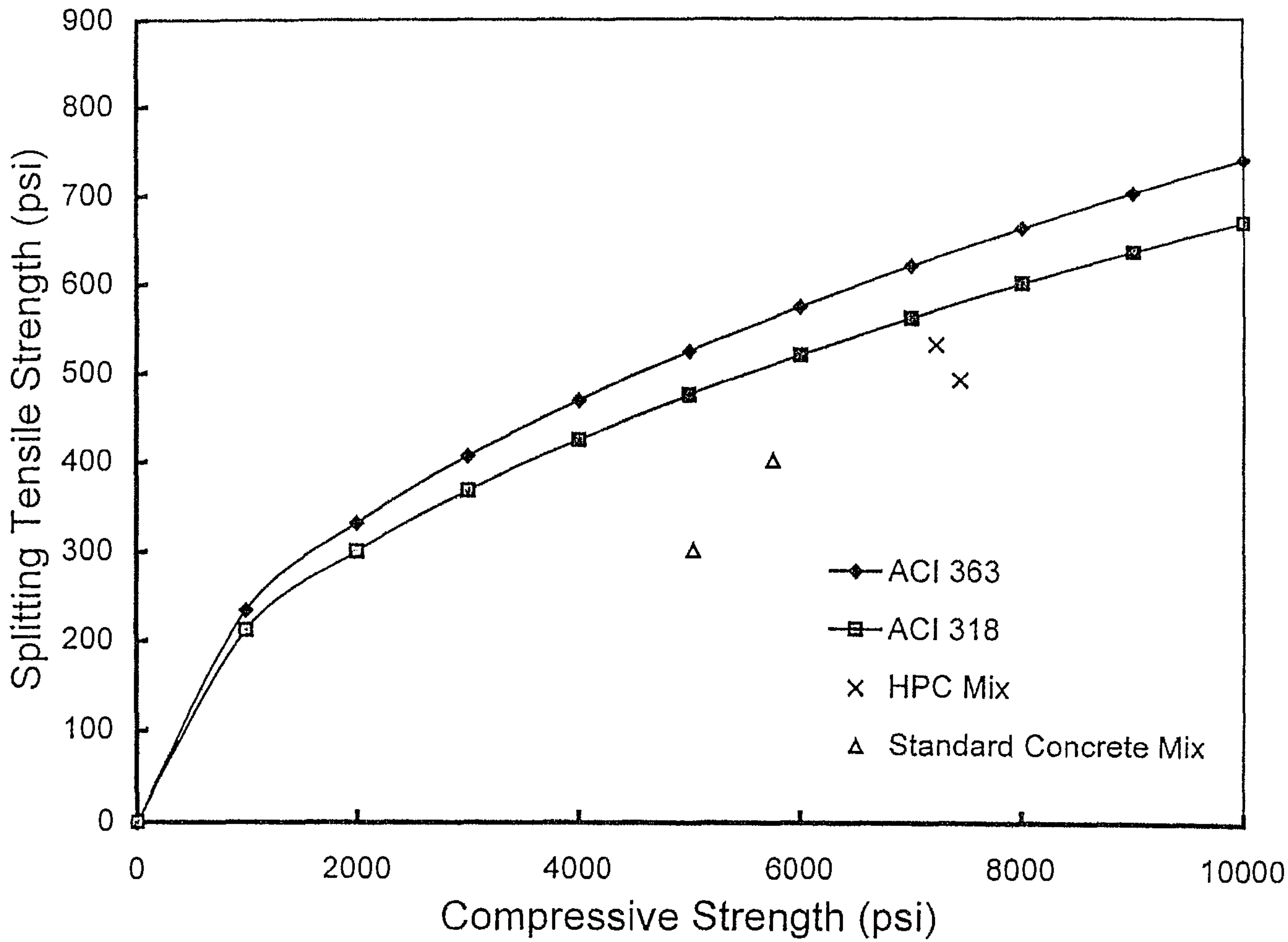


Figure 7.14. Comparison of Splitting Tensile Strength Results for CIP HPC and the Standard Concrete Mix

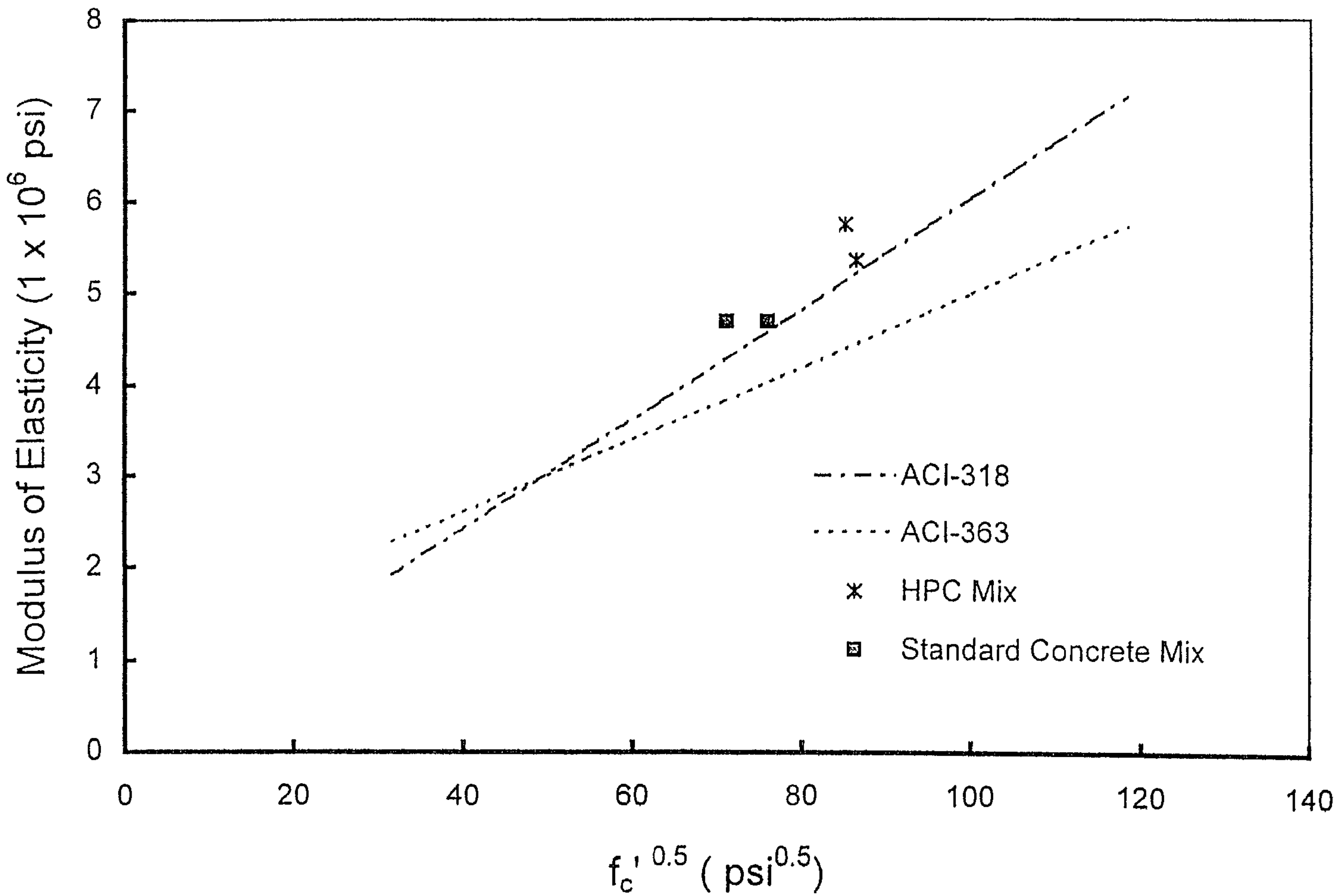


Figure 7.15. Comparison of Modulus of Elasticity Results for CIP HPC and the Standard Concrete Mix

versus Number of Cycles for the CIP HPC and standard concrete mix are given in Figures 7.16 and 7.17 respectively.

Rapid chloride ion permeability of the CIP HPC mix was lower than that of the standard concrete mix. Permeability values obtained for the CIP HPC mix were 2770 coulombs at 56 days versus 5570 coulombs for the standard mix, and 1960 coulombs at 91 days versus 5540 coulombs for the standard mix. At 56 days, the HPC mix fell within the AASHTO T 277 (1997) rating of moderate. At 91 days, the HPC mix improved to a rating of low by AASHTO T 277 (1997) standards. Permeability of the standard concrete mix was considered high at both 56 and 91 days. The lower permeability results obtained by the HPC mix can be attributed to the use of pozzolans in the mix design, as opposed to the standard mix which used only cement.

Shrinkage values obtained at 90 days according to ASTM C 157 (1996) procedures in which the samples were moist cured for 28 days then subjected to air drying were 330 microstrain for the CIP HPC mix and 310 microstrain for the standard concrete mix. A plot of shrinkage versus time for the standard concrete mix and CIP HPC mix is given in Figure 7.18. There was little difference between the drying shrinkage values of the HPC and the standard concrete mix. Even though the HPC mix contained a significantly higher cementitious materials content than the standard concrete mix, the similar drying shrinkage values can be attributed to the increased stiffness of the HPC mix due to the coarse aggregate used. Concretes containing coarse aggregate with a high modulus of

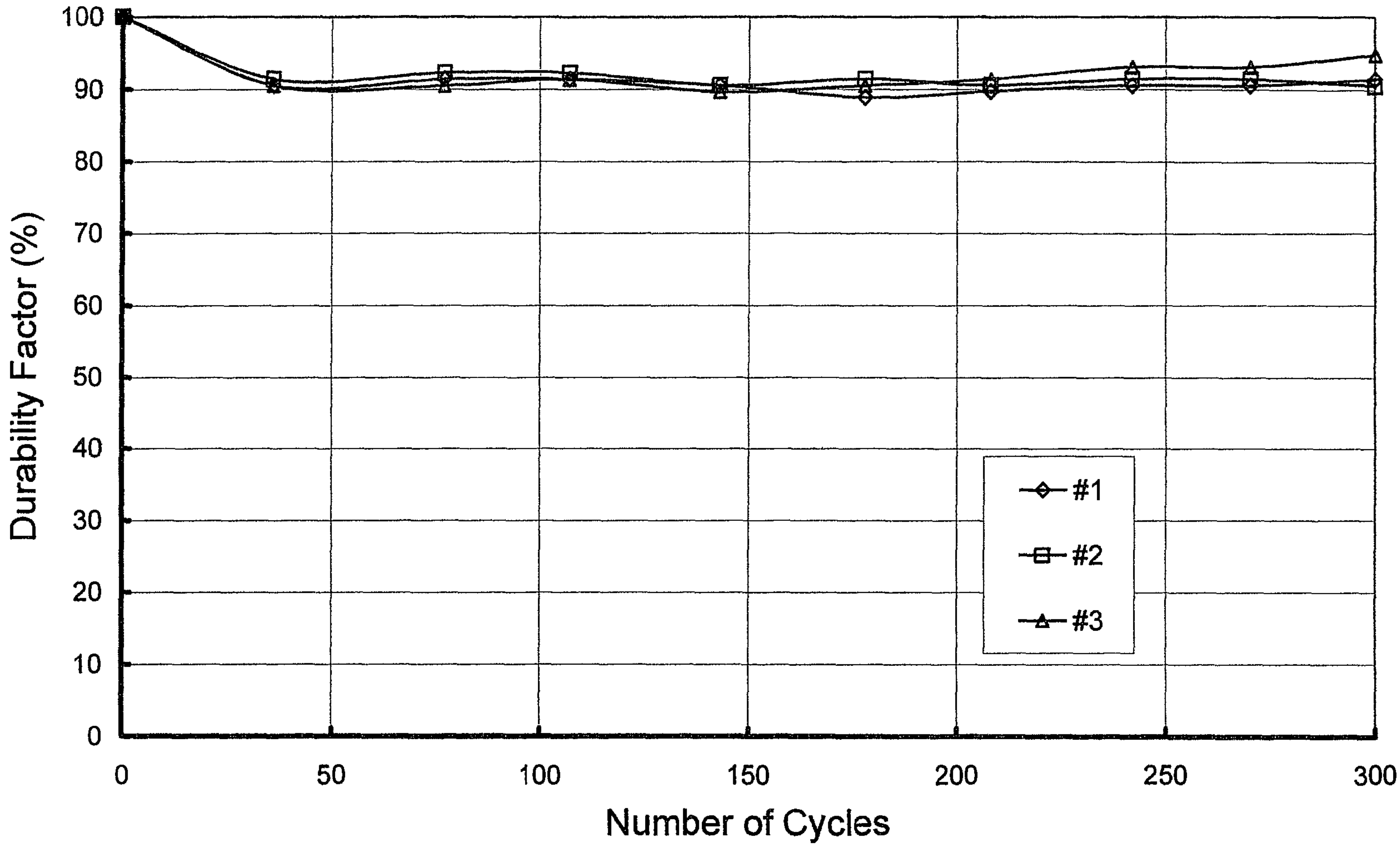


Figure 7.16. Durability Factor for 300 Cycles for the HPC Mix

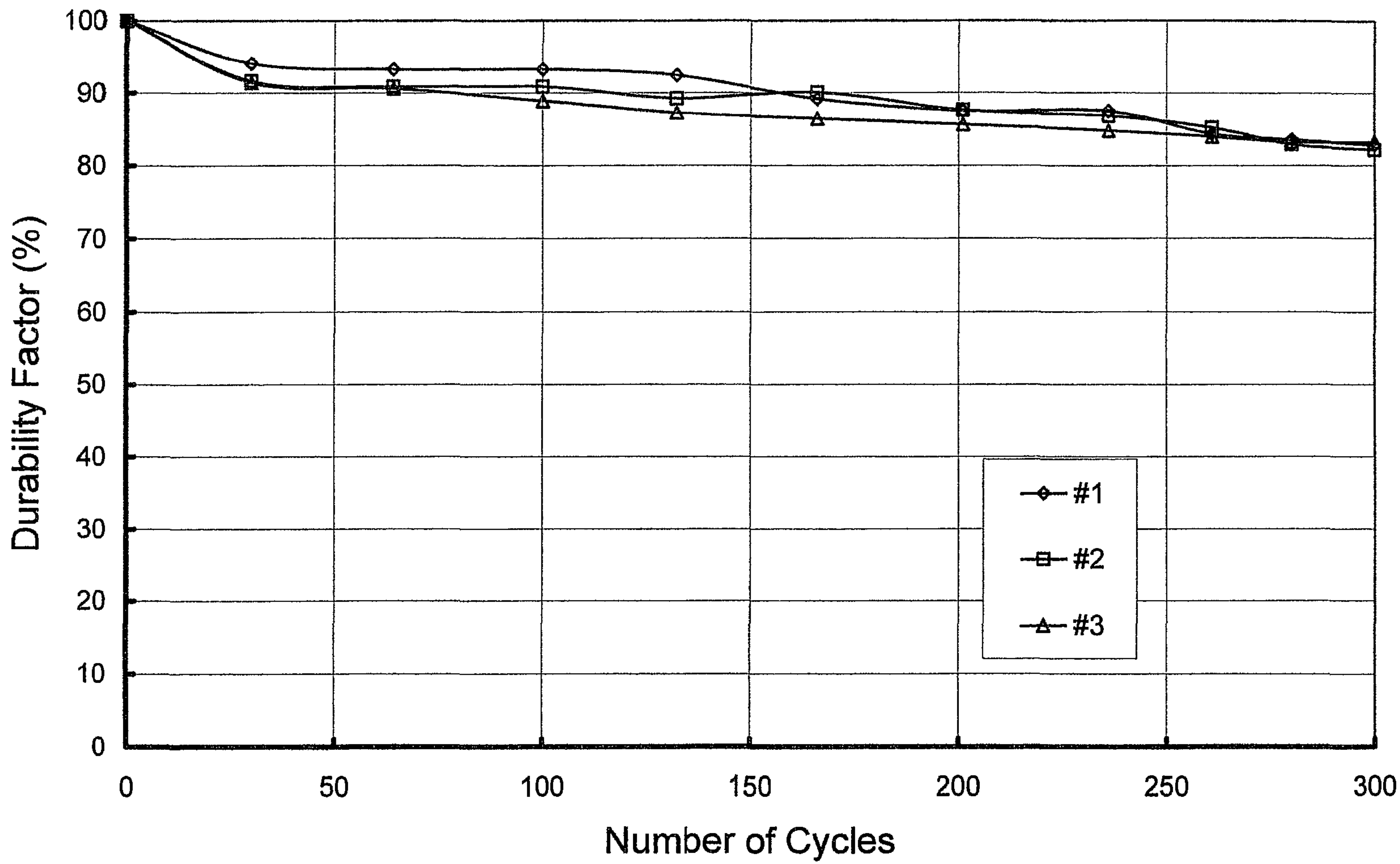


Figure 7.17. Durability Factor for 300 Cycles for the Standard Concrete Mix

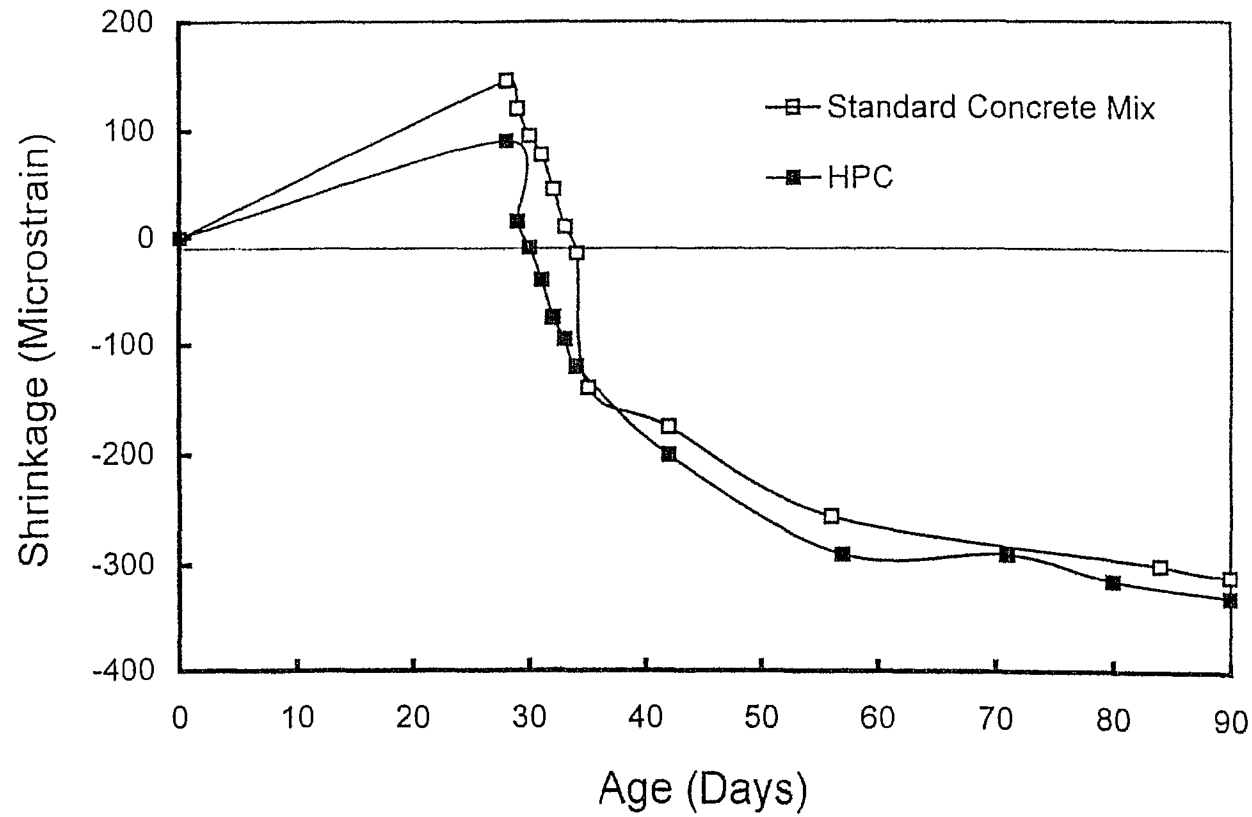


Figure 7.18. Comparison of Drying Shrinkage for Samples Moist Cured for 28 Days then Air Dried

elasticity and rough surface texture are more resistant to drying shrinkage because the coarse aggregate acts to restrain the paste. This was evident here as the HPC mix contained limestone coarse aggregate, and the standard concrete mix used smooth rounded gravel for coarse aggregate.

IS HPC THE PROPER CHOICE?

The question that ultimately arises is whether HPC is a better choice than conventional concrete. Depending on the viewpoint taken, the use of HPC in the production of bridge decks can be argued in many ways. From the standpoint of mechanical properties of a particular concrete, CIP HPC concrete should be considered superior to standard concrete. Results from Tables 7.11 clearly show that the HPC outperformed the standard concrete mix in virtually every comparison. The performance grade criteria used by Goodspeed et al. (1996) to rate HPC was given in Figure 2.1. A comparison of the performance grades received by the standard mix to the average of the CIP HPC samples obtained in this research is given in Table 7.13.

From a designers' point of view, the use of high strength HPC in the design of bridge decks would theoretically allow for larger spacing between girders resulting in fewer girder lines. However, the increased structural capacity of typical decks resulting from higher concrete strength is minimal. The use of HPC on bridge decks with wet curing is likely to provide a more durable and longer lasting deck with fewer cracks resulting in a more aesthetically pleasing structure. With improved durability comes the potential for lower life cycle costs

Table 7.13 Comparison of the Performance Grades[1] Received by the Standard Concrete Mix and HPC Mix

Performance Characteristic		Standard Concrete		HPC	
		Average	Grade	Average	Grade
Compressive Strength (psi)	56-Day	5750	n.a.[2]	7500	1
Tensile Strength (psi)	56-Day	400	n.a.[3]	490	n.a.[3]
Modulus of Elasticity (psi)	56-Day	4,700,000	1	5,750,000	1
Freeze-Thaw Durability (Durability Factor, %)	300 Cycles	82.7	2	92.2	2
Chloride Ion Permeability (coulombs)	56-Day	5570	n.a.[2]	2770	1
Shrinkage (microstrain)[7]	90-Day / Moist	310	3	330	3

Averages of compressive strength, tensile strength, and modulus of elasticity based on standard 6 x 12 in. cylinders made in plastic molds and moist cured until tested.

1. Based on Goodspeed et al. (1996) and Table 1 shown in Figure 2.1.
2. Value outside of range needed to be given a performance grade.
3. Performance grade category not given for splitting tensile strength.

resulting from the use of HPC in bridge decks. For the HPC bridge project reported here, HPC was used in the bridge decks for improved durability characteristics as opposed to increased structural capacity.

The use of cementitious replacement materials especially fly ash, typically provides a cost saving over a cement only mix in obtaining a given strength particularly at later ages, while giving the mix superior strength and durability properties as compared to a mix containing only cement. The improved performance of HPC is evident by the results given in Table 7.11. However, fly ash is not used in HPC to replace cement, and the overall cost of cementitious materials in HPC will generally be higher than for standard concrete. Often associated with the use of HPC on bridge decks is the need for experience using admixtures and pozzolans, better quality control procedures, modern ready-mix trucks, and increased attention to finishing and curing the deck. All of these items can add to the cost of producing HPC bridge decks. The use of HPC in bridge decks will provide improved performance and durability, but the research reported here is insufficient to prove that the improved durability will be worth the additional costs that may be added to future bridges.

CHAPTER EIGHT

EVALUATION OF CONCRETE TESTING AND PRODUCTION PROCEDURES

INTRODUCTION

The methods used to produce concrete, as well as testing procedures utilized to evaluate the strength of concrete can have a significant effect on the measured strength of the concrete. Initial and final curing procedures, cylinder sizes, and methods of capping the test cylinders were studied to assess the effects these parameters had on the measured strength of HPC girder concrete. Various core testing procedures were studied. Internal concrete temperatures of bridge components were monitored during initial curing to study the effects of temperature in mass concrete elements.

CYLINDER SIZE EFFECTS ON COMPRESSIVE STRENGTH

Past research (Cook 1989, Day 1994, Carrasquillo et al. 1981) has indicated that the measured compressive strength of concrete is influenced by the size of the test cylinder used to conduct the test. Previous research has reported a 5% to 10% difference between the strength measured in 4 x 8 in. cylinders and 6 x 12 in. cylinders. Research by Cook indicated that the measured compressive strength of 4 x 8 in. cylinders was approximately five percent higher than that of 6 x 12 in. cylinders. A study by Carrasquillo et al. (1981) concluded that the ratio of 6 x 12 in. cylinder to 4 x 8 in. cylinder was close to 0.90 regardless of the strength of concrete for the ranges tested between 3000 and 11,000 psi. Past research by Burg and Ost indicated that testing machines which meet the dimensional requirements of ASTM C 39 (1996) may produce non-standard load transfer into high-strength 6 x 12 in. cylinders, but can transfer load properly for 4 x 8 in. cylinders (ACI 363 1998). Neville (1996) and others suggest that concrete is composed of elements of variable strength, and the larger the specimen size, the more likely it would be to contain a flaw causing a lower strength test result (Neville 1996).

In this research two series of 4 x 8 in. and 6 x 12 in. cylinders were studied to compare the effects of cylinder size on compressive strength. The first series of test cylinders consisted of 14 sets of cylinders prepared during preliminary mix designs conducted at Auburn University for use in cast-in-place applications. The study consisted of fourteen different mix designs with average 28-day compressive strengths ranging from 5970 psi to 8080 psi. All cylinders in this study were tested on 70 durometer neoprene pads. Compressive strength results are given in Table 8.1. Compressive strength results for 4 x 8 in. cylinders averaged 9.2% higher than 6 x 12 in. cylinders.

A second series of test cylinders were made by Sherman Prestressed Concrete, Auburn University, and the FHWA Mobile Concrete Laboratory during the HPC girder test pour. Comparison tests were conducted on cylinders subjected to the same initial curing and final curing conditions. Initial curing conditions included: cylinders cured under the tarp along with the member, Sure Cure™ match-cured cylinders, and cylinders cured in the contractor's match

Table 8.1. Effect of Cylinder Size on Compressive Strength

Initial Cure	Test Age	Capping Method	Cylinder Size	Compressive Strength (psi)	Difference (%)
Lab	28-Day	Neoprene	4 x 8 in. 6 x 12 in.	7360 6800	8.2
Lab	28-Day	Neoprene	4 x 8 in. 6 x 12 in.	6860 6370	7.7
Lab	28-Day	Neoprene	4 x 8 in. 6 x 12 in.	7560 6680	13.2
Lab	28-Day	Neoprene	4 x 8 in. 6 x 12 in.	7720 7260	6.3
Lab	28-Day	Neoprene	4 x 8 in. 6 x 12 in.	7460 7140	4.5
Lab	28-Day	Neoprene	4 x 8 in. 6 x 12 in.	7560 7160	5.6
Lab	28-Day	Neoprene	4 x 8 in. 6 x 12 in.	8080 7340	10.1
Lab	28-Day	Neoprene	4 x 8 in. 6 x 12 in.	7760 7560	2.6
Lab	28-Day	Neoprene	4 x 8 in. 6 x 12 in.	7360 6500	13.2
Lab	28-Day	Neoprene	4 x 8 in. 6 x 12 in.	7540 6410	17.6
Lab	28-Day	Neoprene	4 x 8 in. 6 x 12 in.	6760 6500	4.0
Lab	28-Day	Neoprene	4 x 8 in. 6 x 12 in.	8060 7120	13.2
Lab	28-Day	Neoprene	4 x 8 in. 6 x 12 in.	6690 5970	12.1
Lab	28-Day	Neoprene	4 x 8 in. 6 x 12 in.	7160 6460	10.8
				Average	9.2

Each test result is the average of two cylinders tested on 70 durometer neoprene pads at Auburn University.

curing box. All test cylinders were cured on the casting yard along with the member after 24 hours.

Compressive strength tests were conducted at release, 7, 14, and 28 days. Results are given in Table 8.2 for cylinders tested on 70 durometer neoprene pads. Cylinders tested on sulfur caps are given in Table 8.3. Compressive strength results for 4 x 8 in. cylinders tested on 70 durometer neoprene pads averaged 10.3% higher than 6 x 12 in. cylinders. For sulfur capped cylinders, 4 x 8 in. cylinders averaged 4.1% higher than 6 x 12 in. cylinders. For the entire study, in which 15 sets of cylinders were subjected to the same initial and final curing conditions, 4 x 8 in. cylinders averaged 7.4% higher strengths than 6 x 12 in. cylinders. These results are similar to past research that reported 4 x 8 in. cylinders broke at loads 5% to 10% higher than 6 x 12 in. cylinders.

