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INVESTIGATING THE NEED FOR AND EFFECT OF AIR ENTRAINMENT IN HIGH PERFORMANCE CONCRETE

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INVESTIGATING THE NEED FOR AND EFFECT OF AIR ENTRAINMENT IN HIGH PERFORMANCE CONCRETE

A Dissertation APPROVED FOR THE SCHOOL OF CIVIL ENGINEERING AND ENVIRONMENTAL SCIENCE

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ABSTRACT

The use of high performance concrete (HPC) in exterior structures has increased in recent years. Its increased strength and durability make HPC very appealing to the prestressed concrete industry, particularly in bridge girders. Due to its increased strength and durability, HPC can reduce the number of girders, increase bridge spans, decrease bridge depth, and improve bridge durability. The research program investigated the need for air entrainment in HPC and its effect on HPC bridge girders. It is necessary to entrain air in some concrete structures that are subjected to freezing and thawing. Current building codes require varying amounts of entrained air depending on the severity of the exposure. Entrained air voids provide air pockets where water can expand and water pressure can be relieved. Without these voids, continuous freeze-thaw cycles will eventually degrade and damage the concrete. A total air content between four and eight percent is generally considered adequate to provide resistance to the freeze-thaw action. Many researchers suggest that entrained air may not be necessary in HPC because of its low permeability, and because of its inherent unsaturated condition. One of the primary objectives of the research program is to determine whether air entrainment is necessary in HPC. Results from the program show that air entrainment may not be necessary for w/cm less 0.36, and a total air content of four percent may be sufficient for w/cm between 0.36 and 0.50.

CHAPTER 1

INTRODUCTION

1.1 INTRODUCTION

The research program investigated the need for air entrainment in high performance concrete (HPC) and its effect on HPC bridge girder performance. It is necessary to entrain air in some concrete structures that are subjected to freezing and thawing. Entrained air is added to concrete through the use of air entraining agents (AEA). Current building codes require varying amounts of entrained air depending on the severity of the exposure. Entrained air voids provide air pockets where water can expand upon freezing and water pressure can be relieved. Without these voids, continuous freeze-thaw cycles will eventually degrade and damage the concrete. For all concrete structures, a total air content between four and eight percent is generally considered adequate to provide resistance to the freeze-thaw action (Mindess et al, 1981). The Oklahoma Department of Transportation requires a total air content of 5 ± 1.5 percent in bridge girders.

The use of HPC in exterior structures has increased in recent years (Neville et al, 1998). Its increased strength and durability make HPC very appealing to the prestressed concrete industry, particularly in bridge girders. Due to its increased strength and durability, HPC can reduce the number of girders, increase bridge spans, decrease bridge depth, and improve bridge durability (B. Russell, 1994). The durability and permeability of normal strength concrete has been thoroughly studied over the past century, but the same properties of HPC have not been studied to the same extent. To increase the durability of normal strength concrete, an air-entraining agent is added to the concrete mixture. While the entrained air increases durability, it also decreases concrete strength. For this reason, air entrainment is not commonly used in HPC (Cohen et al, 1992). Many researchers are questioning the need for air entrainment in HPC (Cohen et al, 1992; Hooten et al, 1993; Lessard et al, 1995; Li et al, 1994; Marchand et al, 1995; Mokhtarzadeh et al, 1995; Pigeon et al, 1991). Many of these researchers have concluded that the low permeability of HPC due to its decreased water to cementitious ratio (w/cm) improves its durability (Pigeon et al, 1991; Cohen et al, 1992; Li et al, 1994; Marchand et al, 1995; and Fagerlund, 1994). Accordingly, several researches have developed durable non-air-entrained HPC mixtures (Cohen et al, 1992; Mokhtarzadeh et al, 1995; Fagerlund, 1994; and Hilsdorf, 1994). Despite these indicators, most researchers still recommend the use of air entrainment for durability (Pigeon et al, 1991; Cohen et al, 1992; Marchand et al, 1995; Fagerlund, 1994; Zia et al, 1993; and Aitcin et al, 1993). The objective of this research program is to examine the need of air entrainment in HPC made with materials commonly available.

The use of HPC also has indirect benefits on the environment. The use of HPC can increase the life of concrete structures, reduce maintenance costs, and for bridges reduce the required number of girders. These examples either involve extending the life of a concrete structure or reducing the quantity and sizes of necessary members, thus promoting resource conservation by furthering the supply of cement. Another environmental benefit of HPC is the incorporation of supplementary materials. Most HPC mixtures incorporate supplementary cementitious materials such as fly ash and blast furnace slag, which are waste products of other manufacturing processes. Fly ash and blast furnace may replace a large percentage of the cement in a concrete mixture thereby reducing the energy required to produce a concrete structure. Prior research at OU examined concrete mixtures where 40 percent of the cement was replaced with fly ash and blast furnace slag (Hale et al, 1999).

Manufacturing cement is very energy intensive. Therefore, reducing the amount of cement in a concrete mixture or extending the life a concrete structure through the use of HPC can have a very beneficial effect on the environment. After aluminum and steel, the "production of portland cement is the third most energy intensive material production process" (Malhotra, 1999). In 1992, cement production accounted for 0.60 percent of the total U.S. energy use and accounted for almost two-thirds of the total energy use in some third world countries (Wilson, 1993). In addition to being very energy intensive, the production of cement also contributes significantly to the quantity of greenhouse gases. For every ton of cement manufactured, one ton of CO_2 is released into the atmosphere. Cement production accounts for "approximately seven percent of the total world CO_2 production" (Malhotra, 1999). Therefore, not only does the use of HPC have a direct beneficial impact on concrete structures, it also has a very important indirect effect on the environment.

1.2 OBJECTIVES

One of the primary goals in the research is to determine whether air entrainment is necessary in HPC. HPC is concrete that has higher strength and lower permeability when compared to normal strength concrete (NSC). The superior qualities are achieved by the low water to cementitious material ratio (w/cm) that is common in HPC. HPC commonly has w/cm equal to or less than 0.45 while NSC commonly has w/cm of 0.45 or greater. Many researchers suggest that AEA might not be necessary in HPC because of its low permeability, and because of its inherent unsaturated condition (Pigeon et al, 1991; Cohen et al, 1992; Li et al, 1994; Marchand et al, 1995; and Fagerlund, 1994).

In addition to investigating the freeze-thaw durability of HPC, the research program will also examine the permeability of HPC by using the Rapid Chloride Ion Penetrability test (RCIP). This portion of the research will examine permeability of mixtures with and without air entrainment and also mixtures with and without pozzolans. Since the RCIP test is a relatively new procedure for measuring permeability, the research program will also provide additional data to ODOT on the effectiveness of