A summary of all data tested on 70 durometer neoprene pads from Tables 8.1 and 8.2 is shown in Figure 8.1. This figure shows the relationship between the compressive strength of 4 x 8 in. cylinders and 6 x 12 in. cylinders tested on 70 durometer neoprene pads. An average for all data from Tables 8.1 and 8.2 indicates that the compressive strength determined from 4 x 8 in. cylinders is 9.6% higher than 6 x 12 in. cylinders.

COMPARISON OF CAPPING METHODS

Carrasquillo and Carrasquillo (1988) performed a comparison of cylinder capping methods for concrete compressive strengths between 4000 and 10,000

Table 8.2. Effect of Cylinder Size on Compressive Strength

Initial Cure	Test Age	Capping Method	Cylinder Size	Compressive Strength (psi)	Difference (%)
Under Tarp	Release	Neoprene	4 x 8 in. 6 x 12 in.	10060 9010	11.7
Under Tarp	14-Day	Neoprene	4 x 8 in. 6 x 12 in.	12210 11090	10.1
Under Tarp	28-Day	Neoprene	4 x 8 in. 6 x 12 in.	12150 10750	13.0
Sure Cure™	Release	Neoprene	4 x 8 in. 6 x 12 in.	9970 8520	17.0
Sure Cure™	28-Day	Neoprene	4 x 8 in. 6 x 12 in.	11550 10390	11.2
CMC	Release	Neoprene	4 x 8 in. 6 x 12 in.	10170 9500	7.1
CMC	14-Day	Neoprene	4 x 8 in. 6 x 12 in.	11320 10370	9.2
CMC	28-Day	Neoprene	4 x 8 in. 6 x 12 in.	11420 11120	2.7
				Average	10.3

Under Tarp – Cylinders cured under tarp with member. Cylinders made and tested by Sherman Prestress. Results are the average of two cylinders.

Sure Cure™ - Sure Cure™ match-cure system. Cylinders tested in FHWA Mobile Concrete Laboratory. Results are the average of three cylinders.

CMC – Contractor's match-cure system. Cylinders made and tested by Sherman Prestressed Concrete. Results are the average of two cylinders.

Table 8.3. Effect of Cylinder Size on Compressive Strength

Initial Cure	Test Age	Capping Method	Cylinder Size	Compressive Strength (psi)	Difference (%)
Under Tarp	Release	Sulfur	4 x 8 in.	9210	9.0
			6 x 12 in.	8380	
Under Tarp	14-Day	Sulfur	4 x 8 in.	10420	1.8
			6 x 12 in.	10240	
Under Tarp	28-Day	Sulfur	4 x 8 in.	11640	10.3
			6 x 12 in.	10550	
CMC	Release	Sulfur	4 x 8 in.	8990	-0.3
			6 x 12 in.	9020	
CMC	7-Day	Sulfur	4 x 8 in.	10340	4.0
			6 x 12 in.	9940	
CMC	14-Day	Sulfur	4 x 8 in.	10340	0.4
			6 x 12 in.	10300	
CMC	28-Day	Sulfur	4 x 8 in.	10940	3.4
			6 x 12 in.	10580	
				Average	4.1

Results are the average of two cylinders tested on sulfur caps.

Under Tarp – Cylinders cured under tarp with member. Cylinders made and tested by Sherman Prestressed Concrete.

CMC – Contractor's match-cure system. Cylinders made and tested by Sherman Prestressed Concrete.

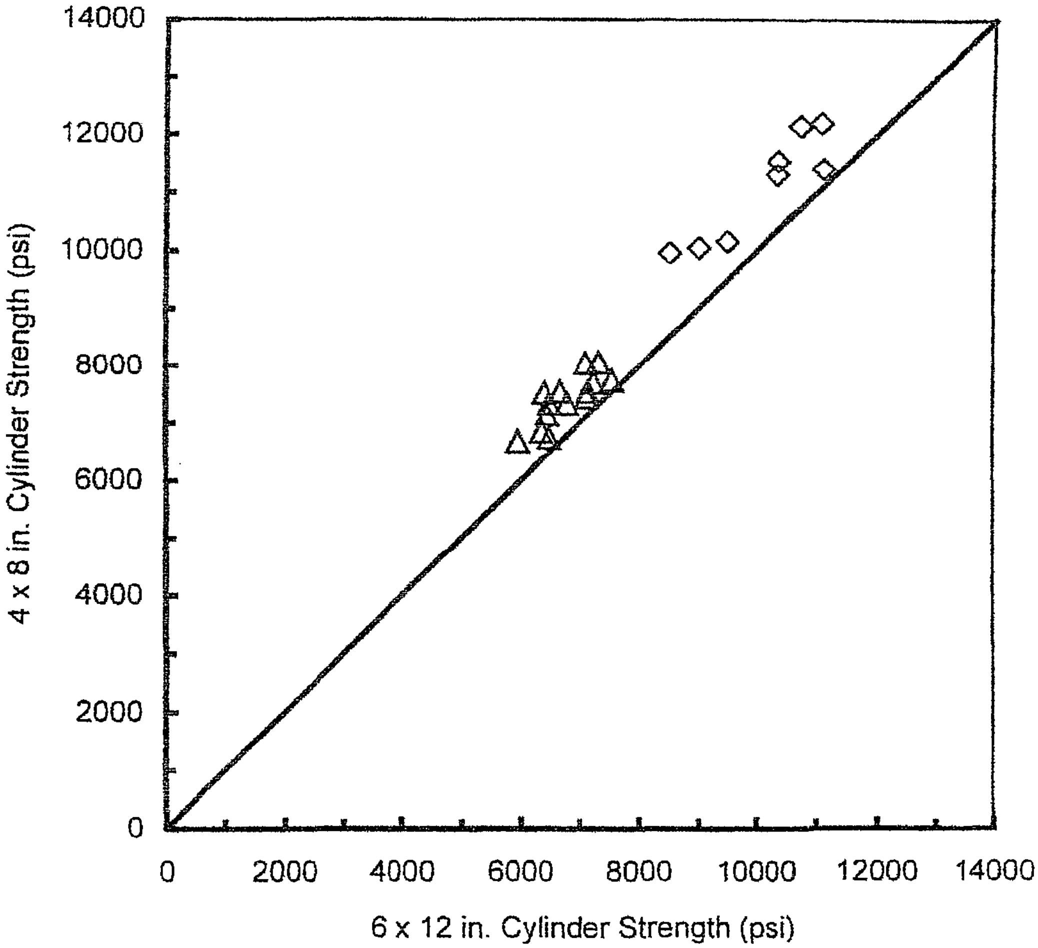


Figure 8.1. Relationship Between the Compressive Strength of 4 x 8 in. and 6 x 12 in. Cylinders Tested on 70 Durometer Neoprene Pads

psi. The comparison indicated that the use of polyurethane inserts with aluminum restraining rings provided compressive strength results within 5% of those found using sulfur mortar caps. The study concluded that use of unbonded caps could provide a cleaner, safer, and more cost-effective alternative to sulfur mortar capping of concrete cylinders.

In this research a series of 4 x 8 in. and 6 x 12 in. cylinders were studied to evaluate the influence of capping method on measured compressive strength. Cylinders were tested using a pad cap system with 70 durometer neoprene pads and capped using high strength sulfur capping compound. Test cylinders were made by Sherman Prestressed Concrete during the HPC girder test pour. Comparison tests were conducted on cylinders subjected to the same initial curing and final curing conditions. Initial curing conditions included: cylinders cured under the tarp along with the member and cylinders cured in the contractor's match curing box. All test cylinders were cured on the casting yard along with the member after the initial curing achieved the required release strength. Compressive strength tests were conducted at release, 14, and 28 days. Compressive strength results for 4 x 8 in. and 6 x 12 in. cylinders are given in Tables 8.4 and 8.5, respectively. Compressive strength results of 4 x 8 in. cylinders indicated that the cylinders tested on neoprene pads broke up to 9.6% higher than the sulfur capped cylinders. Results for 6 x 12 in. indicated that cylinders tested on neoprene pads broke up to 4.8% higher than the sulfur capped cylinders.

Table 8.4. Effect of Capping Method on Compressive Strength

Initial Cure	Test Age	Cylinder Size	Capping Method	Compressive Strength (psi)	Difference (%)
Under Tarp	Release	4 x 8 in.	Neoprene	10060	9.2
			Sulfur	9210	
Under Tarp	14-Day	4 x 8 in.	Neoprene	12210	17.1
			Sulfur	10430	
Under Tarp	28-Day	4 x 8 in.	Neoprene	12150	4.4
			Sulfur	11640	
CMC	Release	4 x 8 in.	Neoprene	10170	13.1
			Sulfur	8990	
CMC	14-Day	4 x 8 in.	Neoprene	11320	9.5
			Sulfur	10340	
CMC	28-Day	4 x 8 in.	Neoprene	11420	4.4
			Sulfur	10940	
				Average	9.6

Results are the average of two cylinders.

Under Tarp – Cylinders cured under tarp with member. Cylinders made and tested by Sherman Prestressed Concrete.

CMC – Contractor's match-cure system. Cylinders made and tested by Sherman Prestressed Concrete.

Table 8.5. Effect of Capping Method on Compressive Strength

Initial Cure	Test Age	Cylinder Size	Capping Method	Compressive Strength (psi)	Difference (%)
Under Tarp	Release	6 x 12 in.	Neoprene	9010	7.4
			Sulfur	8390	
Under Tarp	14-Day	6 x 12 in.	Neoprene	11090	8.3
			Sulfur	10240	
Under Tarp	28-Day	6 x 12 in.	Neoprene	10750	1.9
			Sulfur	10550	
CMC	Release	6 x 12 in.	Neoprene	9500	5.3
			Sulfur	9020	
CMC	14-Day	6 x 12 in.	Neoprene	10370	0.7
			Sulfur	10300	
CMC	28-Day	6 x 12 in.	Neoprene	11120	5.1
			Sulfur	10580	
				Average	4.8

Results are the average of two cylinders.

Under Tarp – Cylinders cured under tarp with member. Cylinders made and tested by Sherman Prestressed Concrete.

CMC – Contractor's match-cure system. Cylinders made and tested by Sherman Prestressed Concrete.

Another study was conducted in which the compressive strength of Sure Cure™ match-cured cylinders tested on sulfur caps by the Contractor during production of the HPC girders were compared with the results of Sure Cure™ match-cured cylinders tested on 70 durometer pads by Auburn University. Results are given in Tables 8.6 and 8.7. These results indicated that 4 x 8 in. cylinders tested on neoprene pads broke up to 8.2% higher than those tested on sulfur caps. Since the tests using sulfur capping were performed on a different testing machine than the tests using neoprene pads, some of the 8.2% difference could be due to the testing machines. A direct comparison of six data sets is given in Table 8.8 to identify the effect of the differences in testing machines. This limited data suggests that the compressive strength of 4 x 8 in. cylinders tested at Auburn University were 1.3% higher than the strengths determined at Sherman Prestressed Concrete. Hence the difference between cylinder strengths resulting from the use of neoprene pads instead of sulfur capping for the data of Tables 8.6 and 8.7 is approximately 7%.

A summary of all data from Tables 8.4 through 8.7 is shown in Figure 8.2. An average determined for all data indicates that the compressive strengths determined using 70 durometer neoprene pads is 7.9% higher than using sulfur capping. A clear explanation of this difference is not available, but it is believed to be the result of exceeding the strength of the sulfur capping compound. The compressive strength results obtained using neoprene pads are believed to be a better indication of the concrete strength than those obtained using sulfur capping.

Table 8.6. Effect of Capping Method on Compressive Strength

Pour Number	Test Age	Cylinder Size	Capping Method	Compressive Strength (psi)	Difference (%)
1	Release	4 x 8 in.	Neoprene Sulfur	8940 8080	10.6
1	28-Day	4 x 8 in.	Neoprene Sulfur	10050 9890	1.6
1	56-Day	4 x 8 in.	Neoprene Sulfur	11280 10110	11.6
3	56-Day	4 x 8 in.	Neoprene Sulfur	10750 10720	0.3
5	28-Day	4 x 8 in.	Neoprene Sulfur	11000 10060	9.3
5	56-Day	4 x 8 in.	Neoprene Sulfur	10850 10360	4.7
6	56-Day	4 x 8 in.	Neoprene Sulfur	9270 8440	9.8
8	Release	4 x 8 in.	Neoprene Sulfur	8950 8830	1.4
9	56-Day	4 x 8 in.	Neoprene Sulfur	9990 9550	4.6
10	28-Day	4 x 8 in.	Neoprene Sulfur	11090 9830	14.7
10	56-Day	4 x 8 in.	Neoprene Sulfur	10350 9670	7.0
11	28-Day	4 x 8 in.	Neoprene Sulfur	10550 10030	5.2
11	56-Day	4 x 8 in.	Neoprene Sulfur	10250 9710	5.6
12	28-Day	4 x 8 in.	Neoprene Sulfur	9950 8950	11.2
12	56-Day	4 x 8 in.	Neoprene Sulfur	10150 8620	17.8

Table 8.7. Effect of Capping Method on Compressive Strength

Pour Number	Test Age	Cylinder Size	Capping Method	Compressive Strength (psi)	Difference (%)
13	28-Day	4 x 8 in.	Neoprene Sulfur	11270 10820	4.2
13	56-Day	4 x 8 in.	Neoprene Sulfur	11350 10320	10.0
14	28-Day	4 x 8 in.	Neoprene Sulfur	10760 10250	5.0
14	56-Day	4 x 8 in.	Neoprene Sulfur	11060 8980	23.2
15	28-Day	4 x 8 in.	Neoprene Sulfur	12300 11060	11.2
15	56-Day	4 x 8 in.	Neoprene Sulfur	12230 11320	8.0
16	28-Day	4 x 8 in.	Neoprene Sulfur	11620 10260	13.3
16	56-Day	4 x 8 in.	Neoprene Sulfur	11540 10540	9.5
17	28-Day	4 x 8 in.	Neoprene Sulfur	11880 10900	9.0
17	56-Day	4 x 8 in.	Neoprene Sulfur	12190 11380	7.1
18	Release	4 x 8 in.	Neoprene Sulfur	9950 9810	1.4
18	28-Day	4 x 8 in.	Neoprene Sulfur	11320 10660	6.2
18	56-Day	4 x 8 in.	Neoprene Sulfur	12370 11600	6.6
				Average	8.2

Results are from 4 x 8 in. Sure Cure™ cylinders made during production of HPC girders.
Cylinders tested on sulfur caps were made and tested by Sherman Prestressed Concrete and results are the average of two cylinders.
Cylinders tested on neoprene pads were made and tested by Auburn University and results are the average of two or three cylinders.

Table 8.8. Comparison of Testing Machines on 4 x 8 in. Cylinder Compressive Strength

Pour Number	Test Age	Capping Method	Testing Machine	Compressive Strength (psi)	Difference (%)
9	56-Day	Neoprene	Auburn Sherman	10960 10540	4.0
10	28-Day	Neoprene	Auburn Sherman	10540 9910	6.4
10	56-Day	Neoprene	Auburn Sherman	10250 10380	-1.3
14	56-Day	Neoprene	Auburn Sherman	11880 11850	0.3
16	28-Day	Neoprene	Auburn Sherman	10960 11380	-3.7
17	56-Day	Neoprene	Auburn Sherman	11770 11540	2.0
				Average	1.3

All cylinders were standard plastic molded 4 x 8 in. test cylinders cured under the tarp along with the member.

Results are the average of two cylinders.

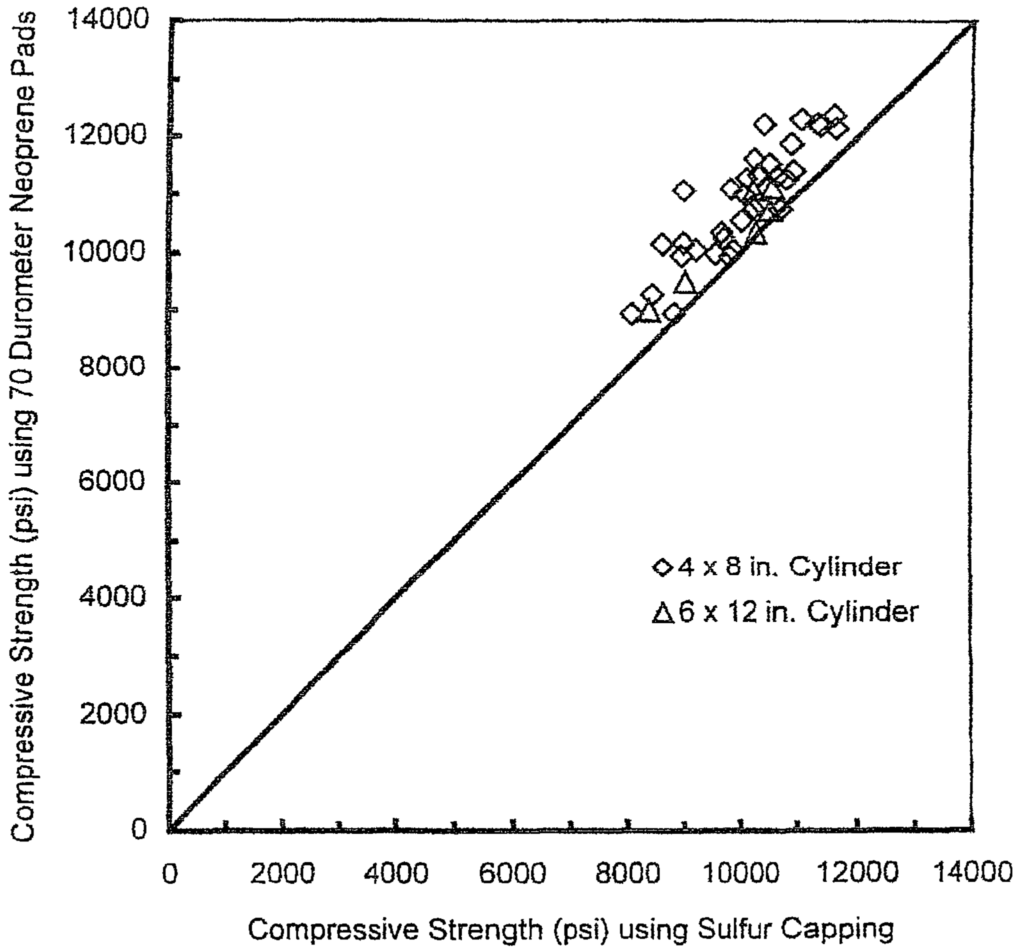


Figure 8.2. Relationship Between the Compressive Strength of Cylinders Tested on 70 Durometer Neoprene Pads and Sulfur Capping

COMPARISON OF INITIAL CYLINDER CURING METHODS IN PRECAST APPLICATIONS

Past research (Myers and Carrasquillo 1997) and trial mixes by Auburn University have indicated that early age compressive strength at release and long-term compressive strength at 28 and/or 56 days is strongly influenced by the temperature history during curing. Research by Myers and Carrasquillo (1997) indicated that match-cured cylinders tend to have a higher strength at release and a lower strength at 28 and 56 days than do cylinders cured under the insulated tarp used to cover the girder formwork prior to release of the prestressing force.

Typical initial curing temperature histories of 4 x 8 in. Sure Cure™ match-cured cylinders and 4 x 8 in. cylinders cured under the tarp along with girders for this project are shown in Figures 8.3 and 8.4. Compressive strength results are presented in Tables 8.9 and 8.10 to illustrate the differences between Sure Cure™ match-cured cylinders and cylinders cured under the tarp. Figures 8.5 and 8.6 compare both curing methods influence on compressive strength at 28 and 56 days respectively.

Comparison of compressive strengths shows that Sure Cure™ match-cured cylinders consistently reached higher compressive strengths at release than cylinders cured under the tarp along with the member by an average of 10.7%. These results agree with past research findings on early age strength gain in steam-cured precast concrete members (Myers and Carrasquillo 1997). Comparison of compressive strengths at 28 days resulted in Sure Cure™ match-cured cylinders averaging 4.9% higher than cylinders cured under the tarp along with the member. At 56 days, very little difference existed between the two curing methods, with the difference between the two methods of curing being less than 0.5%. Figures 8.5 and 8.6 show the strength of the match-cured cylinders are not always higher or lower than the strength of the cylinders cured under the tarp. These compressive strength results for 28 and 56 days do not agree with other research findings (Myers and Carrasquillo 1997) which indicate that match cured cylinder strengths are lower than those for cylinders cured under the tarp. The reason for the difference in trends observed is believed to result primarily from differences in the curing temperature histories. The Sure Cure™ match cured cylinders in this project were not exposed to peak temperatures above 160° F during the initial curing process, while those in the research conducted by Myers and Carrasquillo (1997) were cured at peak temperatures above 190° F. Hence in the project reported by Myers and Carrasquillo (1997) it is likely that the match cured cylinders reached temperatures much higher than the temperatures of the cylinders cured under the tarp. In the project reported here, because of the specified limit of 160° F on the member curing temperature, the difference in peak curing temperatures for the two types of cylinders was not as great.

CYLINDER FINAL CURING METHODS IN PRECAST APPLICATIONS

Two series of 4 x 8 in. cylinders were studied to compare the effects of final curing methods on compressive strength at various ages. The first series of test cylinders were made by Auburn University during the HPC girder test pour.

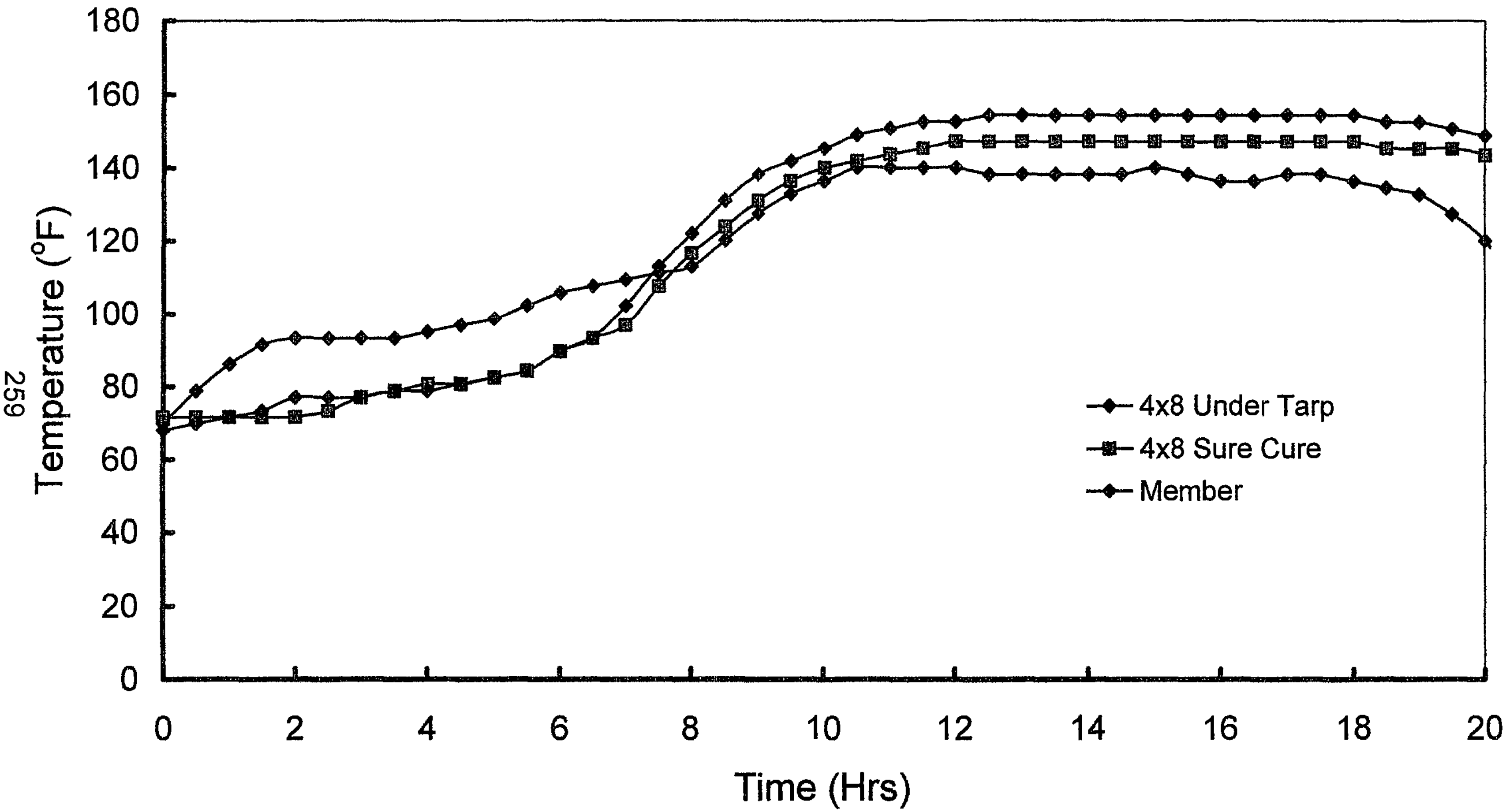


Figure 8.3. Comparison of Sure Cure™ Match-Cured Cylinders and Cylinders Cured Under the Tarp from Pour 10

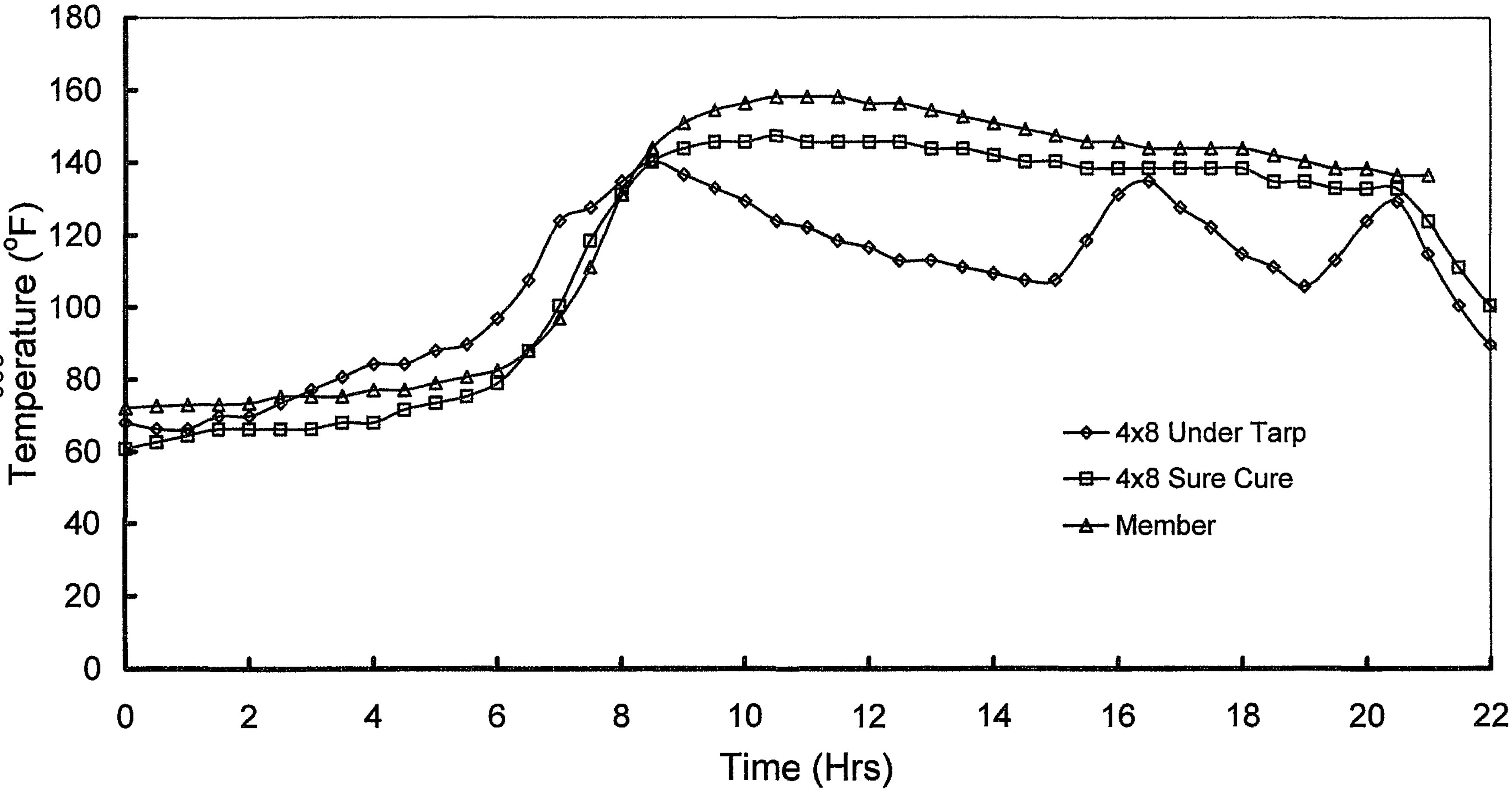


Figure 8.4. Comparison of Sure Cure™ Match-Cure Cylinders and Cylinders Cured Under the Tarp from Creep Sample Pour

Table 8.9. Effect of Initial Curing Method on Compressive Strength

Pour Number	Test Age	Initial Cure	Final Cure	Compressive Strength (psi)	Difference (%)
1	Release	Sure Cure™ Under Tarp	-	8940 8100	10.4
1	7-Day	Sure Cure™ Under Tarp	Air	9780 9130	7.1
1	28-Day	Sure Cure™ Under Tarp	Air	10050 9750	3.1
1	56-Day	Sure Cure™ Under Tarp	Air	11280 10100	11.7
3	56-Day	Sure Cure™ Under Tarp	Air	10750 12290	-12.5
5	28-Day	Sure Cure™ Under Tarp	Air	11000 10460	5.2
5	56-Day	Sure Cure™ Under Tarp	Air	10850 10240	6.0
6	56-Day	Sure Cure™ Under Tarp	Air	9270 10410	-11.0
8	Release	Sure Cure™ Under Tarp	-	8950 7480	19.7
9	56-Day	Sure Cure™ Under Tarp	Air	9990 10540	-5.2
10	28-Day	Sure Cure™ Under Tarp	Air	11090 9910	11.9
10	56-Day	Sure Cure™ Under Tarp	Air	10350 10380	-0.3
11	28-Day	Sure Cure™ Under Tarp	Air	10550 9930	6.2
12	28-Day	Sure Cure™ Under Tarp	Air	9950 8990	10.7

Table 8.10. Effect of Initial Curing Method on Compressive Strength

Pour Number	Test Age	Initial Cure	Final Cure	Compressive Strength (psi)	Difference (%)
13	28-Day	Sure Cure™ Under Tarp	Air	11270 10960	2.8
13	56-Day	Sure Cure™ Under Tarp	Air	11350 11580	-2.0
14	28-Day	Sure Cure™ Under Tarp	Air	10760 10920	-1.5
14	56-Day	Sure Cure™ Under Tarp	Air	11060 11850	-6.7
15	7-Day	Sure Cure™ Under Tarp	Air	11110 9810	13.3
15	28-Day	Sure Cure™ Under Tarp	Air	12300 11600	6.0
15	56-Day	Sure Cure™ Under Tarp	Air	12230 11620	5.3
16	28-Day	Sure Cure™ Under Tarp	Air	11620 11380	2.1
16	56-Day	Sure Cure™ Under Tarp	Air	11540 11400	1.2
17	7-Day	Sure Cure™ Under Tarp	Air	12280 10300	19.2
17	28-Day	Sure Cure™ Under Tarp	Air	11880 11080	7.2
17	56-Day	Sure Cure™ Under Tarp	Air	12190 11540	5.6
18	Release	Sure Cure™ Under Tarp	-	9950 9750	2.1
18	28-Day	Sure Cure™ Under Tarp	Air	11320 11290	0.3
18	56-Day	Sure Cure™ Under Tarp	Air	12370 12010	3.0

Compressive strength results are from 4 x 8 in. cylinders obtained by Auburn University during production of the HPC girders. Individual cylinders break results are given in Appendix A, Tables A.16 through A.18.

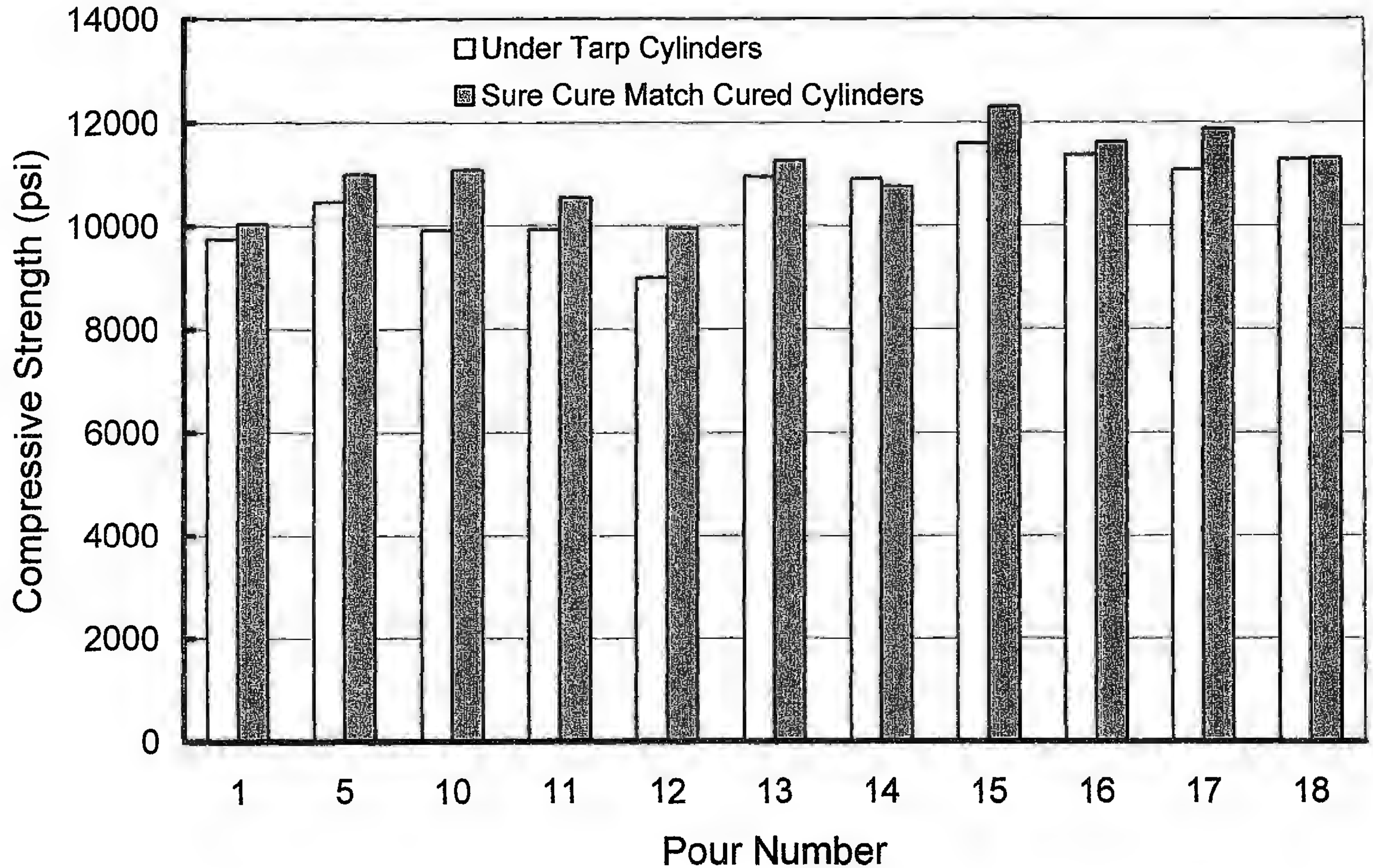


Figure 8.5. Compressive Strength at 28 Days from Varied Curing Conditions

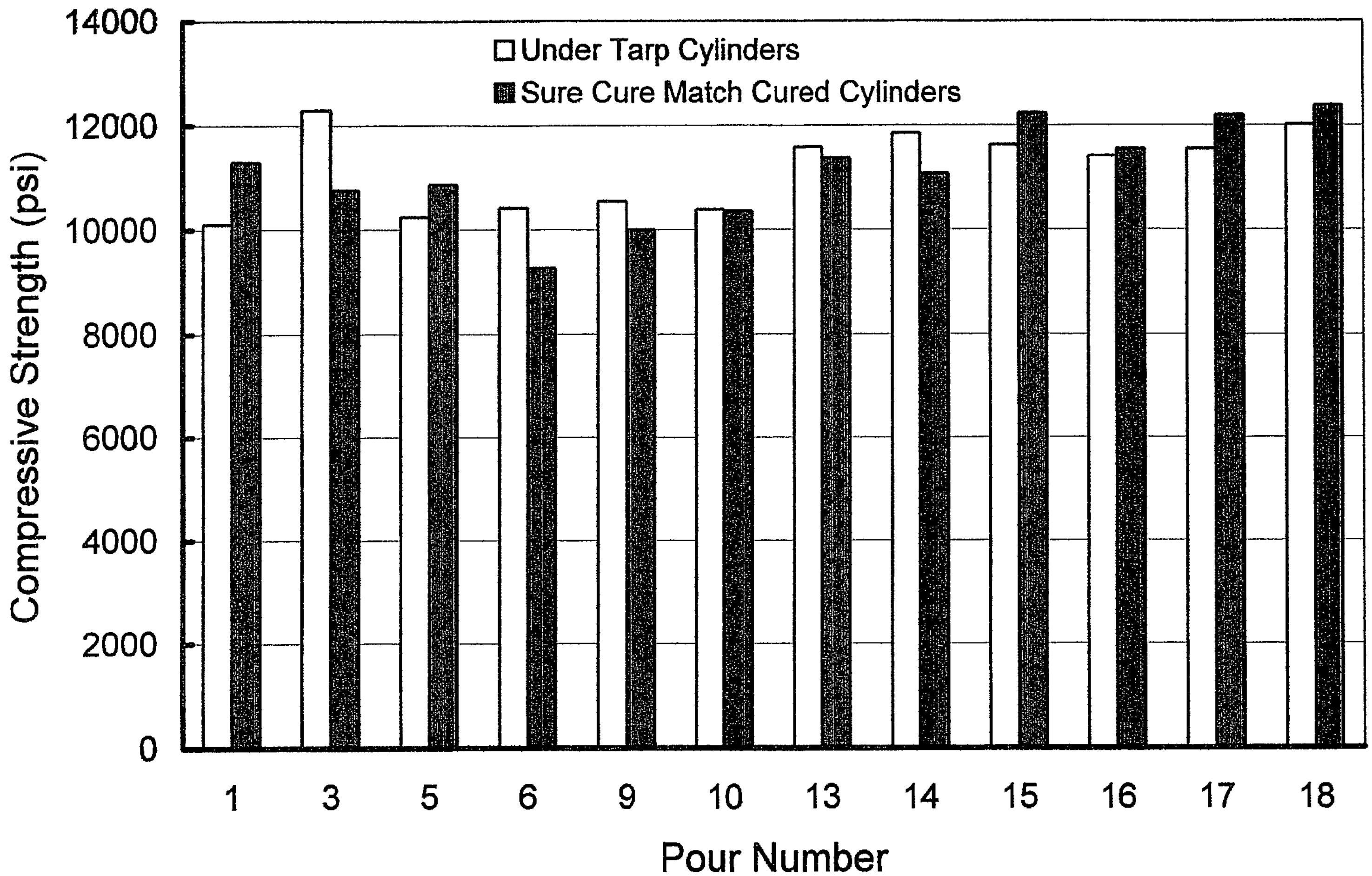


Figure 8.6. Compressive Strength at 56 Days from Varied Curing Conditions

Standard 4 x 8 in. cylinders in plastic molds were initially cured under the tarp along with the member. After the release strength was achieved, test cylinders were subjected to one of the following conditions: (1) cured on the casting yard along with the member, (2) cured in a limebath at 70-74° F, or (3) cured in a moist room that was maintained at 70-74° F and 90-95% humidity. Compressive strength tests were conducted at 7, 28, and 56 days. Results previously presented in Table 4.6 are given again in Tables 8.11 and 8.12. Results indicate that cylinders subjected to wet curing showed improved strength gains at 28 days and even better gains at 56 days as compared to cylinders cured on the casting yard along with the member. Strengths of cylinders cured in a limebath averaged 7.4% higher at 56 days than strengths of cylinders cured on the yard along with the member. Strength of moist-cured cylinders averaged 9.8% higher at 56 days than strengths of cylinders cured on the yard along with the member. These results agree with research findings by Berry and Malhotra (1980) who reported that after the rate of strength gain of portland cement slows, the continued pozzolanic activity of fly ash contributes to increased strength gain at later ages if the concrete is kept moist. The higher strengths achieved by limebath and moist curing of the test cylinders were expected because of the 15% fly ash that was incorporated into the mix design.

Another series of comparisons was conducted to determine the effect of final curing condition on compressive strength for a series of 4 x 8 in. Sure Cure™ match-cured cylinders made during production of the HPC girders. After the release strength was achieved, test cylinders were subjected to one of the

Table 8.11. Effect of Final Curing Method on Compressive Strength

Pour Number	Test Age	Initial Cure	Final Cure	Compressive Strength (psi)	Difference (%)
Test Pour	7	Under Tarp	Limebath Air	11220 11670	-3.9
Test Pour	28	Under Tarp	Limebath Air	11490 11110	3.4
Test Pour	56	Under Tarp	Limebath Air	12630 11760	7.4

Limebath – cylinders cured in a limebath at 70-74° F.
Air – cylinders cured on the casting yard along with member.

Results are the average of two cylinders.

Table 8.12. Effect of Final Curing Method on Compressive Strength

Pour Number	Test Age	Initial Cure	Final Cure	Compressive Strength (psi)	Difference (%)
Test Pour	7	Under Tarp	Moist Air	10860 11670	-6.9
Test Pour	28	Under Tarp	Moist Air	11510 11110	3.6
Test Pour	56	Under Tarp	Moist Air	12910 11760	9.8

Moist – cylinders cured in a moist room at 70-74° F and 90-95% humidity.
Air – cylinders cured on the casting yard along with member.

Results are the average of two cylinders.

following final conditions: (1) cured on the casting yard along with the member, (2) cured indoors at room temperature, or (3) cured in a limebath at 70-74° F. Comparisons of compressive strengths resulting from the various final curing conditions are given in Tables 8.13 and 8.14. Compressive strengths of cylinders cured in a limebath averaged 12.2% and 16.3% higher at 28 and 56 days, respectively, than the strength of cylinders cured on the casting yard along with the member. These results agree with previous research findings.

Strengths of cylinders cured on the casting yard along with the member averaged 5.3% and 0.1% higher at 28 and 56 days, respectively, than strengths of cylinders cured indoors at room temperature. The slightly higher strength of cylinders cured on the casting yard compared to indoors is believed to be the result of moisture from rain coupled with moderate temperatures in the spring months for which the cylinders were on the casting yard.

CORE TESTING PROCEDURES

In this research, three different core testing procedures were followed to provide information on testing methods that most accurately represented the compressive strength of the concrete member in question. The core testing plan was previously presented in Table 4.3. Two of the testing procedures were based on current AASHTO (1997) and ACI 318-95 (1995) procedures. The procedure recommended by AASHTO T 24 (1997) requires cores to be submerged in a limebath for at least 40 hours immediately prior to testing, and requires testing of the core specimens while wet. The procedure recommended

Table 8.13. Effect of Final Curing Method on Compressive Strength

Pour Number	Test Age	Initial Cure	Final Cure	Compressive Strength (psi)	Difference (%)
14	28-Day	Sure Cure™	Limebath Air	12070 10760	12.2
14	56-Day	Sure Cure™	Limebath Air	12640 11060	14.3
16	28-Day	Sure Cure™	Limebath Air	13030 11620	12.1
16	56-Day	Sure Cure™	Limebath Air	13650 11540	18.3
				28-Day Average	12.2
				56-Day Average	16.3

Results are from 4 x 8 in. Sure Cure™ cylinders made during production of HPC girders and are the average of two cylinders.

Limebath – cylinders cured in a limebath at 70-74° F.
Air – cylinders cured on the casting yard along with member.

Table 8.14. Effect of Final Curing Method on Compressive Strength

Pour Number	Test Age	Initial Cure	Final Cure	Compressive Strength (psi)	Difference (%)
11	28-Day	Sure Cure™	Air Indoors	10550 10410	1.3
11	56-Day	Sure Cure™	Air Indoors	10250 10230	0.2
16	28-Day	Sure Cure™	Air Indoors	11620 10630	9.3
16	56-Day	Sure Cure™	Air Indoors	11540 11540	0.0
				28-Day Average	5.3
				56-Day Average	0.1

Results are from 4 x 8 in. Sure Cure™ cylinders made during production of HPC girders. Pour 11 28-day results are the average of three cylinders. Pour 11 56-day and Pour 16 are the average of two cylinders.

Air – cylinders cured on the casting yard along with members.
Indoors – cylinders cured indoors at room temperature.

by ACI 318-95 (1995) states that if concrete in the structure will be dry under service conditions, core specimens shall be air dried for 7 days before the test, and shall be tested dry. These two procedures were used along with an alternate method in which cores were cut and tested on the same day without regard to moisture condition of the core specimen. This method was implemented because cutting and testing cores immediately appears to be the most direct means of addressing a question of concrete quality when the strengths of standard cylinder specimens are below the specified strength. Also, a set of 4 x 8 in. Sure Cure™ match-cured cylinders were tested along with the core samples to make comparisons of core compressive strengths and standard match-cured test cylinders.

Results of the core testing plan were presented in Table 4.4. Since most project cylinders are tested at 28 days for payment, core testing began at 28 days. All cores were cut from the web of the 63 in. Bulb Tee girder made at the HPC girder test pour. Cores strengths were based on the best 3 out of 4 cores tested on 70 durometer neoprene pads without the AASHTO (L/D) ratio reduction factor applied. The correction factor was not applied because AASHTO T 24 (1997) specifies a reduction factor to be applied to compressive strengths between 2000 and 6000 psi. The average strength of cores that were cut and tested at 28 days was 1.9% higher than the strengths of companion Sure Cure™ match-cured cylinders. Also, a set of cores was cut at 28 days and submerged in a limebath for 48 hours according to AASHTO T 24 (1997). The cores was tested wet at 30 days, and the average strength was 2.5% lower than that of companion Sure Cure™ match-cured cylinders but 7.6% higher than the strength of cores cut and tested at 30 days. Finally, a set of cores were cut at 28 days and allowed to air cure for 7 days according to ACI 318-95 (1995). The cores were tested dry at 35 days and the average strength was 9.5% higher than that of companion Sure Cure™ match-cured cylinders, and 8.0% higher than that of cores cut and tested at 35 days. The cores that were tested dry at 35 days after being air cured for 7 days achieved the highest overall compressive strength. A comparison of cores cut and tested at 56 days resulted in an average strength of the cores that was 7.3% higher than the strength of the Sure Cure™ match-cured cylinders. Compressive strength results of core samples based on the testing procedures used in this research agreed with past research regarding moisture condition of core samples prior to testing. A report by The Concrete Society Working Party (1976) states that dry cores achieve higher failure loads than wet cores. In this research, cores that were air-cured for 7 days and tested dry according to ACI 318-95 (1995) had an average compressive strength 13.3% higher than cores submerged in a limebath and tested wet according to AASHTO T 24 (1997).

Core compressive strength results obtained in this research did not agree with all past research or ideologies on core compressive strengths. Research by Malhotra (1977) suggests that even under excellent conditions of placing and curing, the strength of cores is unlikely to exceed 70% to 85% of the strength of standard test specimens. ACI 318-95 (1995) deems a structural member adequate if the average strength of three cores is equal to at least 85% of the specified strength, and no single core has a strength lower than 75% of the specified value. Core compressive strengths obtained in this research were up to 10% higher when compared to strengths of Sure Cure™ match-cured cylinders at a given age. The higher core compressive strengths can be attributed to the quality of vibration, core location, and moisture condition of the cores tested. Research conducted by the University of Nebraska (1996) on steam-cured precast prestressed girders showed similar results with core strengths averaging 5 to 15% higher than 4 x 8 in. test cylinders cured under the tarp along with the member. They attributed the results to the quality of vibration and curing the girders received.

Core Testing of HPC Girders from Production

During production of the HPC girders, eight out of the eighteen pours failed to meet the specified 28-day compressive strength requirement. Sure Cure™ match-cured cylinder and core results from production were presented in Table 4.11. The procedure used by the ALDOT to obtain and test cores is discussed in the following. Four cores were obtained within 7 days of the failed 28-day compressive strength test. Cores were air cured for 7 days before compressive strength testing was conducted. All cores were tested within 42 days from original placement of the concrete. Cores strengths were based on the best 3 out of 4 cores tested on sulfur caps without the AASHTO (L/D) ratio reduction factor applied. A comparison of the 28-day compressive strength of the Contractor's 4 x 8 in. Sure Cure™ match-cured cylinders versus core strength is shown in Figure 8.7. Results in Figure 8.7 are mixed with 4 out of the 8 comparison tests showing the cores having higher compressive strength. The average percent difference between the Sure Cure™ match-cured cylinders and cores was 0.2% for the eight sets of test data. The difference in strength between the core specimens and the match-cured cylinders is very little and does not agree with the popular opinion that core strengths are generally much lower than the strengths of standard test cylinders.

SUMMARY

The testing and curing procedures were found to have a significant effect on the measured strength of the concrete. The compressive strength of 4 x 8 in. cylinders tested on 70 durometer neoprene pads averaged 9.6% higher than that of 6 x 12 in. cylinders. The compressive strength of cylinders tested on 70 durometer neoprene pads averaged 7.9% higher than the strength of cylinders measured using sulfur capping.

The effect of initial curing condition on measured compressive strength was investigated. Strengths of Sure Cure™ match-cured cylinders averaged 10.7% higher at release, 4.9% higher at 28 days, and the same at 56 days when compared to strengths of cylinders cured under the tarp along with the member.

The effect of final curing condition on measured compressive strength was investigated. Compressive strengths of cylinders cured in a limebath averaged up to 16% higher at 56 days than for cylinders cured on the yard.

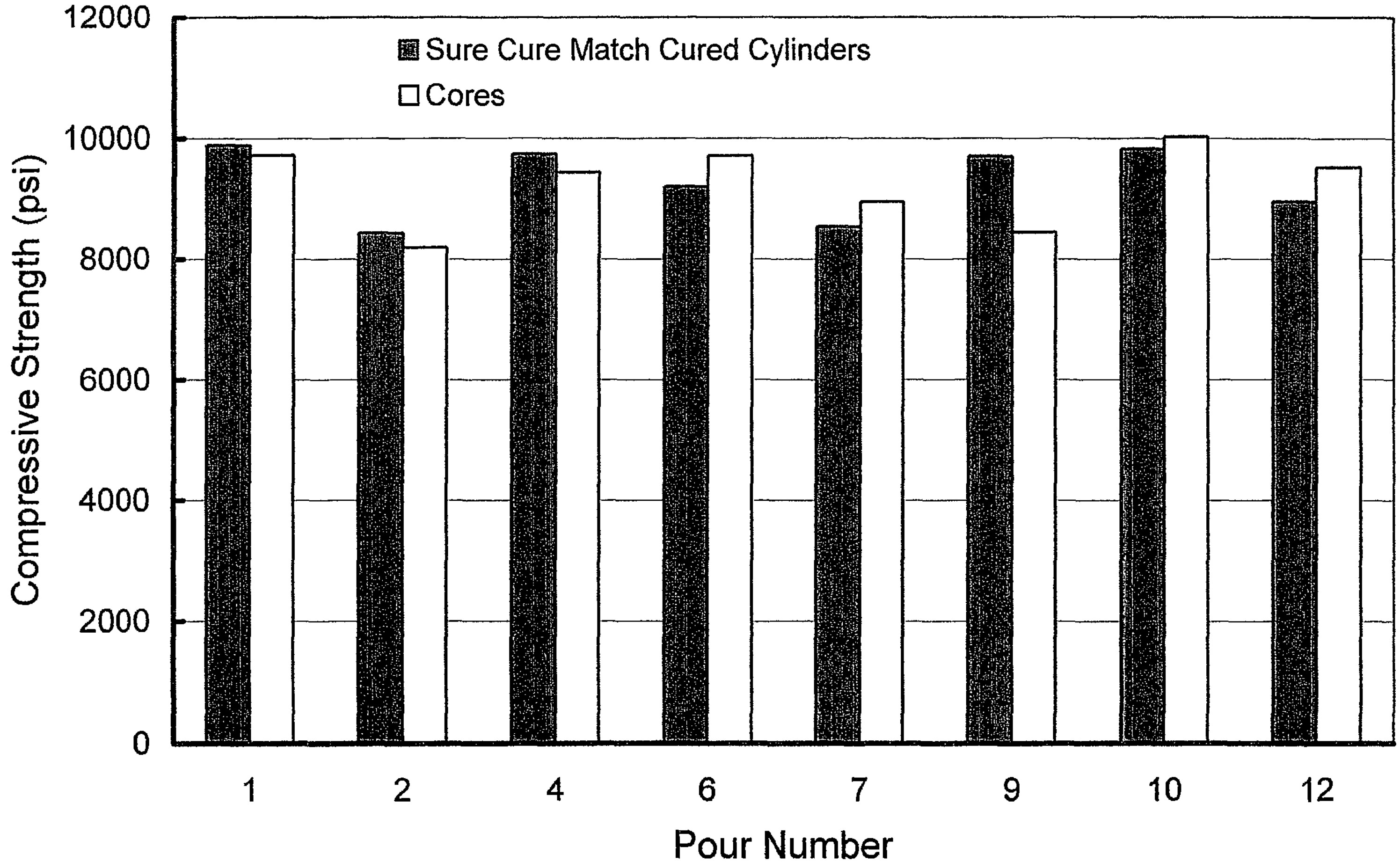


Figure 8.7. Compressive Strength of Sure Cure™ Match-Cured Cylinders at 28 Days Versus Core Compressive Strength

The measured compressive strength of core specimens tested according to ACI 318-95 (1995) resulted in the highest overall compressive strength of all cores and companion cylinders tested. These core strengths averaged 13.3% higher than for cores tested according to AASHTO T 24 (1997) procedures, 8% higher than cores cut and tested on the same day, and 9.5% higher than the strength of Sure Cure™ match-cured cylinders.

Comparisons from core specimens tested during production of the HPC girders produced mixed results, with 4 out of 8 sets of core strengths being higher than the strengths of companion Sure Cure™ match-cured cylinders. The average difference between core compressive strength and 28-day Sure Cure™ match-cured cylinder compressive strength was 0.2%.

CHAPTER NINE

CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS AND RECOMMENDATIONS

The goal of this research was to document the construction of the HPC bridge project while addressing topics regarding production, performance and concrete testing procedures of HPC. Project objectives were met through tests and observations conducted during the HPC bridge project. Field and laboratory tests were conducted to determine short-term and long-term strength and durability properties of the HPC used in the cast-in-place deck and substructure and in the prestressed concrete girders. Comparisons were made between the performance characteristics of HPC and standard concrete mixes to provide evidence of the improved performance.

General Project Specifications

Development of the project specifications generally followed the rule of minimizing new and unusual aspects and making the minor modifications required to produce HPC. Then the research reported here was oriented toward determining whether or not HPC resulted from the specifications. This was contrary to developing a purely performance based specification that might have greatly increased the cost of the concrete since the project was openly bid as any other bridge project. Results from testing of specimens from production of the girders and cast-in-place concrete indicate that high-performance concrete was indeed provided by the Contractor. Comparisons made in Chapters 6 and 7 between performance characteristics of standard concrete and the HPC show that the HPC was superior in almost every category. Hence, the project specifications are judged to have been successful in leading the Contractor to provide high-performance concrete.

Test pours required during this project were quite beneficial to the Contractors and to ALDOT because they provided an opportunity to work with the HPC and adjust to many potential problems that might have been encountered during subsequent production and construction. Test pours are recommended for future HPC projects undertaken by ALDOT until a time when HPC becomes routine. However, production of the HPC girders pointed out one potential flaw in the test pour concept. The girder test pour was made during hot weather, and the mix performed very well. Production started during the coldest part of the year, and the mix did not perform as well, and problems achieving the design strength were encountered. Hence, a truly effect test pour must be made during weather conditions that are similar to the actual production conditions. The dependence of the performance of HPC mixes on the weather conditions can be expected to be very strong. Much of the performance of the mixtures is generated by the appropriate use of admixtures which can be very sensitive to the concrete temperature. Improved methods of mix approval that allow flexibility of admixture usage and dosage need to be developed.

Use of crushed limestone for coarse aggregate in decks is also beneficial because it provides the ability to achieve improved strength and less shrinkage as compared to mixes using river gravel coarse aggregate. However, these beneficial effects might be overcome in future mixes by poor choices of aggregates. Poor combinations of coarse and fine aggregates, and poor shapes of coarse aggregates can lead to a reduction in the amount of coarse aggregate used in a deck mix to provide better workability. This can lead to increased shrinkage when compared to standard deck mixes using river gravel. The development of specifications for coarse aggregate shape for deck mixes should be considered. Also, the development of specifications for the gradations of combinations of coarse and fine aggregates should be considered. The use of an intermediate size aggregate (between coarse aggregate and sand) is believed to have greatly improved the mixture used for the HPC girders.

The required use of pozzolans in HPC mixtures is essential since they provide increased durability in the form of reduced permeability and better resistance to freeze-thaw damage, improved long-term strength characteristics, and lower concrete temperatures due to reduced heat of hydration.

Resistance to freeze-thaw damage was excellent for both the HPC girder and cast-in-place mixes. The HPC in this project received the highest FHWA HPC performance grade rating for freeze-thaw durability. Project specifications required the use of an air-entraining admixture. The required 3.5% to 6% air content was apparently sufficient to provide adequate resistance to freeze-thaw damage.

Further research is recommended to determine whether there is a need for entrained air in HPC. The use of non-air-entrained concrete could provide higher strengths and less problems in the field. Mixed results have been obtained by others regarding the freeze-thaw resistance of non-air-entrained concrete with very low permeability.

Based on AASHTO T 277 (1997) standards, resistance to rapid chloride ion penetration was considered moderate for the HPC girder mix and moderate or low for the cast-in-place mix. Reducing the permeability of the HPC in this project to values considered very low by AASHTO T 277 (1997) standards (below 1000 coulombs) would have required the use of more fly ash, silica fume, or a combination of fly ash and silica fume. While the project specifications allowed the use of a higher percentage of pozzolans, the Contractor chose not to use more. Probably because of the relatively high required compressive strengths specified in this project. For future projects in coastal areas of Alabama where very low permeability is desirable, additional refinement of the specifications is needed.

HPC Girders

It was observed during production of the HPC girders that the internal temperature history of the girders was not the same as the ambient temperature under the tarp which covered the girders and formwork up to release of the prestress force. The curing temperature history of a member can be monitored best by a thermocouple inside the member at a thick part of the member cross section.

The a maximum allowable internal temperature requirement used in this project should be used for curing HPC girders in future projects. A 160° F maximum allowable internal temperature for a concrete member provides a more durable product by helping to eliminate problems often associated with curing concrete at very high temperatures. These problems include microcracking which can lead to high permeability and delayed ettringite formation which can result in premature deterioration of the member. One set of test specimens was exposed to curing temperatures of approximately 200 °F during this project. The measured permeability of those specimens was approximately twice the permeability of the other test specimens. A maximum curing temperature requirement based on the ambient temperature under the tarp does not accurately represent the maximum temperature at which a member is cured. The maximum temperature requirement should be a limit on the internal temperature of the member.

Comparisons were made between measured concrete properties of the girder mix and values calculated from standard equations. The AASHTO (1996) value of allowable tensile stress ($6\sqrt{f'_c}$) provided a reasonable lower bound to the measured splitting tensile strength of the HPC girder mix in this project. Since the flexural strength of concrete is dependent upon the shape and texture of the coarse aggregate used, an increase in allowable tension for girder design should be examined for each particular case where this is desirable. The current AASHTO limits on tensile stress appear appropriate for the HPC mixture investigated.

A majority of the modulus of elasticity data for the HPC girder mix was between values from the equations recommended by ACI 318-95 (1995) and ACI 363 (1992). The use of these equations in design would provide reasonable estimates for the modulus of elasticity of the HPC girder mix used in this project.

Creep values predicted by ACI 209 (1997) equations provided very conservative values as compared to the creep values measured from tests of the HPC girder mix. Drying shrinkage values of the HPC girder mix correlated very well with those predicted by equations from ACI 209 (1997).

From a designer's point of view, the use of the ACI predicted values for modulus of elasticity, creep, and shrinkage in design utilizing the HPC girder mix from this research would result in a conservative yet well performing design. Use of this HPC girder mix would result in less prestress losses and less camber growth than predicted using standard estimates of modulus of elasticity, creep and shrinkage. The issues of camber and prestress losses are explored further in a subsequent report of this project.

Cast-In-Place HPC Decks

In this project, the HPC bridge decks were wet cured for 7 days. It is recommended that prolonged wet curing of at least 7 days on the HPC bridge decks is necessary to provide adequate moisture to the deck so that pozzolans incorporated in the mix design can develop improved strength and durability properties with which they are associated. The decks of the Bulger Creek Bridge and Uphapee Creek Relief Bridge were practially crack free at the end of construction. This is an indication that the wet curing successfully controled plastic and drying shrinkage cracking in the HPC decks. However, the deck of the Uphapee Creek Bridge has regularly spaced transverse cracks in most spans. The reasons for the differences in performance of the bridges are unclear.

Testing of HPC

Results from this project confirm that the methods used to cure and test concrete cylinders can have a significant effect on the measured strength. This observation is based primarily on data obtained from testing the high-strength prestressed girder concrete. Hence, conclusions and recommendations presented in this section apply to future projects where release strengths of 8000 psi and 28-day strengths of 10,000 psi or higher are required.

Comparisons presented in Chapter 8 show that the compressive strengths from 4 x 8 in. cylinders were approximately 10% higher than from 6 x 12 in. cylinders. After the testing of this project was essentially complete, Burg et al.

(1999) reported the results of a very comprehensive study on testing methods for high-strength concrete. Burg's conclusion is that oberved differences between strengths from 4 x 8 in. and 6 x 12 in. cylinders are a result primarily of the testing machine, and the cylinder size effect is only approximately 2% instead of the 10% reported in Chapter 8. Based on all available information, including the fact that the most reliable match curing systems available allow the use of only 4 x 8 in. cylinders, it is recommended that the use of 4 x 8 in. cylinders be allowed in future projects.

Comparisons of capping methods showed that the compressive strength of cylinders tested on 70 durometer neoprene pads was approximately 8% higher than for sulfur capped cylinders. This points out the possibility that available sulfur capping compounds and application techniques are not adequate for accurately measuring the strength of high-strength concrete. No conclusive argument for or against sulfur capping can be made based on the test results from this project. Burg et al. (1999) conclude that in the absence of comprehensive tests to evaluate the suitability capping compounds, cylinders with strengths in excess of 10,000 psi should be tested using end grinding. The use of end grinding does not appear practical for controlling concrete strength in bridge construction. Further development of reuseable pads such those described by Kahn (1999) is recommended. Reuseable pads, such as neoprene pads, provide a safer, more efficient and economical way to test high-strength concrete than does sulfur capping.

Comparisons of initial curing methods indicated that Sure Cure™ match-cured cylinders broke at loads 11% higher at release and 5% higher at 28 days than cylinders cured under the tarp along with the member. The conclusion from this project is that Sure Cure™ match-cured cylinders provide the most accurate indication of a concrete member's strength because the test cylinders are cured at a temperature history most like the concrete member they represent. Cylinders cured under the tarp along side the member may not provide an accurate indication of the strength of the concrete member because the temperature history of the test cylinders is not controlled. It is recommended that a match curing system be used for initial curing of release and 28-day cylinders.

Comparisons of final curing conditions showed that the compressive strengths of cylinders cured in a moist environment were up to 12% higher at 28 days and up to 18% higher at 56 days than the strengths of cylinders cured on the casting yard. The conclusion is that the highest possible compressive strength of HPC test cylinders is achieved when cylinders are cured in a moist environment. Similarly, the best possible performance of HPC girders would result from mosit curing of the girders, but this is not economically practical or necessary. Curing of test cylinders on the production yard with the member after release is recommended for future projects. It is likely that the concrete cylinders might stop gaining strength due to a lack of moisture before the girders do, but it is equally likely that limebath or moist room curing provides an unrealistically high source of moisture relative to prestressed concrete girders that are not moist cured after release.

Limited test results from this project show that the strength of cores cut and tested on the same day provided a suitable correlation to the strength of Sure Cure™ match-cured cylinders. This points out that cores cut horizontally from the web of a prestressed concrete girder should not automatically be assumed to have a lower strength than standard cylinders. Since the process of cutting and testing cores without regard to the moisture condition of the cores is not standard, further development of this procedure may be warranted.

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APPENDICES

APPENDIX A

TABLES

Table A.1. Individual Core Compressive Strength (psi) Tests from Test Pour

Test Procedure	28-Day		30-Day		35-Day		56-Day	
		Avg.		Avg.		Avg.		Avg.
(1)	11490 11480 12320	**11770**	10870 10710 10390	**10660**	12100 11880 12100	**12030**	12660 12680 12700	**12680**
(2)	-	-	11390 10850 12160	**11470**	-	-	-	-
(3)	-	-	-	-	12190 13480 13290	**12990**	-	-
(4)	11630 11690 11340	**11550**	12030 11550 11710	**11760**	12220 11300 12050	**11860**	11870 11770 11810	**11820**

Table A.2. Individual Cylinder Test Results from Test Pour

Cylinder Initial Cure	Compressive Strength (psi)										Tensile Strength (psi)	
	Release		7-Day		14-Day		28-Day		56-Day		Release	
		Avg.		Avg.		Avg.		Avg.		Avg.		Avg.
4 x 8 in. CMC	10940 9390	**10170**	-	-	11340 11300	**11320**	11540 11300	**11420**	13210 13090	**13150**	-	-
4 x 8 in. Sure Cure	9750 9810 10350	**9970**	10580 10760 11050	**10800**	10880 10680 11220	**10930**	11630 11690 11340	**11550**	11870 11770 11810	**11820**	860 860	**860**
4 x 8 in. Under Tarp	10020 10100	**10060**	-	-	12250 12170	**12210**	11890 12410	**12150**	-	-	-	-
4 x 8 in. ASTM	8720 8830 8790	**8780**	10890 10680 10080	**10550**	11640 11110 11400	**11380**	12170 12650 12500	**12440**	13250 14030 13470	**13580**	580 680	**630**
6 x 12 in. CMC	9730 9270	**9500**	-	-	10470 10260	**10370**	11440 10790	**11120**	-	-	-	-
6 x 12 in. Sure Cure	8380 8650	**8520**	-	-	-	**-**	9830 10940	**10390**	-	-	640 690	**670**
6 x 12 in. Under Tarp	8860 9160	**9010**	-	-	11040 11140	**11090**	10910 10590	**10750**	-	-	-	-

Table A.3. Individual Cylinder Test Results for Final Curing Conditions on Compressive Strength

Cylinder Initial Cure	Final Cure	Compressive Strength (psi)					
		7-Day		28-Day		56-Day	
			Avg.		Avg.		Avg.
4 x 8 in. Under Tarp	Air	11570 11770	**11670**	10870 11340	**11110**	11730 11780	**11760**
4 x 8 in. Under Tarp	Moist	10410 10950	**10680**	11750 11270	**11510**	12900 12920	**12910**
4 x 8 in. Under Tarp	Lime	11220 11210	**11220**	11380 11600	**11490**	12910 12340	**12630**

Table A.4. Individual Cylinder Test Results for Capping Method on Compressive Strength

Cylinder Initial Cure	Capping Method	Compressive Strength (psi)							
		Release		7-Day		14-Day		28-Day	
			Avg.		Avg.		Avg.		Avg.
4 x 8 in. CMC	Neoprene	10940 9390	**10170**	-	-	11340 11300	**11320**	11540 11300	**11420**
4 x 8 in. CMC	Sulfur	8350 9630	**8990**	10340 10340	**10340**	10740 9940	**10340**	10940 10940	**10940**
4 x 8 in. Under Tarp	Neoprene	10020 10100	**10060**	-	-	12250 12170	**12210**	11890 12410	**12150**
4 x 8 in. Under Tarp	Sulfur	8750 9670	**9210**	-	-	10270 10580	**10430**	11770 11500	**11640**
6 x 12 in. CMC	Neoprene	9730 9270	**9500**	-	-	10470 10260	**10370**	11440 10790	**11120**
6 x 12 in. CMC	Sulfur	9200 8840	**9020**	9810 10080	**9950**	10220 10380	**10300**	10610 10540	**10580**
6 x 12 in. Under Tarp	Neoprene	8860 9160	**9010**	-	-	11040 11140	**11090**	10910 10590	**10750**
6 x 12 in. Under Tarp	Sulfur	8280 8490	**8390**	10740 10490	**10620**	10470 10010	**10240**	10510 10590	**10550**

Table A.5. Results from Production of the HPC Girders

Pour Number[1]	Compressive Strength (psi)							
	Release		28-Day		56-Day		Core[2]	
		Avg.		Avg.		Avg.		Avg.
1	8440 7720	**8080**	9910 9870	**9890**	10340 9870	**10110**	10420 9630 9140	**9730**
2	8280 7880	**8080**	8520 8360	**8440**	9830 9390	**9610**	8630 8460 7520	**8200**
3	7960 8520	**8240**	10500 9980	**10240**	10260 11180	**10720**	-	-
4	7960 8200	**8080**	9790 9710	**9750**	9550 9390	**9470**	8940 10450 8960	**9450**
5	8120 8480	**8300**	10020 10100	**10060**	10340 10380	**10360**	-	-
6	7800 8440	**8120**	9710 8710	**9210**	8520 8360	**8440**	10270 9220 9660	**9720**
7	8120 8200	**8160**	8950 8120	**8540**	8120 9070	**8600**	9420 8670 8760	**8950**
8	8830 8830	**8830**	10020 9980	**10000**	10060 10300	**10180**	-	-
9	7960 8120	**8040**	9710 9710	**9710**	9470 9630	**9550**	8020 8920 8410	**8450**
10	8440 8520	**8480**	9940 9710	**9830**	9670 9670	**9670**	9710 10400 9980	**10030**
11	8240 7920	**8080**	10220 9830	**10030**	9390 10020	**9710**	-	-
12	8280 8440	**8360**	9110 8790	**8950**	8440 8790	**8620**	9890 9510 9190	**9530**
13	8910 9350	**9130**	11140 10500	**10820**	10340 10300	**10320**	-	-
14	8830 8520	**8680**	9710 10780	**10250**	9830 8120	**8980**	-	-
15	9230 9510	**9370**	11250 10860	**11060**	11060 11580	**11320**	-	-
16	8440 8240	**8340**	9940 10580	**10260**	10780 10300	**10540**	-	-
17	9630 9270	**9450**	10500 11300	**10900**	11040 11720	**11380**	-	-
18	9710 9910	**9810**	10100 11220	**10660**	11820 11370	**11600**	-	-

1. No. 67 limestone used in pours 1 through 12, No. 7 limestone used in pours 13 through 18. All strength tests were conducted on sulfur caps.
2. Core testing was conducted if the required 28-day strength was not achieved. 4 cores were cut according to AASHTO T 24. Core strengths were based on the best 3 out of 4 cores tested dry on sulfur caps without the AASHTO (L/D) reduction factor applied.

Table A.6. Batch Ticket Information From the Production of the HPC Girders Pours 1-6

			Average Mix Proportions (lbs/yd^3)						Admixtures			
									(oz/cwt)		(oz)	
Pour	Total Yards	Number Of Batches	Cement Type III	Fly Ash Type C	#67 Limestone	#89 Sand	#100 Sand	Water	Rheobuild® 1000	Delvo®	Micro Air®	Avg. w/cm
1	41	10	758	132	1867	386	758	241	16	2.5	4	0.271
2	39	10	750	132	1863	382	778	238	16	2.5	4	0.27
3	41	11	753	132	1859	389	760	242	16	1.4	4	0.273
4	42.5	10	750	132	1845	375	757	225	16	1.4	4	0.255
5	40	10	752	133	1847	379	747	233	16	1.4	4	0.263
6	43.5	11	751	131	1846	383	761	228	16	1.4	4	0.259

Table A.7. Batch Ticket Information From the Production of the HPC Girders Pours 1-6

			Average Mix Proportions (lbs/yd^3)						Admixtures (oz/cwt)		Admixtures (oz)	
Pour	Total Yards	Number Of Batches	Cement Type III	Fly Ash Type C	#67 Limestone	#89 Sand	#100 Sand	Water	Rheobuild® 1000	Delvo®	Micro Air®	Avg. w/cm
7	41	10	753	132	1864	384	748	256	14	2.5	4	0.289
8	40.5	10	751	132	1865	383	751	254	14	2.5	4	0.288
9	40.5	10	755	133	1863	386	758	255	14	1.4	4	0.287
10	45	12	753	132	1867	385	773	245	14	1.6	4	0.277
11	40	10	752	131	1861	385	761	249	14	1.4	4	0.282
12	40.5	11	753	132	1839	381	751	243	14	1.7	4	0.275

Table A.8. Batch Ticket Information From the Production of the HPC Girders Pours 1-6

			Average Mix Proportions (lbs/yd^3)						Admixtures (oz/cwt)		Admixtures (oz)	
Pour	Total Yards	Number Of Batches	Cement Type III	Fly Ash Type C	#67 Limestone	#89 Sand	#100 Sand	Water	Rheobuild® 1000	Delvo®	Micro Air®	Avg. w/cm
13	40	10	754	132	1831	375	749	230	16	1.6	4	0.26
14	48	12	752	132	1833	377	736	250	16	1.6	4	0.283
15	41	11	750	132	1844	377	749	232	16	3.5	4	0.263
16	44.5	11	750	132	1831	377	764	247	16	3.4	4	0.28
17	40.5	10	753	133	1824	380	761	240	16	3.2	4	0.271
18	44.5	11	754	132	1834	371	746	228	16	3.3	4	0.257

Table A.9. Measured Air, Slump, and Placement Temperature Results from Production of the HPC Girders

Pour Number	Air Content (%)		Slump (in.)		Concrete Placement Temperature (°F)	
	#1	#2	#1	#2	#1	#2
1	3.8	3.5	7	6	71	-
2	4.6	5.4	8.5	9	66	-
3	4.5	3.9	8.5	5.25	65	-
4	5.0	5.0	8	7	60	61
5	4.9	4.8	8.25	8.25	62	62
6	3.8	4.6	5.5	8.75	68	68
7	4.2	4.4	8	8	65	65
8	3.6	4.1	7.5	8.25	70	72
9	4.1	5.2	7.75	8.5	70	71
10	4.0	3.8	8.5	8	71	74
11	3.6	3.6	7.5	8	77	78
12	3.7	4.2	8	7.5	71	72
13	5.1	4.1	8.5	8	78	84
14	4.6	4.7	7	7.75	75	76
15	6.0	4.0	8	4.75	67	71
16	5.2	5.1	7	7.5	64	69
17	5.2	4.5	8.75	8.25	74	74
18	4.8	4.5	7.5	8.25	75	-

Table A.10. Results from Production of the CIP HPC Substructure at Uphapee Creek Relief Bridge

Date	Location or Station Number	Compressive Strength (psi)				Fresh Properties	
		7-Day		28-Day		Air (%)	Slump (in.)
			Avg.		Avg.		
10/5/98	357+14	4620 4720	**4670**	5950 5400	**5680**	5.3	3.25
10/5/98	357+14	3980 3920	**3950**	4210 4280	**4250**	10.0	3.5
10/9/98	357+96	4840 4960	**4900**	6120 6290	**6210**	4.7	4.5
10/14/98	357+14	5190 5310	**5250**	6600 6560	**6580**	4.0	6.75
10/21/98	358+37	4700 4720	**4710**	6200 5960	**6080**	5.2	6.75
10/28/98	358+78	4990 5200	**5100**	6200 6240	**6220**	5.0	6.25
11/3/98	358+37	5040 4940	**4990**	6190 6790	**6490**	4.3	6.5
11/10/98	358+78	4950 4920	**4940**	6420 6460	**6440**	4.0	6.75
11/11/98	359+19	4960 4970	**4970**	6620 6440	**6530**	5.4	6.0
11/18/98	359+19	4770 4800	**4790**	6460 6650	**6560**	3.9	6.75
2/5/99	359+60	4650 4640	**4650**	6170 6100	**6140**	3.2	4.75
2/23/99	357+14	4380 3860	**4120**	5730 5790	**5760**	3.5	7.5
2/26/99	Webwalls	5200 5280	**5240**	6660 6740	**6700**	4.5	5.75
3/2/99	Webwalls	4990 5090	**5040**	6480 6340	**6410**	6.5	7.0
3/5/99	Webwalls	5470 5410	**5440**	6280 6850	**6570**	6.0	6.25

Table A.11. Results from Production of the CIP HPC Substructure at Bulger Creek Bridge

Date	Location or Station Number	Compressive Strength (psi)				Fresh Properties	
		7-Day		28-Day		Air (%)	Slump (in.)
			Avg.		Avg.		
11/16/98	414+27	4940 4970	**4960**	6360 6220	**6290**	4.8	7.0
11/19/98	414+27	4740 4700	**4720**	6270 6320	**6300**	4.5	7.75
11/30/98	412+62	5220 5240	**5230**	6700 6780	**6740**	5.0	5.75
12/3/98	413+72	4710 4970	**4840**	6490 6520	**6510**	4.0	7.5
12/9/98	412+62	5350 5490	**5420**	6840 6770	**6810**	3.5	3.5
12/14/98	413+17	5610 5630	**5620**	7320 7310	**7320**	4.5	4.75
3/17/99	Webwalls	5240 5160	**5200**	7250 6880	**7070**	5.5	6.25
3/26/99	Webwalls	4980 4970	**4980**	6230 6440	**6340**	4.6	6.0

Table A.12. Results from Production of the CIP HPC Superstructure at Uphapee Creek Bridge

Date	Location or Station Number	Compressive Strength (psi)				Fresh Properties	
		7-Day		28-Day		Air (%)	Slump (in.)
			Avg.		Avg.		
10/5/99	349+86	5520 5600	**5560**	6610 6850	**6730**	4.5	4.25
10/6/99	351+01	5800 5590	**5685**	6980 6660	**6820**	5.6	4.5
10/7/99	346+44	5370 5390	**5345**	6340 6650	**6495**	5.9	5.25
10/8/99	346+44	5370 5390	**5380**	6930 6650	**6790**	6	5.75
10/12/99	349+86	5310 5220	**5265**	6500 6510	**6505**	5.8	5.75
10/14/99	351+01	6160 6240	**6200**	7140 7290	**7215**	5.2	4.25
10/15/00	347+58	5770 6010	**5890**	6870 7200	**7035**	5.3	7
10/18/00	352+14	6420 6280	**6350**	7310 7140	**7225**	4.2	5.25
10/18/00	347+58	5880 6080	**5980**	6910 6960	**6935**	4.2	5.25
10/19/00	348+72	6120 6670	**6395**	7040 7560	**7300**	4.1	5
10/20/00	348+72	5680 5710	**5695**	7190 7220	**7205**	4.6	4.25
10/22/00	353+28	5670 6000	**5835**	7110 7240	**7175**	4.7	4.75
10/22/00	353+28	6490 6450	**6470**	8080 6830	**7455**	4.6	4.25
10/25/00	353+28	6010 6010	**6010**	7310 7460	**7385**	4.2	4
10/26/00	345+30	6250 6080	**6165**	7690 7660	**7675**	4.3	6
10/27/00	348+72	5540 5690	**5615**	6940 7100	**7020**	4.2	5.75
10/28/00	345+30	5710 5730	**5720**	7710 7220	**7465**	3.5	5
10/28/00	347+58	6400 6210	**6305**	7530 7660	**7595**	3.7	6
10/29/00	346+44	5910 5730	**5820**	7380 7000	**7190**	4.3	5.5

Table A.13. Results from Production of the CIP HPC Superstructure at Bulger Creek Bridge

Date	Location or Station Number	Compressive Strength (psi)				Fresh Properties	
		7-Day		28-Day		Air (%)	Slump (in.)
			Avg.		Avg.		
5/12/99	Span 1	5450 4970	**5210**	6770 6710	**6740**	4.8	6.25
5/12/99	Span 1	5460 5450	**5460**	6600 6580	**6590**	4.7	6.0
5/13/99	Span 3	5680 5680	**5680**	7360 6980	**7170**	4.6	6.25
5/13/99	Span 3	5350 5290	**5320**	6820 6810	**6820**	4.5	6.25
5/18/99	Span 2	5240 5410	**5330**	6560 6480	**6520**	5.4	6.5
5/18/99	Span 2	5000 4910	**4960**	6360 6320	**6340**	5.5	6.5

Table A.14. Results from Production of the HPC Girders (1-7)

Pour Number	Curing Method	Compressive Strength (psi)							
		Release		7-Day		28-Day		56-Day	
			Avg.		Avg.		Avg.		Avg.
1	1	9110 8780 8930	**8940**	10010 9650 9680	**9780**	10130 10270 9760	**10050**	11170 11250 11420	**11280**
1	4	8200 8000	**8100**	9350 8910	**9130**	9710 9790	**9750**	10020 10180	**10100**
1	5	-	-	-	-	10410 10330 9660	**10130**	10900 11160 10990	**11020**
2	4	8990 7160	**8080**	9150 8200	**8680**	9630 8910	**9270**	-	-
3	1	-	-	-	-	-	-	10940 10580 10740	**10750**
3	4	8590 8040	**8320**	9550 9510	**9530**	10700 10500	**10600**	11850 12730	**12290**
4	4	-	-	8630 8750	**8690**	10500 10100	**10300**	10460 11160	**10810**
5	1	-	-	-	-	10820 11180	**11000**	10740 11340 10460	**10850**
5	4	7040 7080	**7060**	9230 9710	**9470**	10420 10500	**10460**	10340 10140	**10240**
6	1	-	-	-	-	-	-	8910 10350 8550	**9270**
6	4	7760 7760	**7760**	9150 8590	**8870**	10020 11020	**10520**	10230 10580	**10410**
7	4	7720 7720	**7720**	9030	**9030**	-	-	-	-

Table A.15. Results from Production of the CIP HPC Superstructure at Uphapee Creek Bridge

Date	Location or Station Number	Compressive Strength (psi)				Fresh Properties	
		7-Day		28-Day		Air (%)	Slump (in.)
			Avg.		Avg.		
10/29/00	Webwalls	6150 5850	**6000**	7340 7350	**7345**	4.9	4.75
11/3/00	Webwalls	5790 5670	**5730**	6780 6550	**6665**	4.2	6
11/5/00	Webwalls	5570 5660	**5615**	7120 7070	**7095**	4.6	5.25
11/9/00	Webwalls	5380 5430	**5405**	7130 6950	**7040**	3.9	5.75
11/15/00	Span 5 Center	6290 6060	**6175**	7490 7500	**7495**	4.2	5
11/15/00	Span 5 Center	5710 5670	**5690**	7090 6910	**7000**	4	6
11/16/00	Span 6 Center	6470 6200	**6335**	7940 7750	**7845**	4.9	5.5
11/16/00	Span 6 Center	5630 5780	**5705**	6970 7060	**7015**	5.7	5.75
11/16/00	Webwalls	5700 5590	**5645**	6990 6900	**6945**	3.9	5.5
11/18/00	Span 7 Center	5000 5000	**5000**	6070 5930	**6000**	4.9	6.5
11/18/00	Span 7 Center	5700 5760	**5730**	6630 6780	**6705**	3.8	6
11/18/99	Webwalls	4970 5060	**5015**	5920 6010	**5965**	4.4	6
11/19/99	Span 5 North ¼	5810 6210	**6010**	6890 7310	**7100**	4.5	7
11/22/99	Span 6 South ¼	5460 6110	**5785**	6780 6910	**6845**	4.2	5
11/23/99	Span 6 North ¼	5870 5850	**5860**	7010 6930	**6970**	5	5
11/24/99	Span 7 North ¼	5740 5500	**5620**	7240 7030	**7135**	4.5	5.25
11/29/99	Span 7 South ¼	5130 5120	**5125**	6590 6430	**6510**	5.1	6.75
11/30/99	Span 5 South ¼	5910 5810	**5860**	7300 7420	**7360**	4.9	5.5
12/2/99	Span 4 Center	5500 5940	**5720**	7180 6620	**6900**	5.9	6
12/2/99	Span 4 Center	5610 5330	**5470**	7400 7190	**7295**	4.8	5.25
12/6/99	Span 3 Center	5710 5820	**5765**	8070 8490	**8280**	4	5.5
12/6/99	Span 3 Center	6480 6630	**6555**	7840 7670	**7755**	5.1	6

12/7/99	Span 4 North ¼	5890 5390	**5640**	7150 7050	**7100**	5.3	5.5
12/8/99	Span 4 South ¼	5850 5790	**5820**	6940 7200	**7070**	5.6	5.5
12/14/99	Span 2 Center	6620 6590	**6605**	8130 7880	**8005**	4.6	5
12/14/99	Span 2 Center	6320 6950	**6635**	7940 7960	**7950**	4.6	6
12/15/99	Span 1 Center	6890 6250	**6570**	8510 7840	**8175**	4.6	5.5
12/15/99	Span 1 Center	6720 6940	**6830**	7950 7130	**7540**	4.3	6
12/16/99	Span 3 North ¼	6280 6920	**6600**	8230 8050	**8140**	4.9	6
12/16/99	Span 3 South ¼	6420 6920	**6670**	8190 8210	**8200**	3.7	5.5
12/20/99	Span 2 South ¼	6350 6280	**6315**	7590 7520	**7555**	5.3	6
12/22/99	Span 2 North 1/4	5740 6100	**5920**	7160 6760	**6960**	6.2	6
12/23/99	Span1 South ¼	6110 6360	**6235**	7600 7790	**7695**	5	5.5
12/23/99	Span 1 North ¼	5970 6140	**6055**	7780 7800	**7790**	3.25	5.5

Table A.15. Results from Production of the HPC Girders (8-12)

Pour Number	Curing Method	Compressive Strength (psi)							
		Release		7-Day		28-Day		56-Day	
			Avg.		Avg.		Avg.		Avg.
8	2	8970 8930	**8950**	-	-	10350 10540 10540	**10480**	10150 10150 10150	**10150**
8	4	7080 7880	**7480**	8990 8830	**8910**	9940 9830	**9890**	10540 10180	**10360**
9	1	-	-	-	-	-	-	9950 10270 9750	**9990**
9	4	7720 7320	**7520**	8670 8910	**8790**	9630 9310	**9470**	10580 10500	**10540**
10	1	-	-	-	-	11540 10580 11140	**11090**	10580 10500 9960	**10350**
10	4	8200 8200	**8200**	9270 9230	**9250**	10100 9710	**9910**	10180 10580	**10380**
11	1	-	-	-	-	10350 11140 10150	**10550**	9950 10540	**10250**
11	2	-	-	-	-	9950 10940 10350	**10410**	10190 10270	**10230**
11	3	-	-	-	-	10270 10940 10740	**10650**	-	-
11	4	-	-	9470 9310	**9390**	10100 9750	**9930**	-	-
12	1	-	-	-	-	10150 9750	**9950**	10350 9950	**10150**
12	3	-	-	-	-	9950 10150	**10050**	11140 11440	**11240**
12	4	-	-	9150 8400	**8780**	9310 8670	**8990**	-	-

Table A.16. Results from Production of the HPC Girders (13-18)

Pour Number	Curing Method	Compressive Strength (psi)							
		Release		7-Day		28-Day		56-Day	
			Avg.		Avg.		Avg.		Avg.
13	1	-	-	-	-	10740 11540 11540	**11270**	11540 11380 11140	**11350**
13	4	9070 8910	**8990**	10540 9710	**10130**	11020 10900	**10960**	11540 11620	**11580**
14	1	-	-	-	-	10940 10580	**10760**	11300 10820	**11060**
14	3	-	-	-	-	12410 11740	**12070**	12140 13130	**12640**
14	4	8990 8750	**8870**	10020 10340	**10180**	10700 11140	**10920**	11930 11770	**11850**
15	1	-	-	10740 11140 11440	**11110**	12530 12100 12260	**12300**	12220 11940 12530	**12230**
15	4	8670 8790	**8730**	10140 9470	**9810**	11540 11660	**11600**	11700 11540	**11620**
16	1	-	-	-	-	11620 11620	**11620**	11460 11620	**11540**
16	2	-	-	-	-	10820 10430	**10630**	11940 11140	**11540**
16	3	-	-	-	-	13330 12730	**13030**	13770 13530	**13650**
16	4	8560 8320	**8440**	10020 9510	**9770**	12050 10700	**11380**	11140 11650	**11400**
17	1	-	-	12330 12330 12180	**12280**	12020 11700 11940	**11880**	12530 12490 11540	**12190**
17	4	9150 9630	**9390**	10180 10420	**10300**	10740 11420	**11080**	11700 11380	**11540**
18	1	9950 9950	**9950**	-	-	12100 10540	**11320**	12200 12540	**12370**
18	4	9910 9590	**9750**	10700 11140	**10920**	11220 11360	**11290**	12010 12010	**12010**

Table A.17. Individual Cylinder Test Results for Comparison of HPC and Standard Girder Concrete Properties

Hardened Properties		HPC		Standard Concrete	
			Avg.		Avg.
Compressive Strength (psi)	Release	8950 9350	**9150**	6880 7160 7160	**7070**
	28-Day	11140 11540	**11340**	10540 10190 10350	**10360**
	56-Day	10940 11340	**11140**	9550 10350 10740	**10210**
Tensile Strength (psi)	Release	770 770	**770**	600 620	**610**
	28-Day	850 850	**850**	850 890	**870**
Modulus of Elasticity (psi)	Release 28-Day 56-Day	5,900,000 6,300,000 6,000,000		5,250,000 5,900,000 5,900,000	
Freeze-Thaw Durability (Durability Factor, %)	300 Cycles	96.0% 99.2% 97.6%	**97.6%**	94.5% 96.1% 94.6%	**95.1%**
Chloride Ion Permeability (Coulombs)	56-Day	2070 2180	**2130**	2730 2850	**2790**
	91-Day	1840 1870	**1850**	2930 3220	**3070**
Creep (microstrain/psi)	90-Day	0.20		0.54	
Shrinkage (microstrain)	90-Day	280 140 290		350 250 430	

4 x 8 in. Sure Cure™ match-cured cylinders were used where applicable.
Final cure was indoors at 70-74 degrees Fahrenheit.

Table A.18. Batch Ticket Information for CIP HPC Production Pours

Sample Number	1	2	3	4
Mix Design	HPC-3	HPC-4	HPC-4	HPC-4
Cubic Yards	5.3	8.5	7.0	7.0
Delivery Time	87 min	66 min	73 min	62 min
Average Mix Proportions (lbs/yd^3)				
Type II Cement	646	658	657	662
Type C Fly Ash	158	165	167	161
Coarse Aggregate	1939	1858	1856	1844
Fine Aggregate	1018	1042	1039	1058
Water	285	288	300	296
Admixtures (oz)				
Polyheed® 997	64.0	82.4	66.0	66.0
Pozzolith® 100-XR	12.6	19.6	19.6	19.6
MB AE® 90	4.0	3.9	3.7	3.7
Average w/cm ratio	0.354	0.35	0.364	0.36

Mix proportions are dry weights.

Table A.19. Individual Compressive Strength Test Results from Production of the CIP HPC

Sample Number	Mix Number	Compressive Strength (psi)							
		7-Day		28-Day		56-Day		91-Day	
			Avg.		Avg.		Avg.		Avg.
1	HPC-3	5130 5540	**5330**	7070 6810	**6940**	7460 7340	**7400**	7890 8310	**8100**
2	HPC-4	6100 5750 5570	**5810**	7250 7920 7160	**7440**	8050 8130 8490	**8220**	8750 8310 8840	**8630**
3	HPC-4	5160 5310 5380	**5280**	6990 7340 7340	**7220**	7460 7430 7430	**7440**	7960 7750 7890	**7870**
4	HPC-4	5060 5230 5220	**5170**	6370 6510 6470	**6450**	6930 6930 6970	**6940**	7600 7250 7250	**7370**

Table A.20. Individual Tensile Strength Test Results from Production of the CIP HPC

Sample Number	Mix Number	Compressive Strength (psi)							
		7-Day		28-Day		56-Day		91-Day	
			Avg.		Avg.		Avg.		Avg.
1	HPC-3	350 450	**400**	350 530	**440**	490 440	**460**	530 510	**520**
2	HPC-4	420 460	**440**	550 510	**530**	560 490	**520**	490 490	**490**
3	HPC-4	420 400	**410**	530 530	**530**	490 490	**490**	600 510	**560**
4	HPC-4	340 350	**350**	470 470	**470**	490 490	**490**	460 400	**430**

Table A.21. Individual Cylinder Test Results for Comparison of CIP HPC Mix and Standard Concrete Mix Properties

Hardened Properties		HPC		Standard Concrete	
			Avg.		Avg.
Compressive Strength (psi)	28-Day	6990 7340 7340	**7220**	4780 5220 5130	**5040**
	56-Day	7460 7430 7430	**7440**	5920 5570 5750	**5750**
	91-Day	7960 7750 7890	**7870**	5660 5310 5840	**5600**
Tensile Strength (psi)	28-Day	530 530	**530**	340 270	**310**
	56-Day	490 490	**490**	390 410	**400**
Modulus of Elasticity (psi)	28-Day 56-Day	5,750,000 5,350,000		4,700,000 4,700,000	
Freeze-Thaw Durability (Durability Factor, %)	300 Cycles	91.4% 90.5% 94.8%	**92.2%**	82.8% 82.2% 83.2%	**82.7%**
Chloride Ion Permeability (Coulombs)	56-Day	2680 2850	**2770**	5590 5550	**5570**
	91-Day	1990 1930	**1960**	5540 5560	**5540**
Shrinkage (microstrain)	90-Day / Moist	330		310	

Standard 6 x 12 in. cylinders made in plastic molds were used were applicable. Cylinders were moist cured until tested.

APPENDIX B

FIGURES

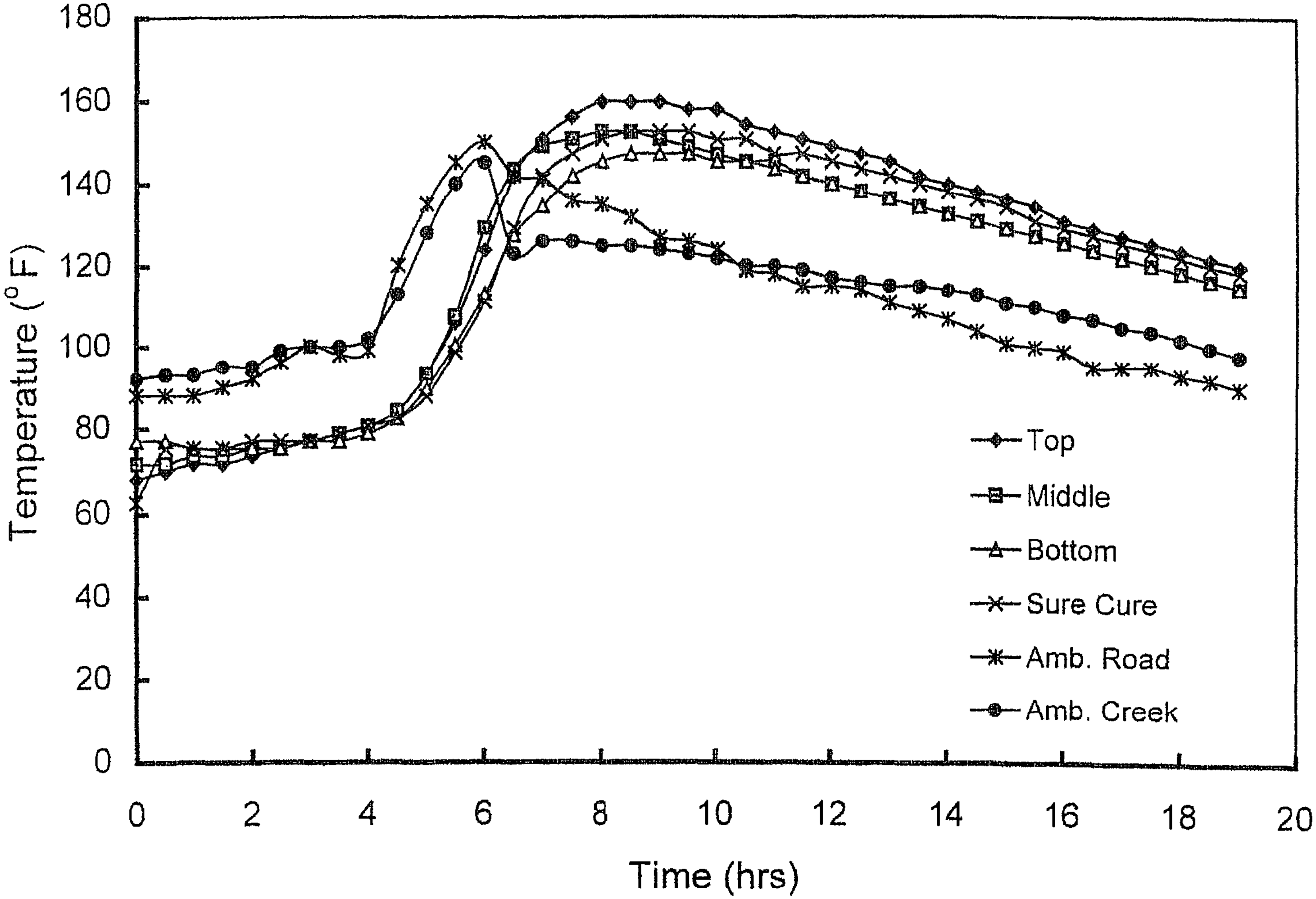


Figure B.1. Temperature History of Pour 1

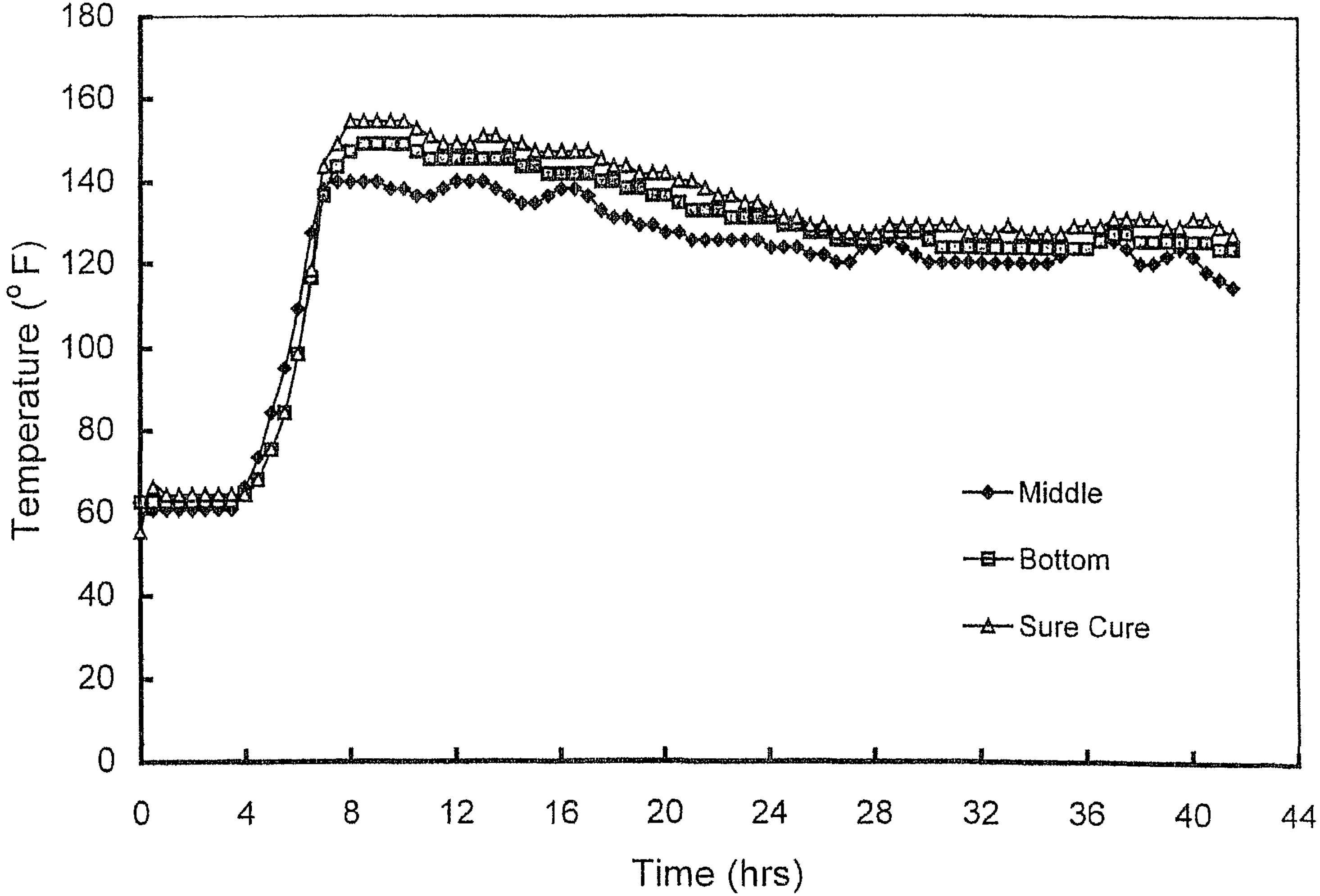


Figure B.2. Temperature History of Pour 2

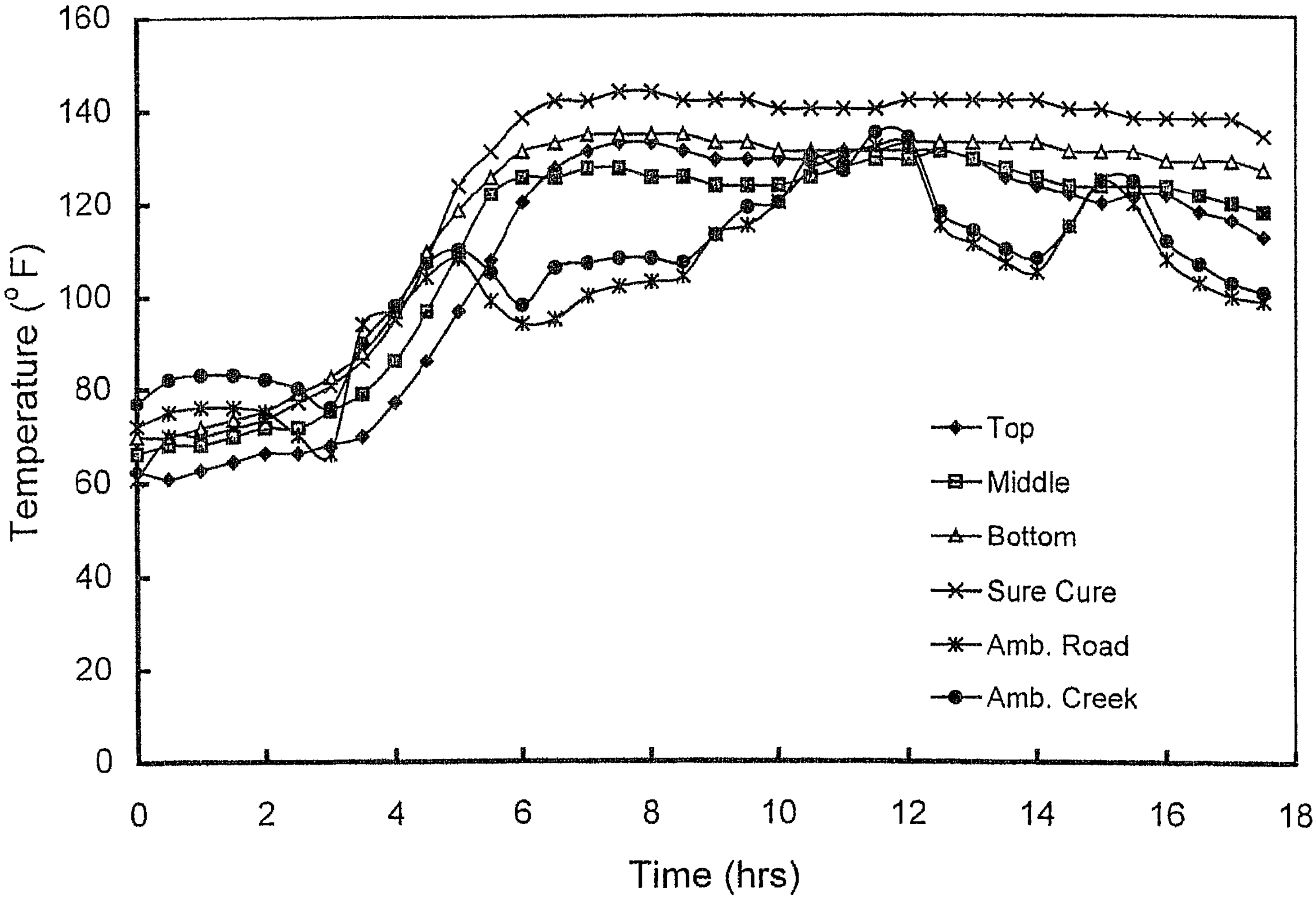


Figure B.3. Temperature History of Pour 3

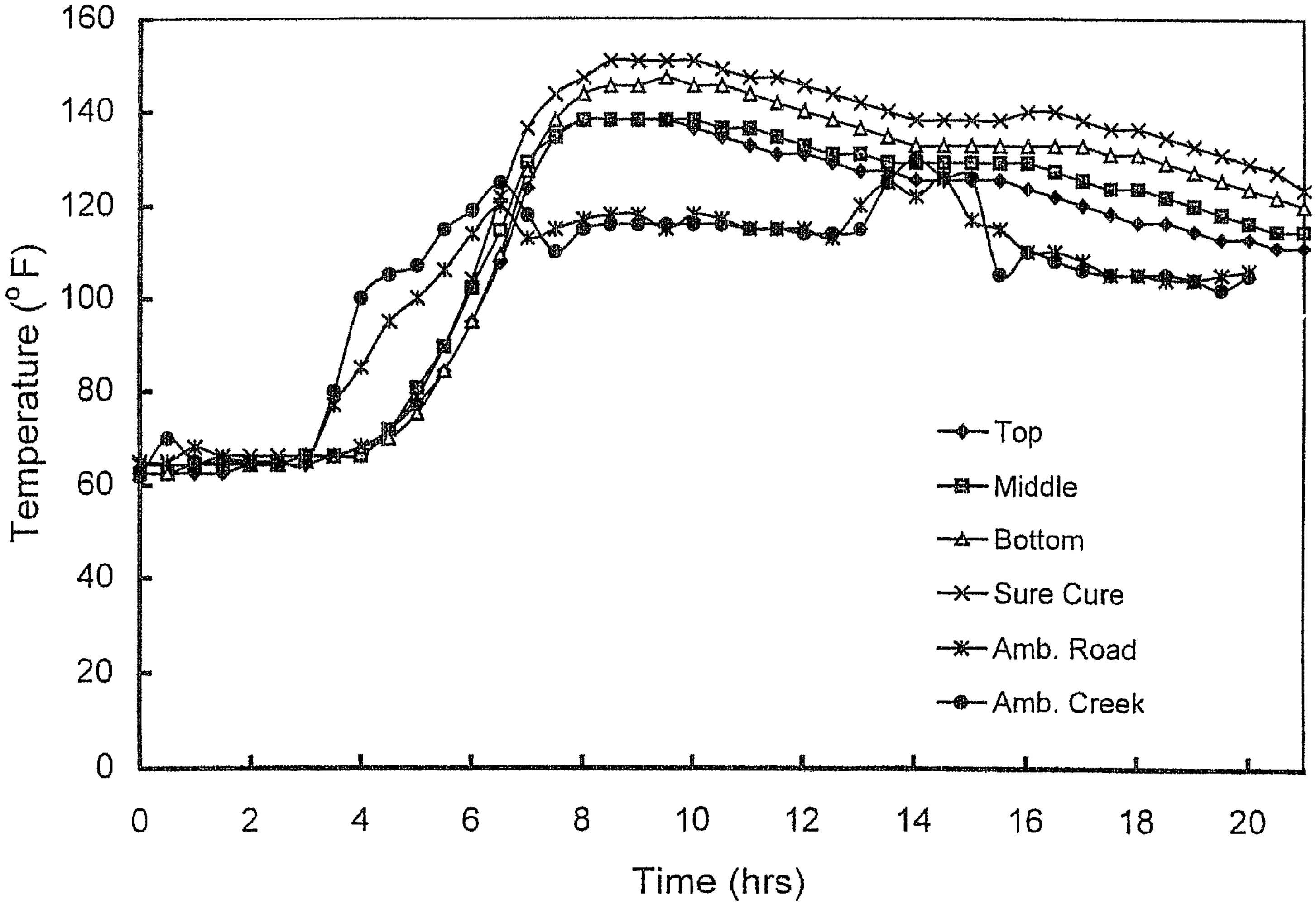


Figure B.4. Temperature History of Pour 4

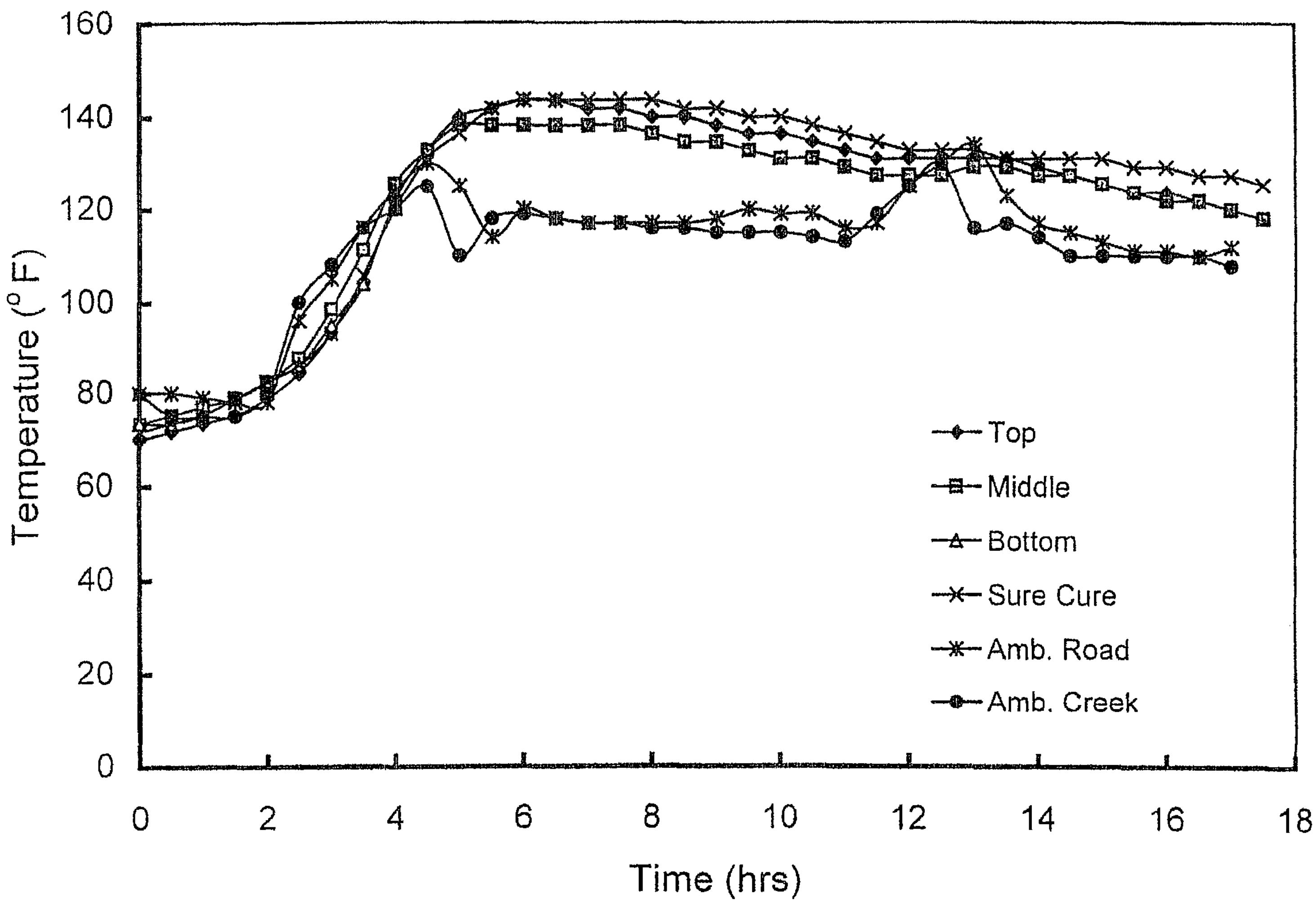


Figure B.5. Temperature History of Pour 5

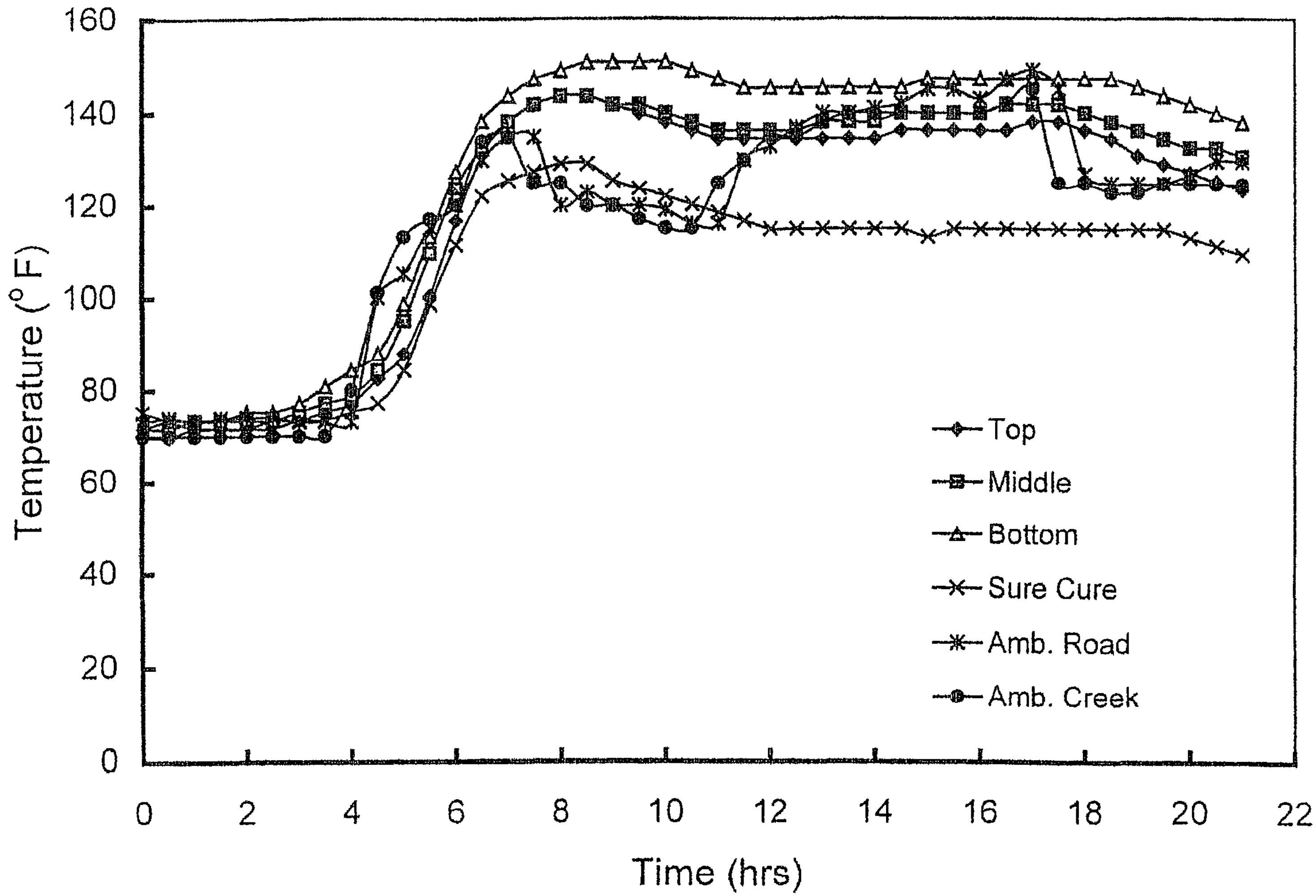


Figure B.6. Temperature History of Pour 6

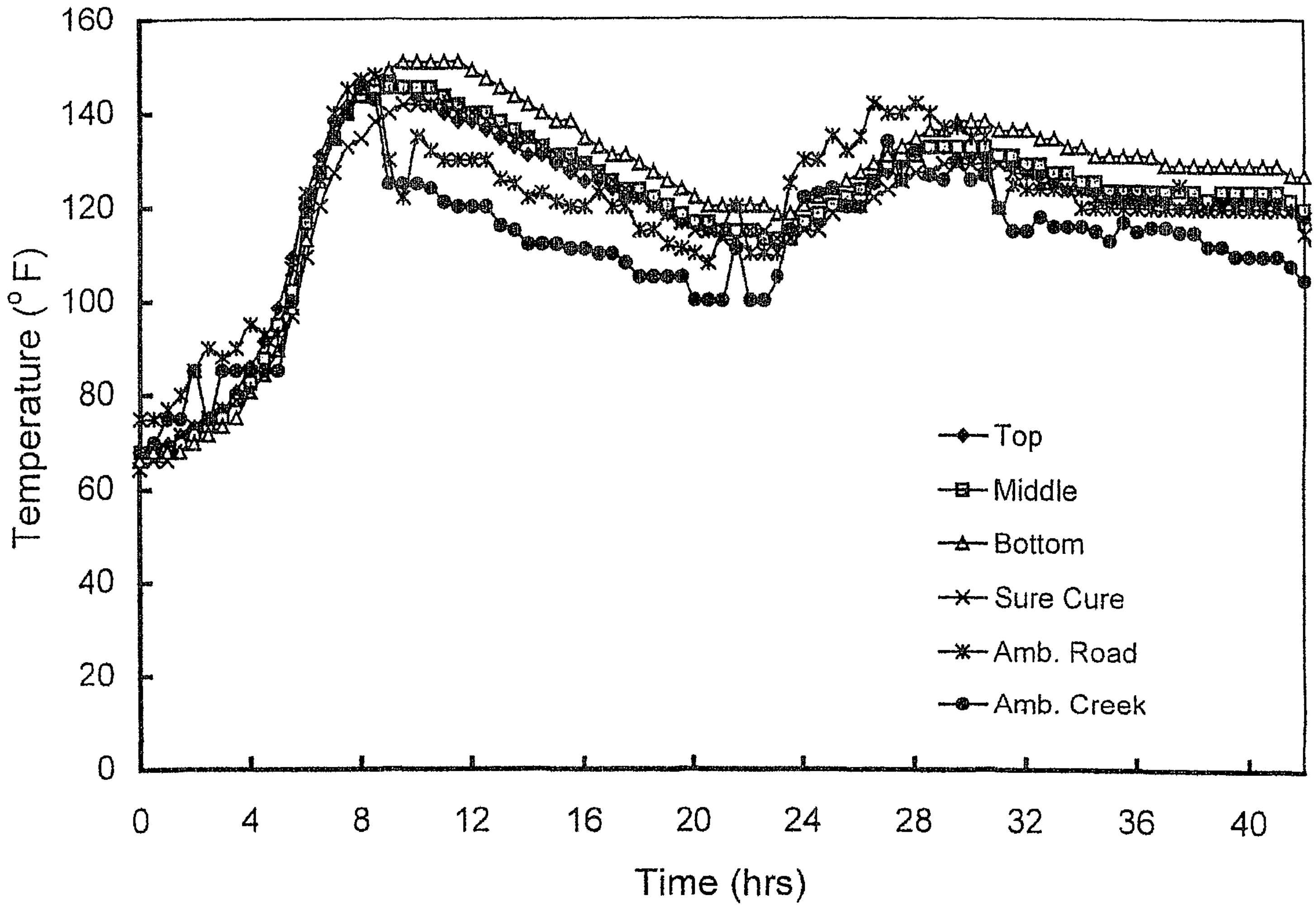


Figure B.7. Temperature History of Pour 7

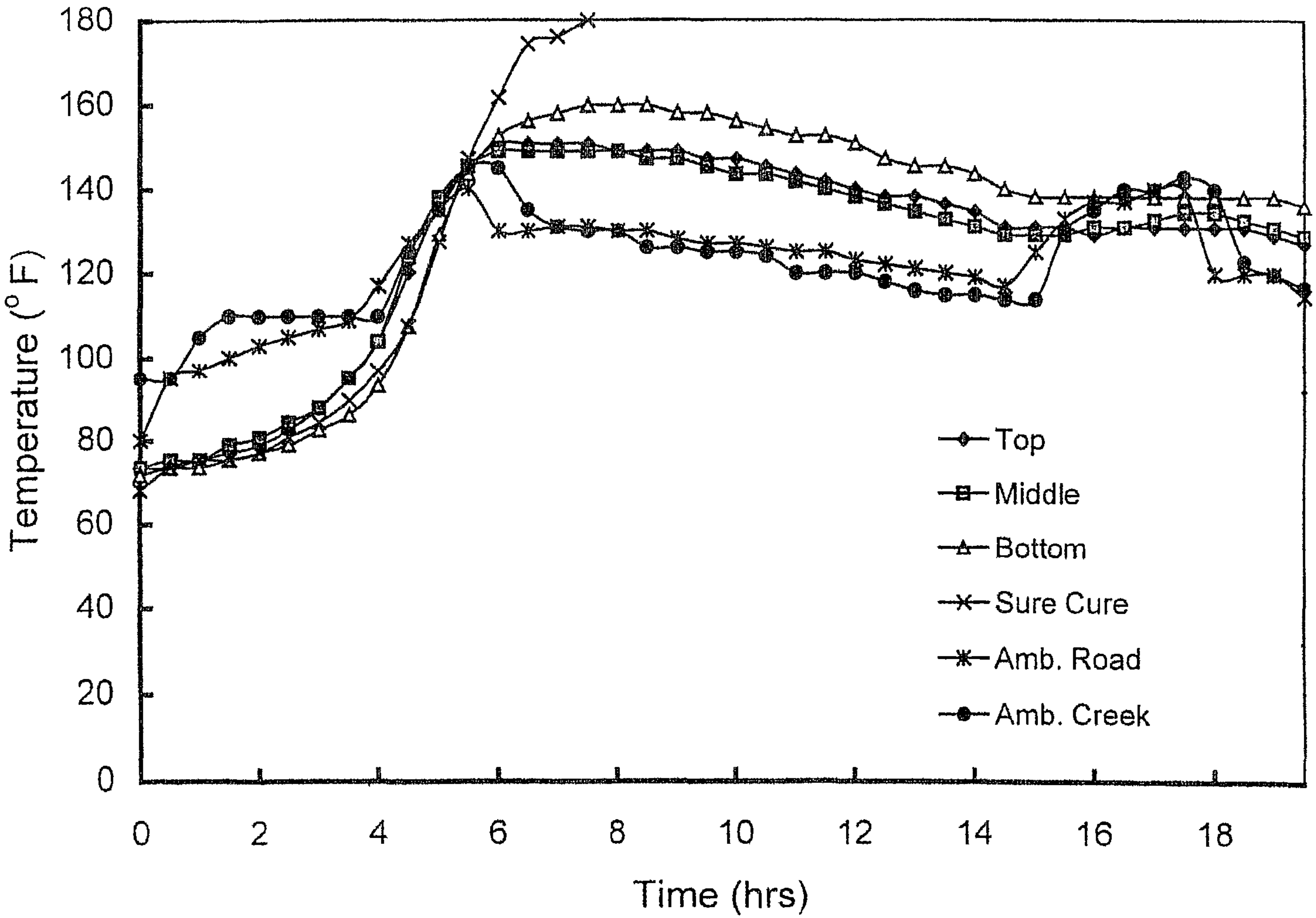


Figure B.8. Temperature History of Pour 8

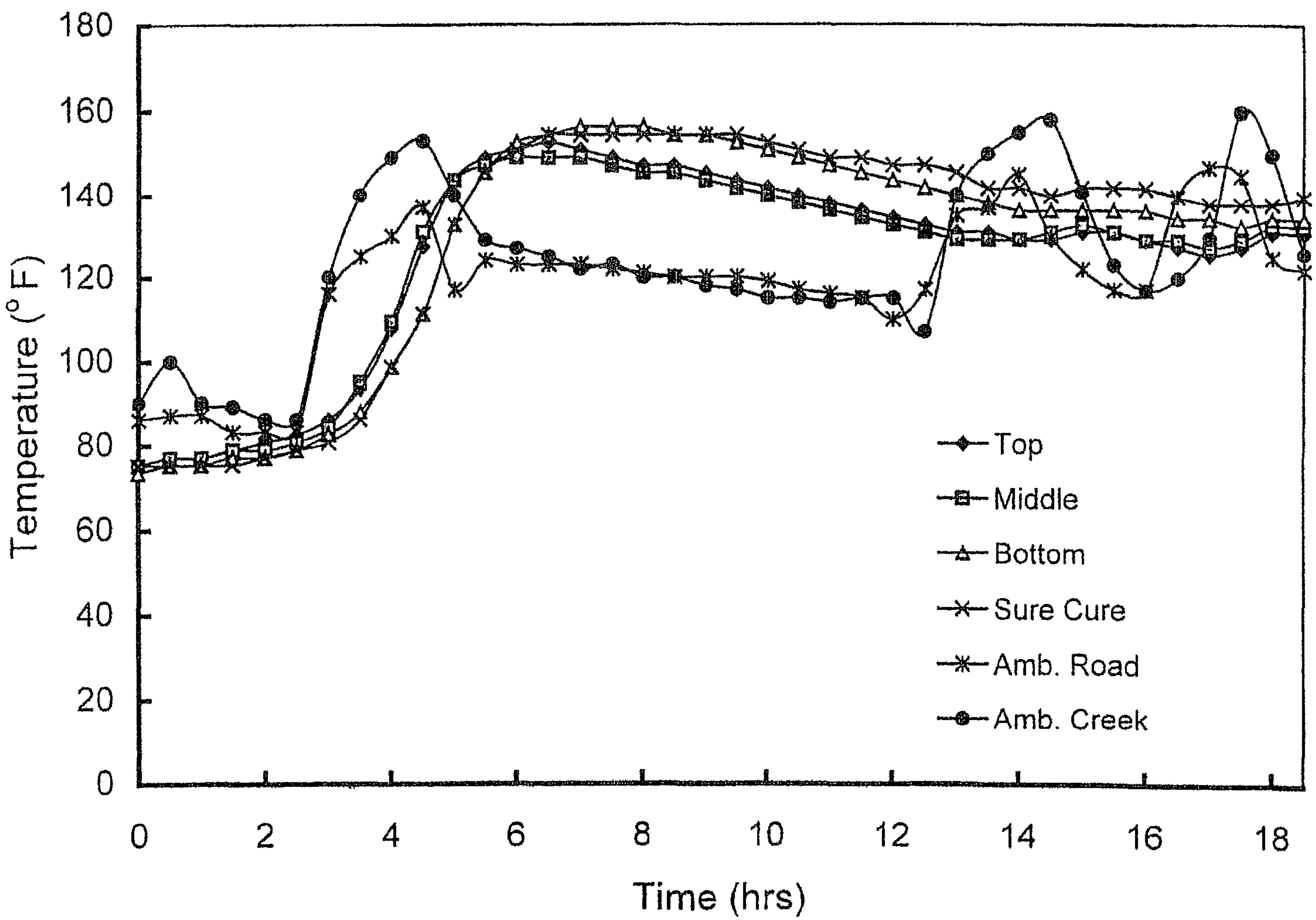


Figure B.9. Temperature History of Pour 9

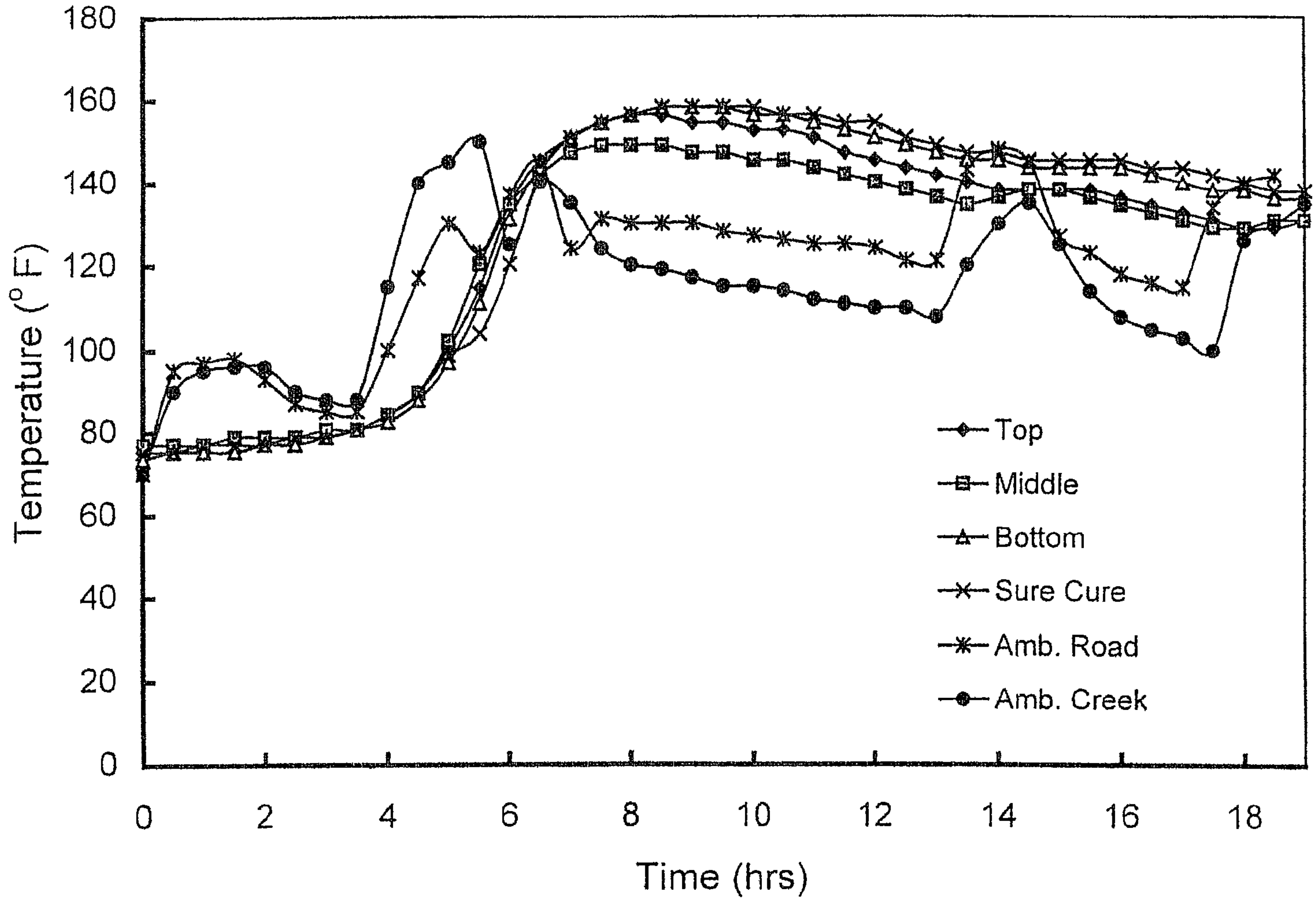


Figure B.10. Temperature History of Pour 10

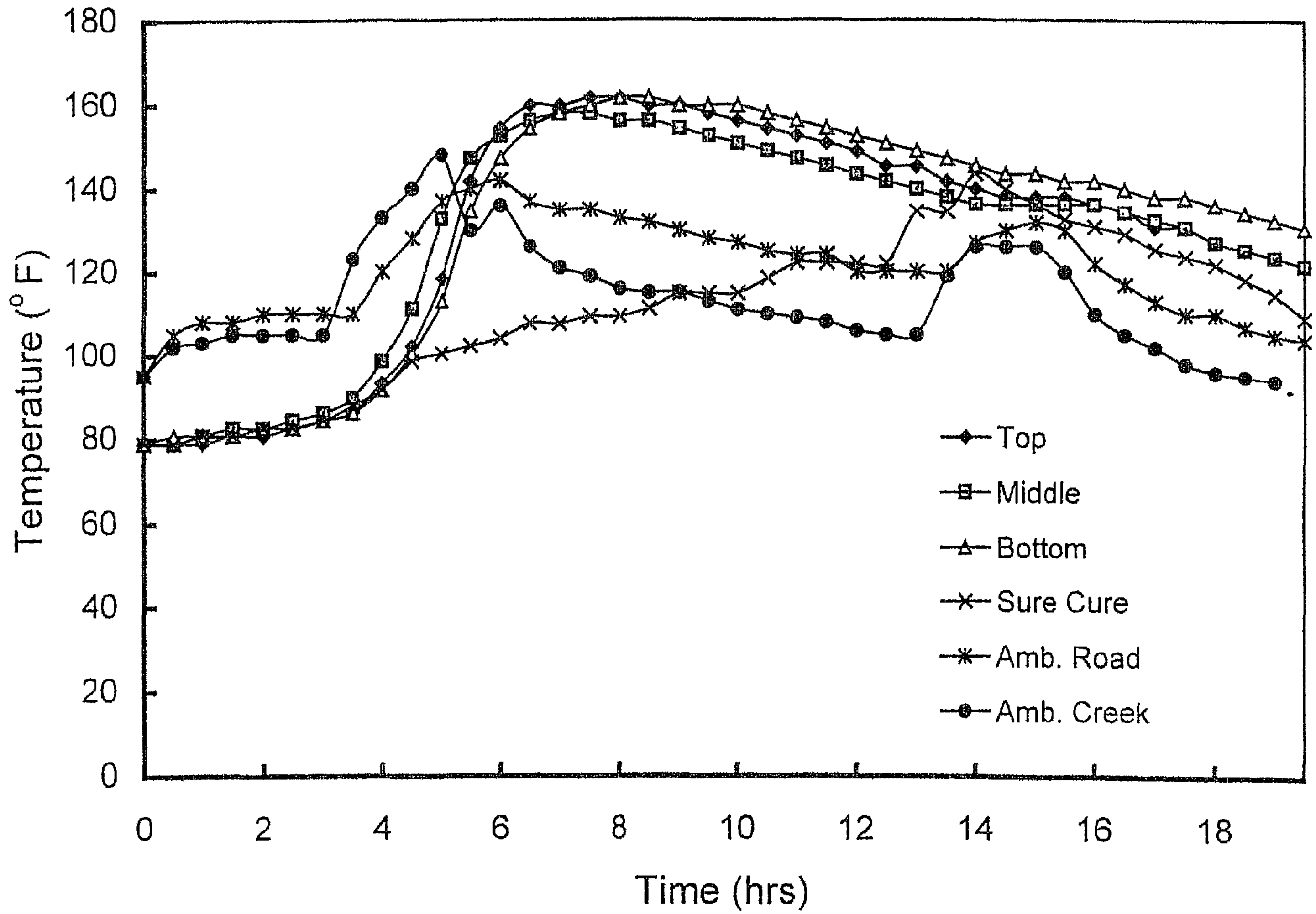


Figure B.11. Temperature History of Pour 11

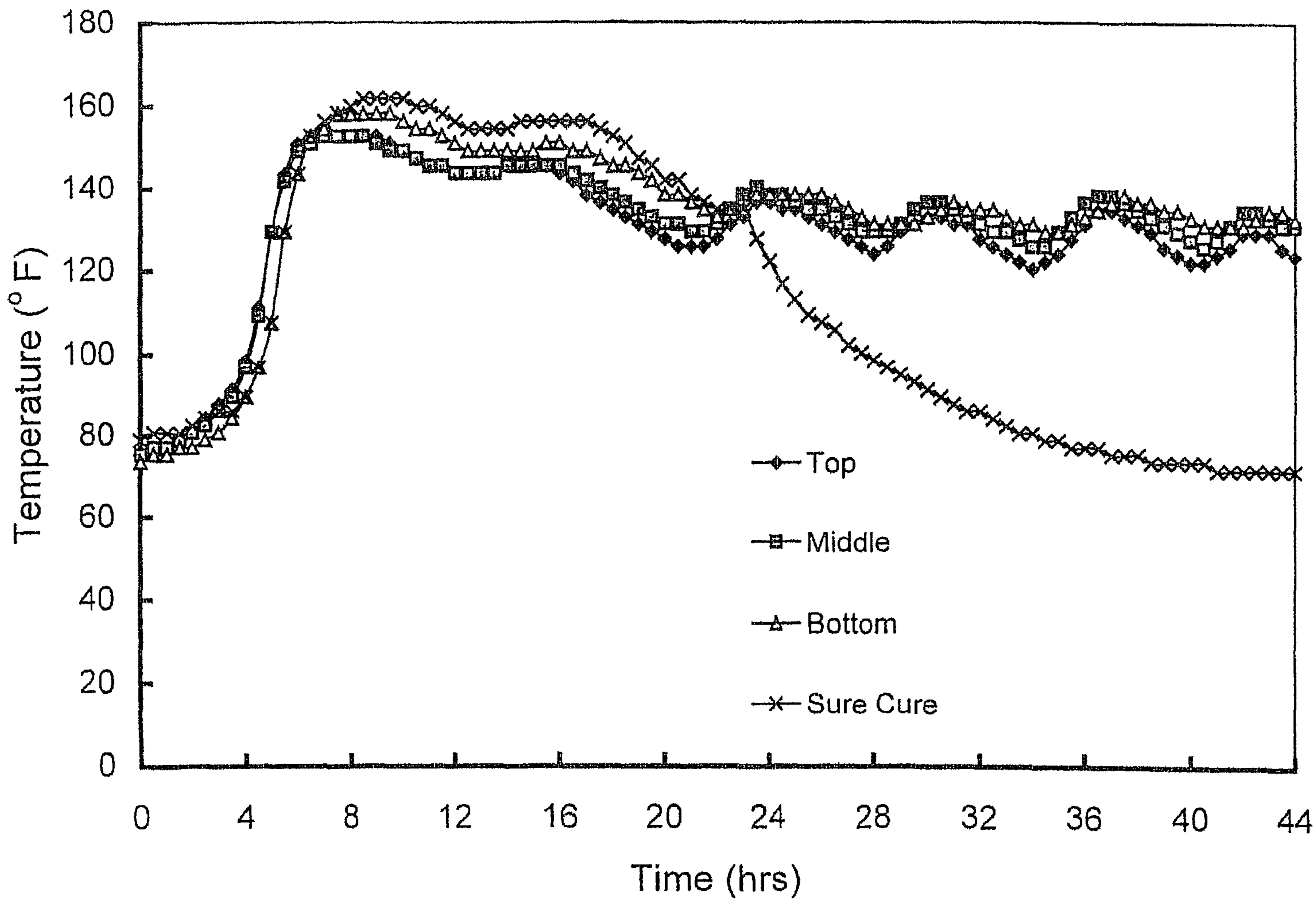


Figure B.12. Temperature History of Pour 12

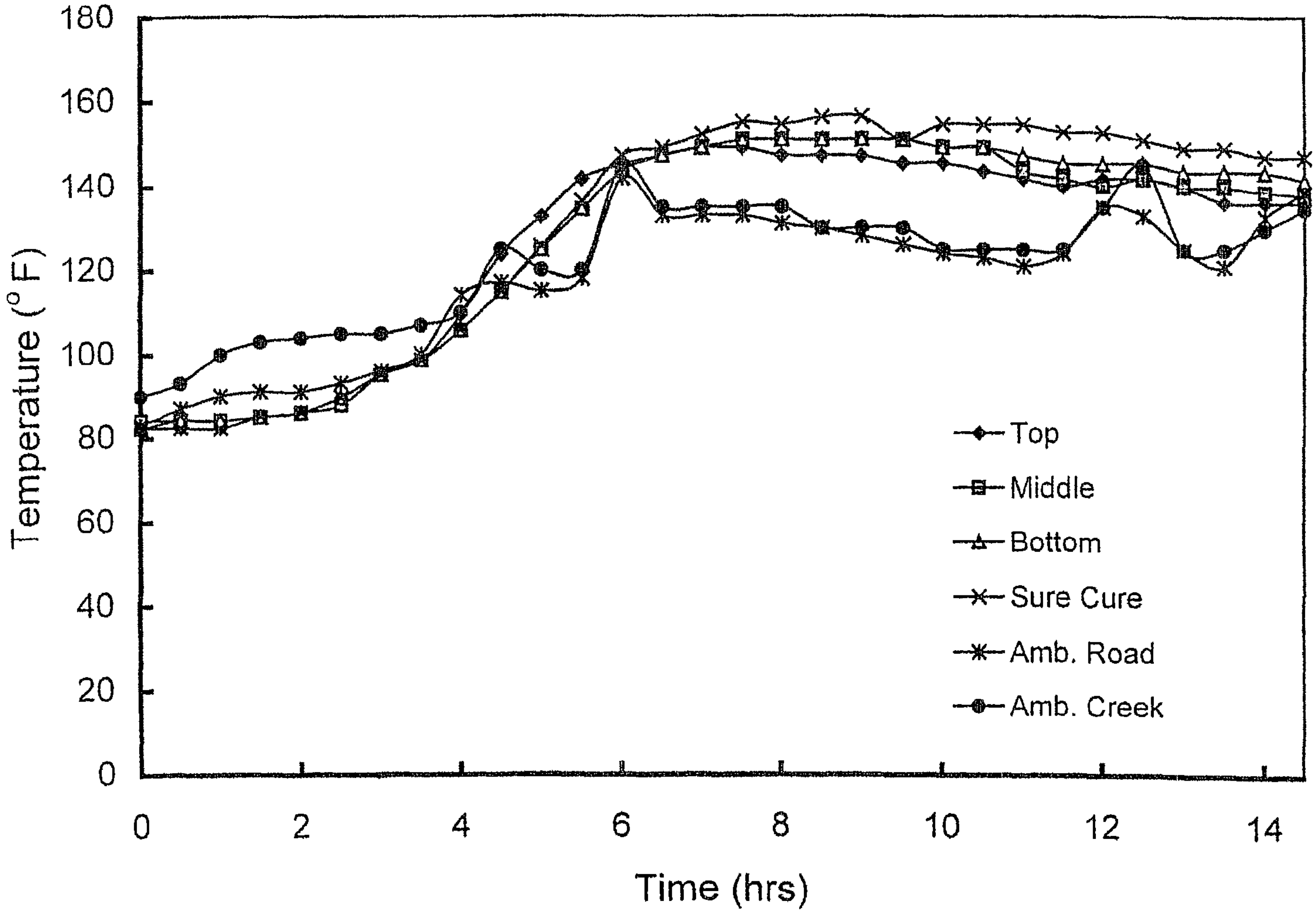


Figure B.13. Temperature History of Pour 13

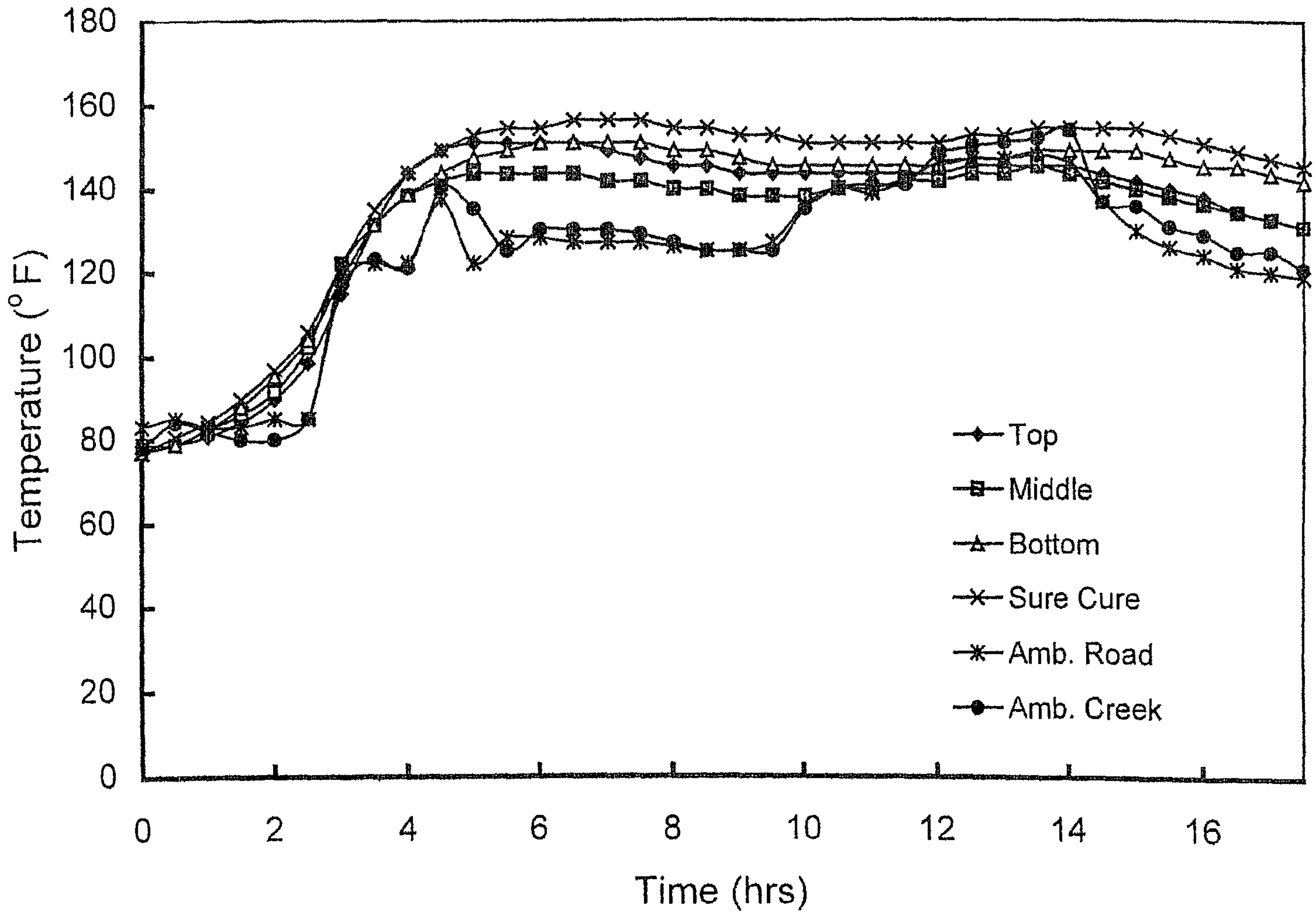


Figure B.14. Temperature History of Pour 14

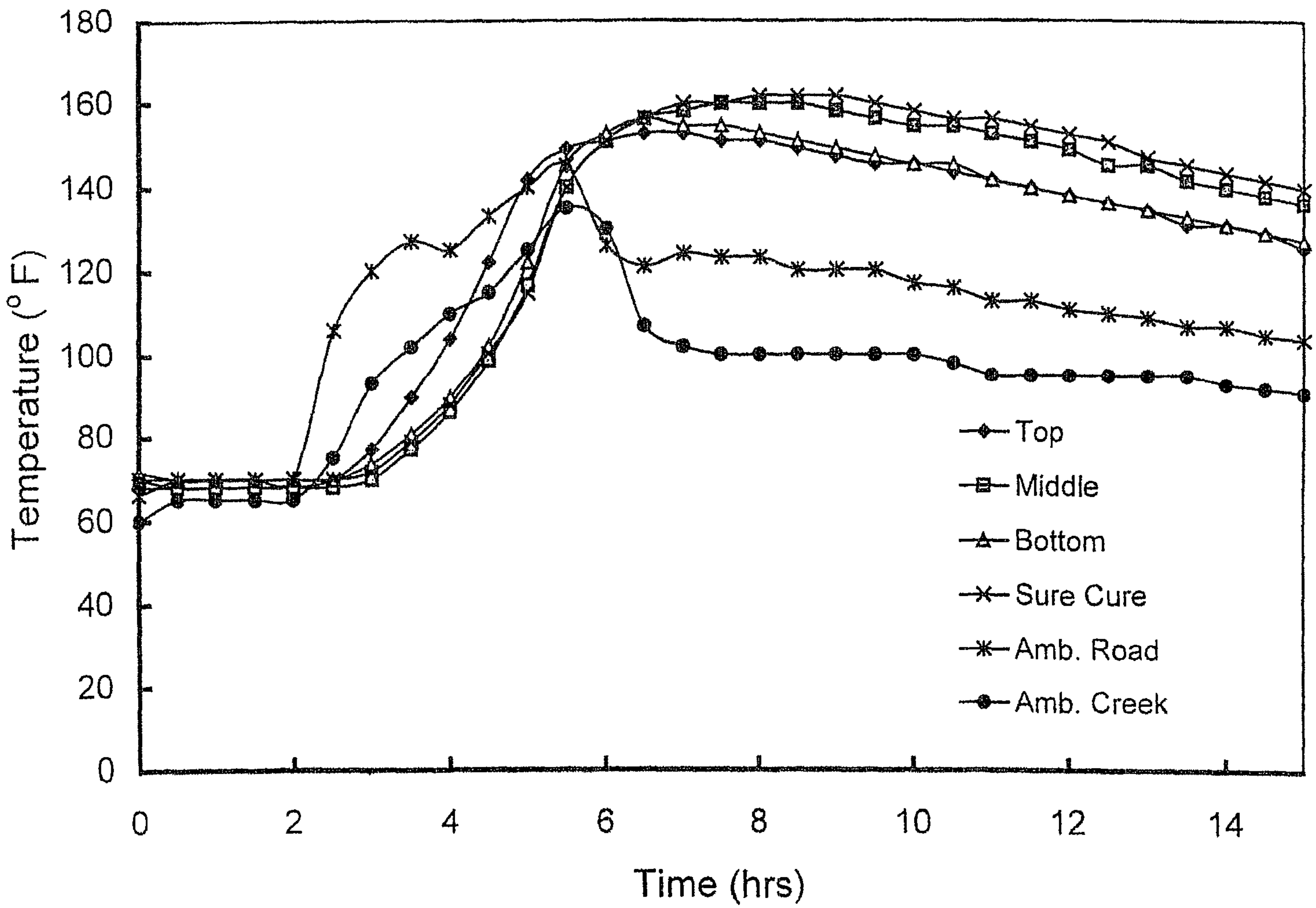


Figure B.15. Temperature History of Pour 15

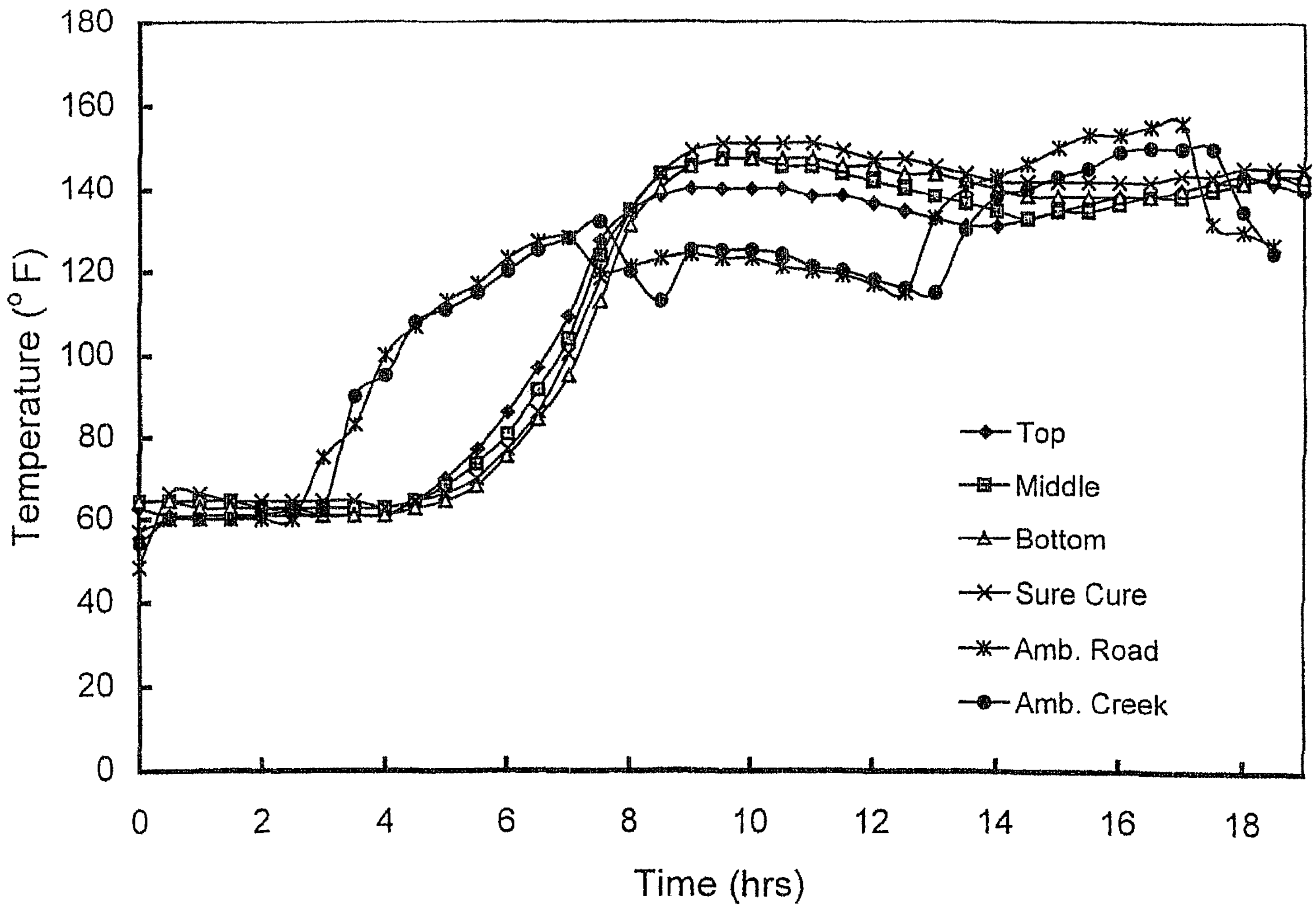


Figure B.16. Temperature History of Pour 16

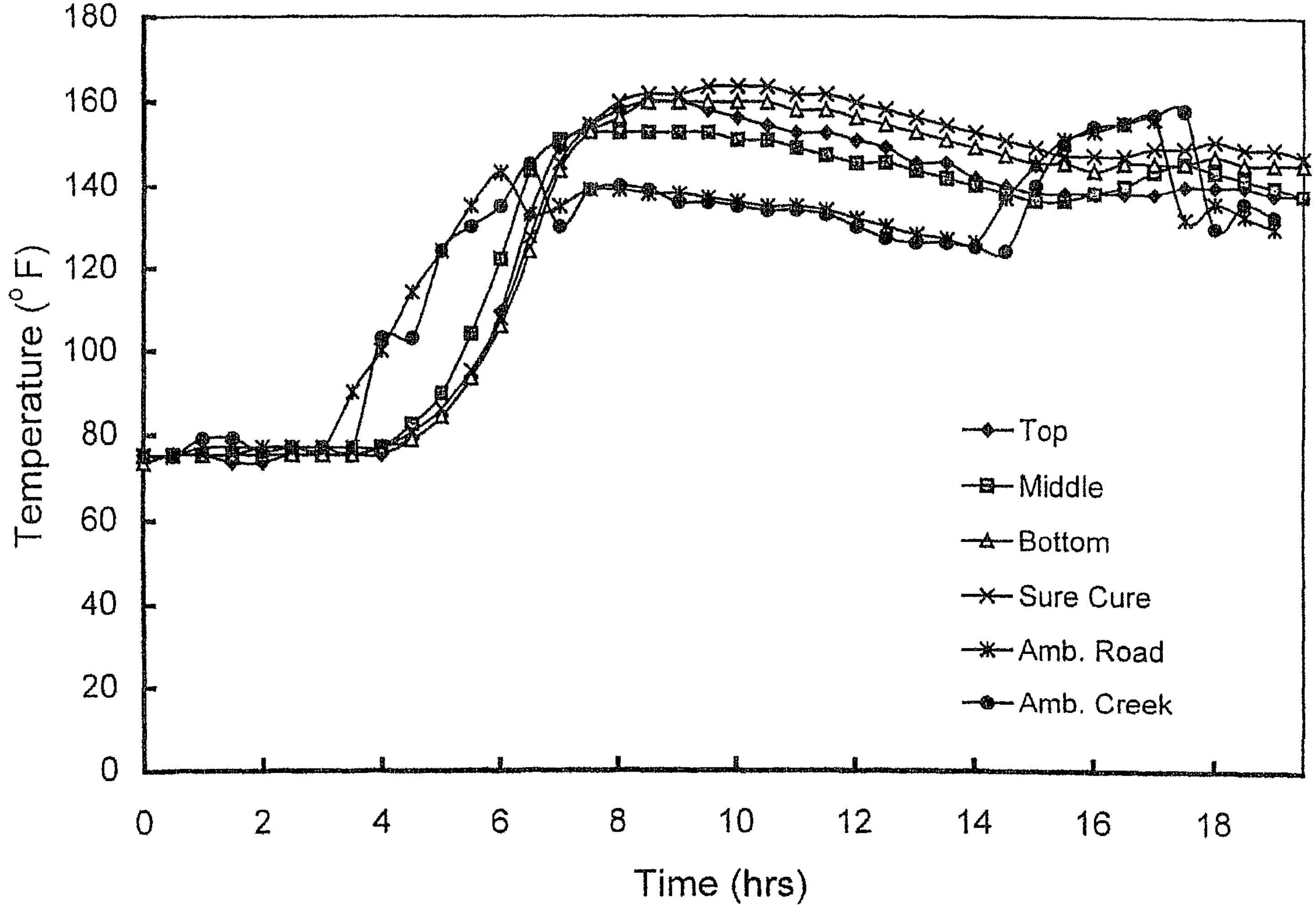


Figure B.17. Temperature History of Pour 17

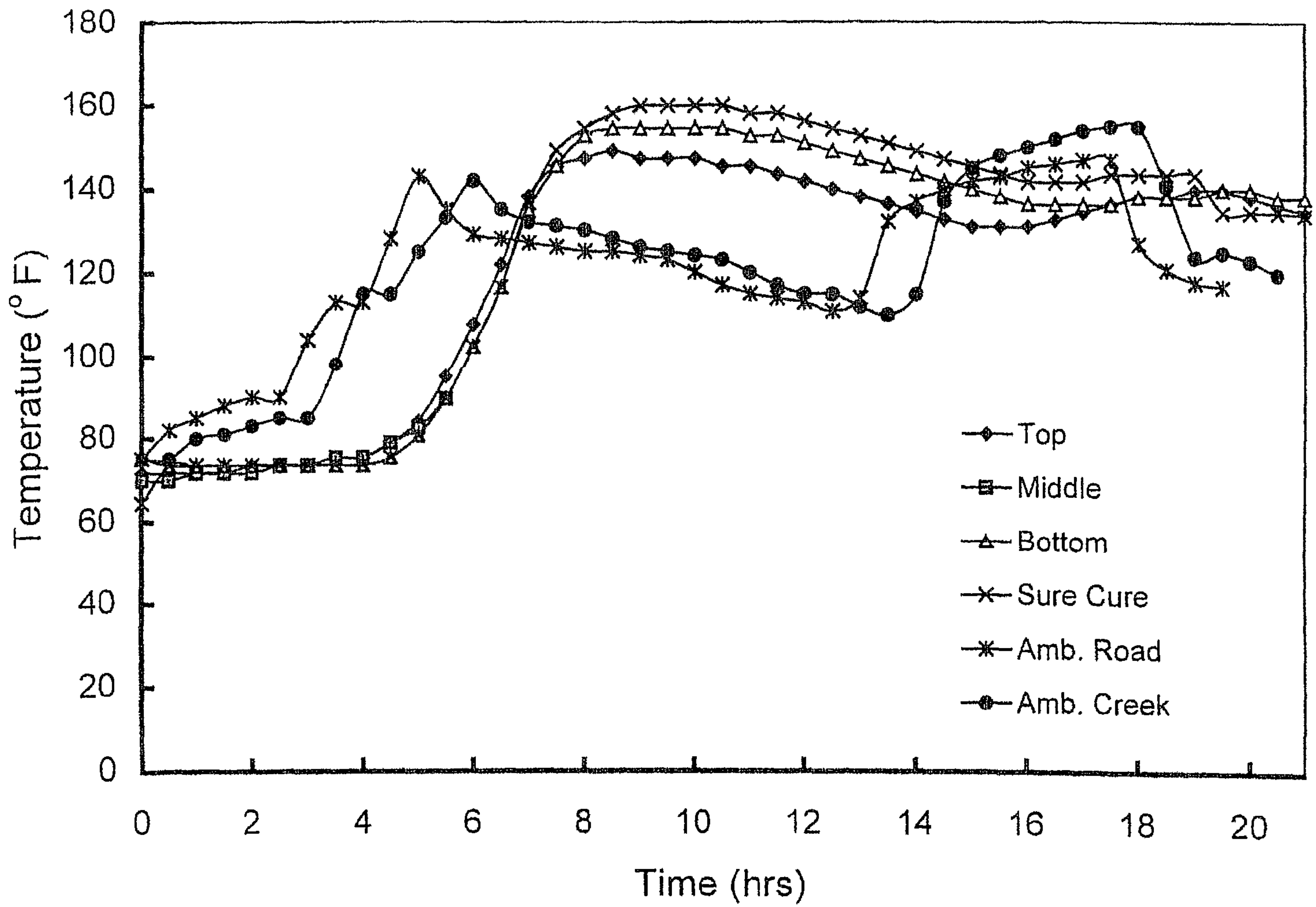


Figure B.18. Temperature History of Pour 18

APPENDIX C

PULL-OUT TESTS

INTRODUCTION

In 1974 the Concrete Technology Corporation (CTC) tested ½ in. diameter prestressing strands to determine two day “simple” (no pretension) pull-out strengths. The data collected by CTC in 1974 has been used as a benchmark for comparing strand performance. As part of Alabama’s HPC Bridge project, FHWA requested pull-out tests using the HPC girder concrete and the 0.6 in. diameter prestressing strand used in the HPC girders. The pull-out specimen described here was fabricated by Sherman Prestressed Concrete as part of the girder test pour.

OBJECTIVE

The tests preformed by CTC in 1974 and more recently by Rose and Russell (1992) were for ½ in. diameter strands. The tests conducted at Auburn University are for 0.6 in. diameter strands. The data allow comparison of the benchmark based on the ½ in. strands with results collected for strands of 0.6 in. diameter.

Strand Preparation and Inspection

Strands are cast in the as-received condition. The original test specifications call for the strands to be visually inspected and then wiped with a clean paper towel before casting. The intent of this inspection is to determine the extent of surface contamination (i.e. rust).

The strands tested in this experiment were inspected after arrival at Auburn University cast into the pull-out specimens as shown in figure C.1. Only the exposed ends of the strand could be inspected. Upon completion of the visual inspection a paper towel was wrapped around the strand and slid from top to bottom. Refer to Table C. 1 and Figures C.2a, C.2b, and C.2c for results of this procedure. The strand numbers shown in Table C.1 correspond to the 12 individual strands in the pull-out specimens block.

TEST SETUP AND PROCEDURE

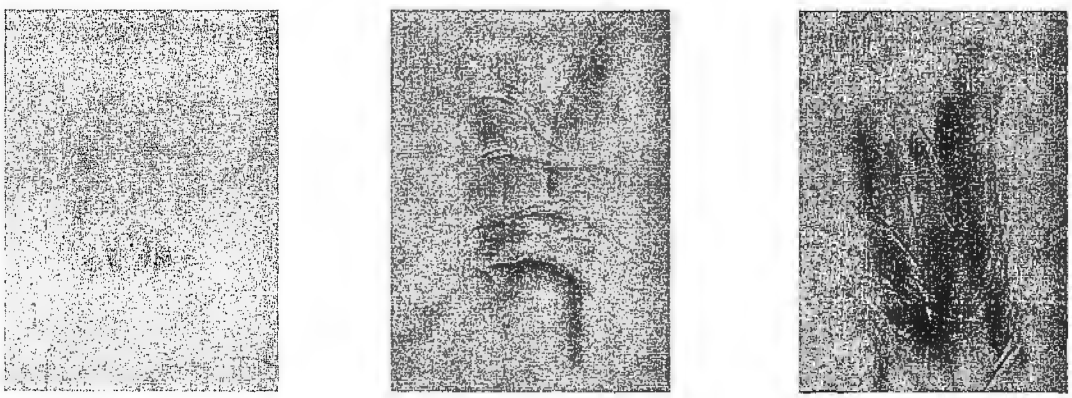
This test is termed a "simple" pull-out test as the strands are not pretentioned. To minimize effects of the jacking procedure on the concrete surface a 6 in. tall stand is used to support the jacking ram (see Figure C.3.). A 60 ton center hole ram is used to provide the jacking force.

Force provided by the ram is measured through monitoring of the system's (ram, hose, pump) hydraulic pressure. An Omega PX605 in-line pressure transducer and Omega DP25-E digital process meter measure the hydraulic pressure in the system. The ram, pressure transducer, and digital pressure meter were calibrated as a system preceding the test to eliminate any extraneous effects on data collection due to the measuring system components (i.e. ram friction).

Twin dial gauges are fitted above the jacking plate situated on the top portion of the ram. (see Figure C.3). The dial gauges have range to measure one inch of travel by the jacking plate beyond its initial position.

Table C.1. Surface Condition of Strands

Strand Number	Block A	Block B
1	Surface Rust	Very Light Rust
2	Light Rust	Surface Rust
3	Light Rust	Light Rust
4	Light Rust	Light Rust
5	Light Rust	No Rust
6	Very Light Rust	No Rust
7	Very Light Rust	Very Light Rust
8	Very Light Rust	Very Light Rust
9	No Rust	Very Light Rust
10	Very Light Rust	Surface Rust
11	No Rust	Light Rust
12	Very Light Rust	Light Rust



(a) (b) (c)

Figure C.2. Surface Conditions
(a) very light rust, (b) light rust, (c) surface rust

The test is commenced by setting up the components as seen in Figure C.3. The jacking plate is raised slightly to engage the dial gauge legs. The gripping chuck is lowered to contact the jacking plate. An initial pressure of 200 psi is introduced into the system to induce gripping by the chuck. The dial gauges and hydraulic system components are then zeroed and the test is begun.

Pressure is added to the system in 200 psi increments. While the pressure is held steady at each increment the jacking plate displacement is recorded from the twin dial gauges. This process continues to termination when one of the following occurs: One inch of travel is experienced by the jacking plate, the strand yield strength is reached, or fracture of wire(s) in the strand.

Three pieces of data are recorded during test: Load at which first movement of the strand is detected (approximate), displacement of the strand at maximum load (approximate), and maximum load supported by the strand.

TEST RESULTS

The original test specifications call for the test to be preformed on the strands at the two-day strength of the block concrete. The goal of the tests reported here was to perform the tests after essentially the full strength of the HPC was achieved. The tests at Auburn University were conducted 35 days after the pull-out specimen was cast. The 4 x 8 in. cylinder strength of the concrete was 11,850 psi.

A wire fractured in the first strand tested at a load of 45,000 lbs and a ram of 0.715 in. The dial gages were not used in subsequent tests. Slow continuous

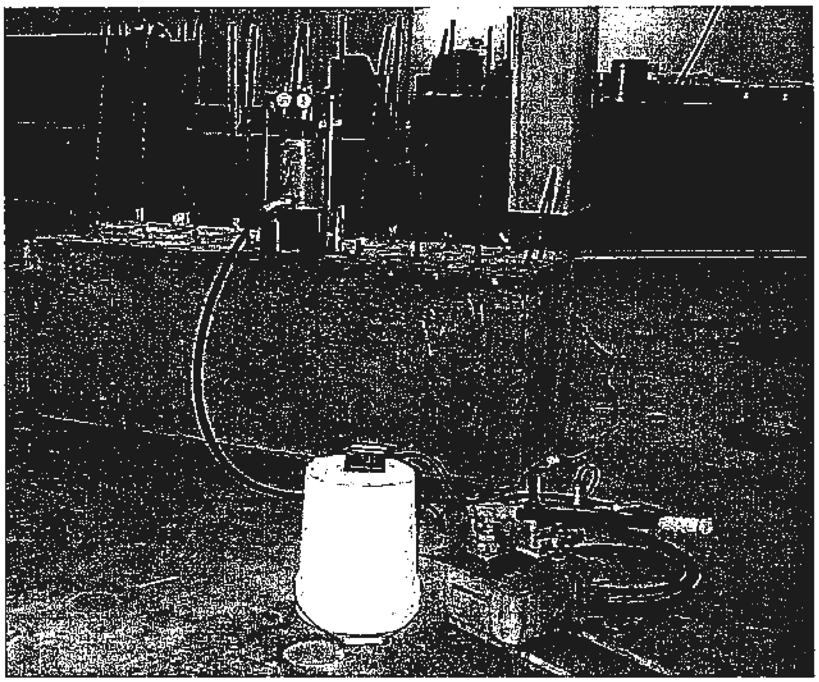


Figure C.1. Pull-Out Specimen Block and Test Set-Up

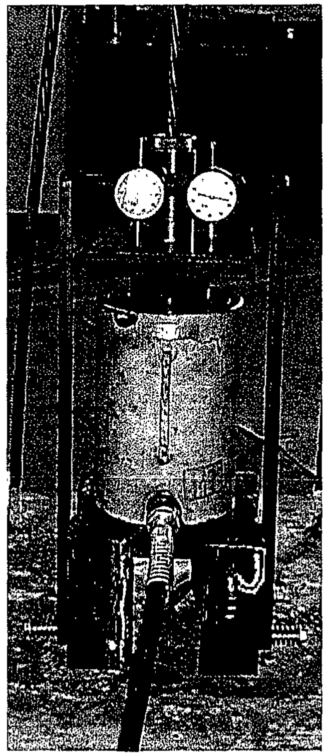


Figure C.3. Test Stand


Figure C.4. Fractured Strand

increase in hydraulic pressure was used in subsequent tests until wire fracture occurred. The fracture loads of the individual strand tests were: 45,000 lbs., 38,000 lbs., 46,800 lbs., 36,800 lbs., and 47,000lbs. Testing was stopped since it was apparent that no strand slips were likely.

CONCLUSIONS

The purpose of this test is to determine if the 0.6 in. diameter strands perform satisfactorily when compared to the results obtained by CTC. CTC tested samples after two days of concrete curing, tests here were conducted after 35 days. All of the strands tested by Auburn University failed by fracture of one or more wires, none failed by slippage. This is excellent bond performance.

During the High Performance Concrete Bridge Showcase in Houston, The University of Texas (1996) reported similar findings. After three days of curing the strands tested by Texas failed by fracture of one or more strands. These results indicate that the high strength of the concrete, both in early and late age, produces a very strong bond with the strands.

REFERENCES

Rose and Russell (1997). "Investigation of Standardized Tests to Measure the Bond Performance of Prestressing Strand." *PCI Journal*. July/August pp. 56-80.

University of Texas (1996). *SHRP High Performance Concrete Bridge Showcase, Proceedings*. March 25-27, Houston, Texas. Section 6 page 48.

APPENDIX D

PHOSPHATE TESTS

INTRODUCTION

Several "simple" pull-out tests on Grade 270 seven-wire strand were performed by Auburn University as part of Alabama's HPC Bridge Project. These tests included a determination of the phosphate coating weight on the strands used in the pull-out specimen. This report contains the method and results of the coating weight tests.

OBJECTIVE

Phosphate is used as a lubricant in the drawing of steel wire. The objective of the experiment performed at Auburn University is to determine the average weight of the phosphate coating on prestressing strands used in "simple" pull-out tests conducted during the fall of 1998.

OVERVIEW OF PROCEDURE

Small samples of the prestressing strand are subjected to a hot chromic acid (CrO_3) bath. The weight of the sample is recorded before and after the acid treatment. Approximately two minutes is required to remove the phosphate from the wire surface. The surface area of the sample is determined to allow for the calculation of the average phosphate coating weight per unit of surface area.

SAMPLE PREPARATION

A typical analytic balance does not read accurately beyond a load of 100g. Strands are cut using a grinding wheel cutter to lengths of 3.75 in. to produce sample weights in the 90 to 98 gram range. The ends of each of the seven wires belonging to a sample group are ground square and smooth (see Figure D.1.). Each sample group of seven wires is the weighed as a group and individually. Using a micrometer, the diameters of the seven wires are measured then recorded.

SURFACE AREA DETERMINATION

The outer wires in the prestressing strand are not perfectly straight. It is therefore difficult to accurately measure the length (nearest 0.01 in.) of each strand; yet this information is required for the calculation of each wire's surface area. In this experiment the length of each wire is determined indirectly from its weight.

By determining the density and diameter of each wire its length can be calculated. Using the analytic balance and a pipette accurate to the nearest 0.1mL the density of the strand steel is computed. The center wire in each strand is straight and allows for a check of the density-length procedure. Using calipers the lengths of several center wires are checked against the computed lengths using the diameter and density data. These checks reveal an accuracy of ±0.05 in. When calculating the surface area for each wire only the cylindrical surface area is used. The surface area provided by the wire ends is not accounted for.

Acid Bath Preparation:

Anhydrous chromium trioxide (CrO_3) is diluted with water to yield a solution of molarity 2.0 (mol/L). For each sample group 200mL of the acid

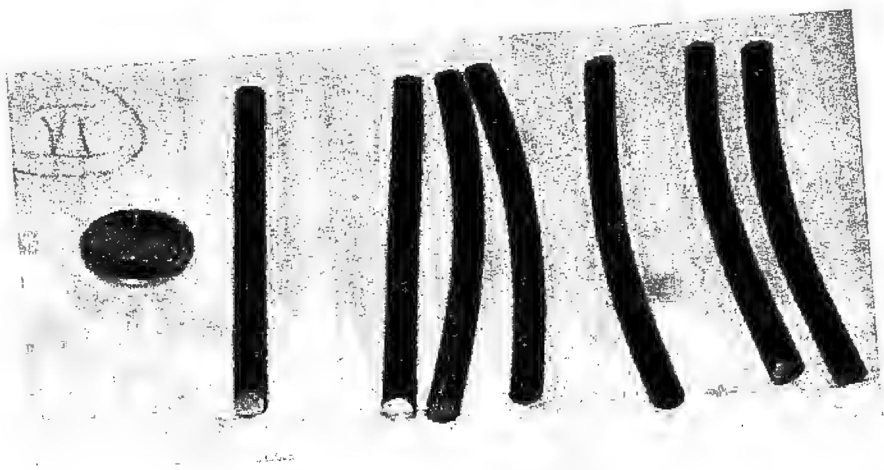


Figure D.1. Samples

solution is heated to 85°C. The hot acid bath will rapidly remove the phosphate coating and subsequently start removing metal from the samples. The removal of metal from the sample is very slow and occurs at a constant rate.

The Wire Association International (1989) recommends a bath time of two minutes. A trial group is used to verify the suggested bath time of two minutes. The trial group is placed in the bath and then removed periodically for weight determination. A graph of the weight and time reveals a point at which phosphate removal is complete (see Figure D.2). The transition from phosphate removal to metal removal occured at the anticipated two-minute mark as shown in Figure D.2.

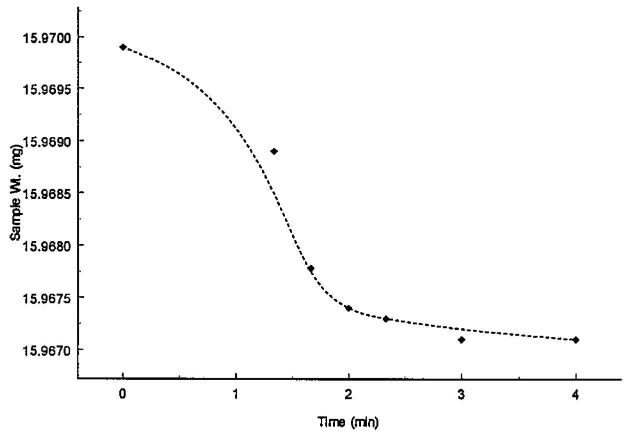


Figure D.2. Phosphate Removal with Time

TEST PROCEDURE

A 200mL acid solution is brought to a temperature of 85°C. After weighing, the seven-wire sample is placed in the acid bath using stainless steel crucible tongs. The samples are treated in the bath for two minutes and then removed. The samples are cleaned with water after removal and then placed in acetone. Each wire is removed from the acetone and set out to dry. The sample is weighed after air-drying.

TEST RESULTS

Six samples of seven wires were tested. The average coating weight for each sample is shown in Table D.1. The Average coating weight for all samples is determined to be 185.36 mg/ft^2.

Table D.1. Average Coating Weights Determined for Seven Wires.

Sample Number	Coating Weight (mg/ft^2)
1	185.71
2	187.48
3	220.30
4	187.61
5	162.11
6	168.95

Figure D.3. Test Set-Up

CONCLUSION

The method for calculating the length and surface area of the test samples strongly influences the final test results. Because of the difficulty in obtaining accurate lengths of the outer wire specimens, the wire density was used to calculate a length for each specimen. This method produces length estimations accurate to ±0.05in. In calculating the surface area of each wire the end areas were ignored. This procedure results in an average coating weight of 185.36mg/ft^2.

REFERENCES

The Wire Association International (1989). Ferrous Wire.

APPENDIX E

TEMPERATURE HISTORIES FROM BENT CAPS AND DRILLED SHAFTS OF THE UPHAPEE CREEK BRIDGE

DESCRIPTION OF TEMPERATURE MEASUREMENTS

Concrete temperature histories were measured for two drilled shafts and two bent caps in the Uphapee Creek Bridge. The temperatures were recorded using thermocouples and a four channel concrete maturity meter. The concrete placed in the drill shafts was not HPC. The concrete placed in the bent caps was the HPC-4 mix described in Chapter 5. The temperature data collected has limited use at the time of this writing. However, these temperature histories may prove to be of interest in future research related to heat buildup in large structural members and to strength gain at elevated temperatures.

DRILLED SHAFTS

Thermocouples were placed at the four locations. One thermocouple was in air and measured the ambient temperature around the top of the shaft. Three thermocouples were placed approximately 4 ft down into the fresh concrete measured from the top of the drilled shaft. The permanent drilled shaft casing had a 66 in. diameter. The outside diameter of the circular ties around the longitudinal shaft reinforcement was 54 in. One thermocouple was placed at the outside of the circular ties, one was placed at the center of the shaft, and one was placed half way between those two.

On January 8, 1999 the first drilled shaft was instrumented. The maximum internal temperature of the concrete was 125 °F, and it occurred approximately 45 hours after placement. Temperature histories for the first 90 hours after placement are shown in Figure E.1.

On January 15, 1999 the second drilled shaft was instrument. The maximum temperature in the drilled shaft was 135 °F, and it occurred approximately 45 hours after placement. Temperature histories for the first 140 hours after placement are shown in Figure E.2.

BENT CAPS

Thermocouples were placed at the four locations. One thermocouple was in air and measured the ambient temperature around the bent cap. Three thermocouples were placed inside each bent cap over the center of one of the columns. Each bent cap had a 5.75 ft square cross section, and the nearest free surface to each thermocouple was the top of the cap. One thermocouple was placed 3 in. below the top surface, one was placed 17.25 in. from the top surface, and one was placed 34.5 in. from the top surface (at the center of the cap). These measurements were made in October and November of 1999.

The first measurements were made at Bent 6. The temperature histories are shown in Figure E.3 for the first 200 hours after placement. The maximum recorded temperature was 189 °F, and it occurred approximately 9.5 hours after placement. This maximum temperature was measured at the center of the bent cap.

The second measurements were made at Bent 5. Also, concrete was placed into a hydration chamber and brought back to Auburn. The maximum temperature measured in the cap was 169 °F, and it occurred approximately 37 hours after placement. This maximum temperature was measured at the location 17.25 in. below the top surface. Unfortunately the thermocouple at the center of the bent cap malfunctioned. Based on the results from Bent 6, it is likely that the temperature at the center of the bent cap was higher than the maximum recoreded temperature. Temperature histories for the first 200 hours after placement are shown in Figure E.4. The maximum temperature recorded during hydration of the specimen in the hydration chamber was 176°F at approximately 26 hours after placement.

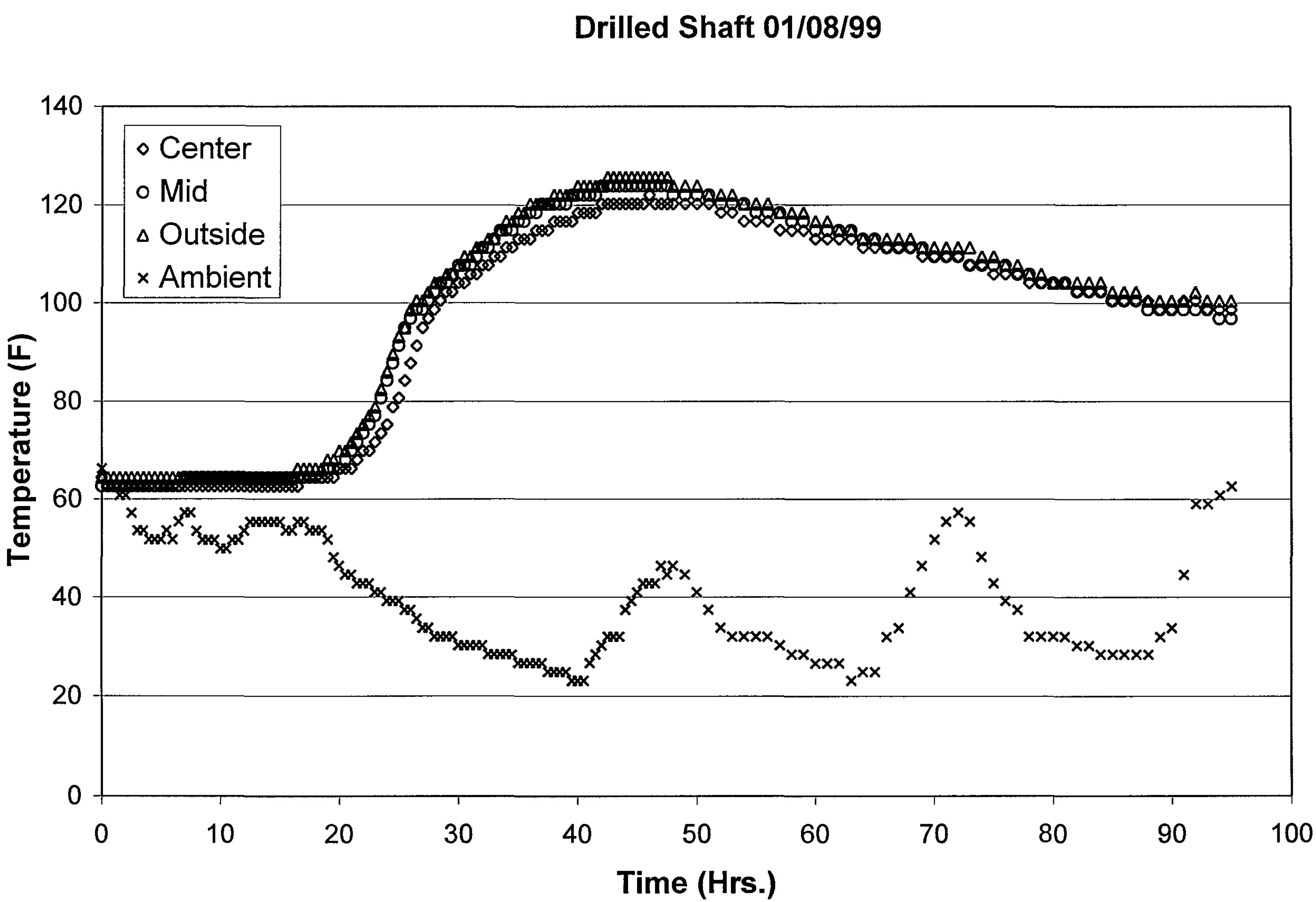
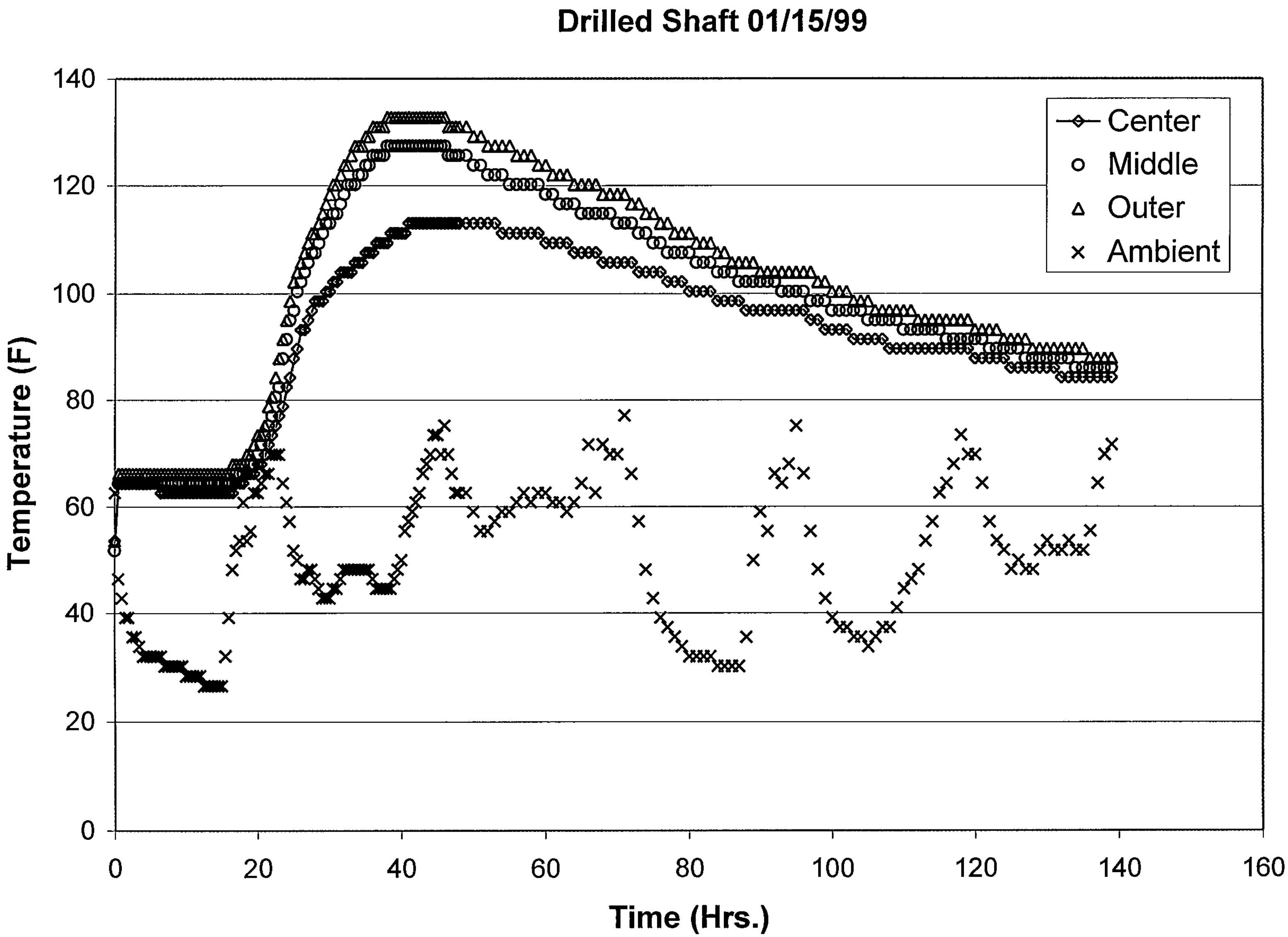

Drilled Shaft 01/08/99
Center
Mid
Outside
Ambient
Temperature (F)
Time (Hrs.)
0
20
40
60
80
100
120
140
0
10
20
30
40
50
60
70
80
90
100

Figure E. 1.



Drilled Shaft 01/15/99
Center
Middle
Outer
Ambient
Temperature (F)
Time (Hrs.)
0
20
40
60
80
100
120
140
160

Figure E.2.

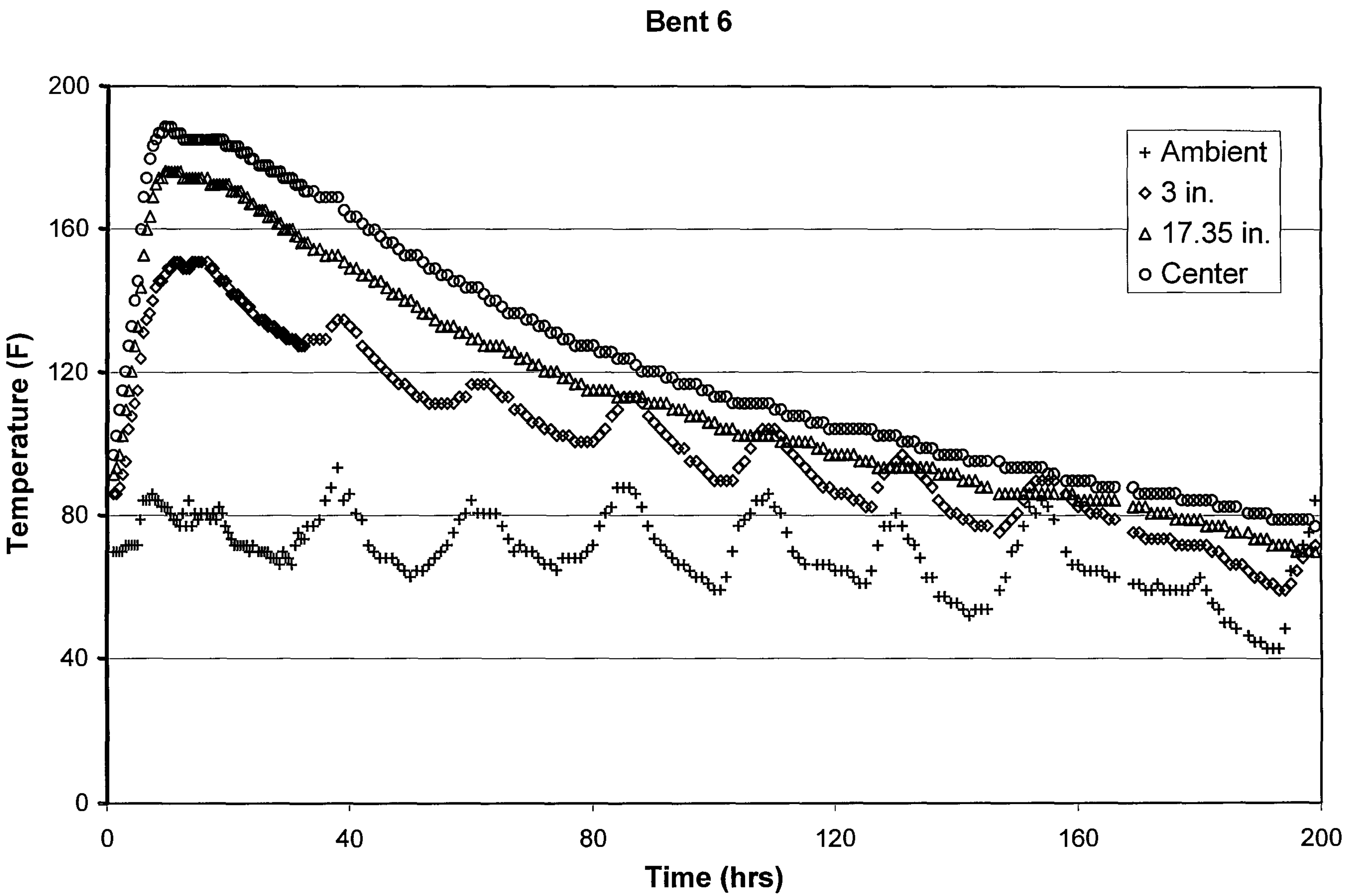
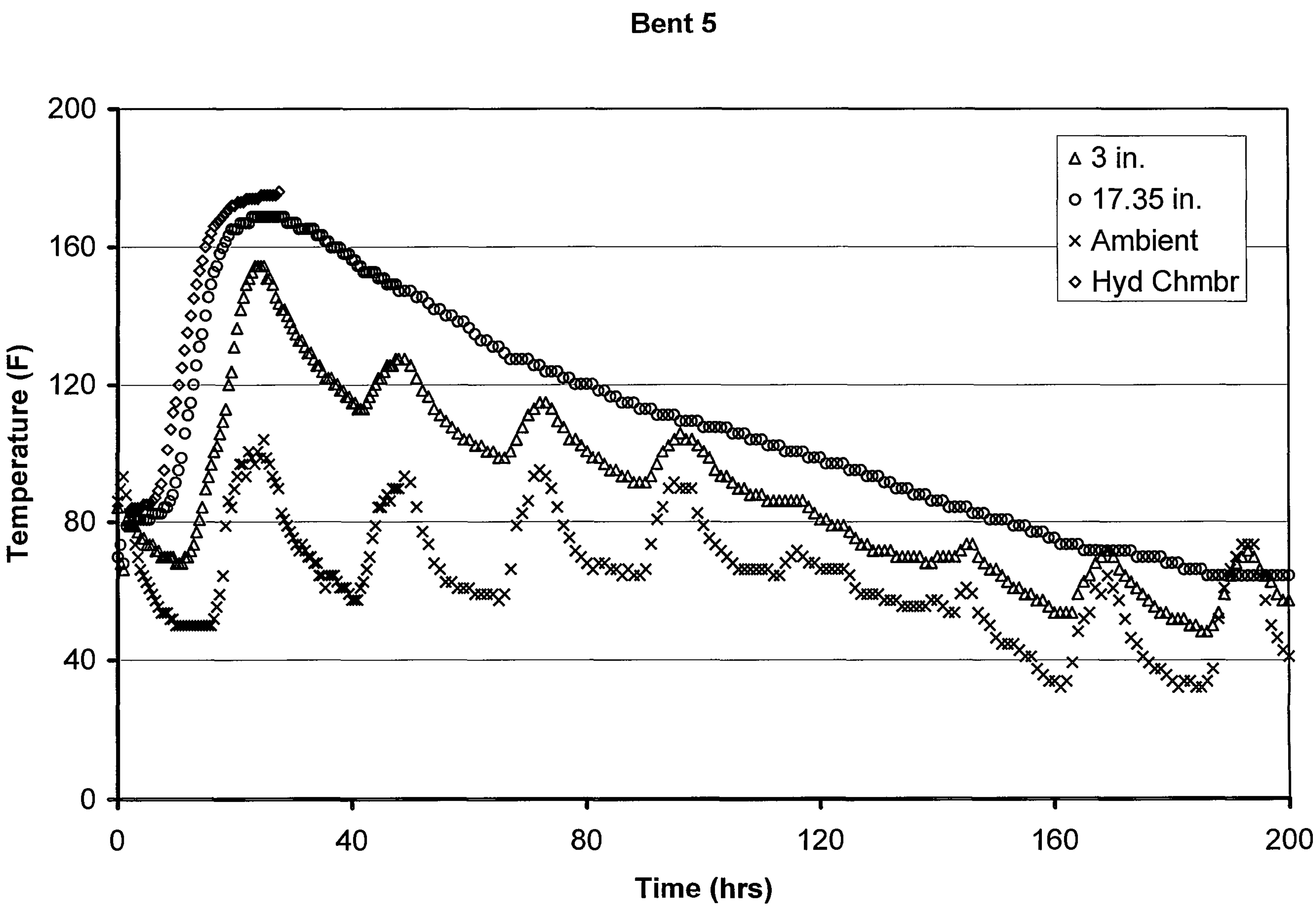

Bent 6
Temperature (F)
Time (hrs)
+ Ambient
◇ 3 in.
△ 17.35 in.
○ Center
0
40
80
120
160
200
0
40
80
120
160
200

Figure E.3.



Bent 5
200
160
120
80
40
0
Temperature (F)
0
40
80
120
160
200
Time (hrs)
3 in.
17.35 in.
Ambient
Hyd Chmbr

Figure E.4.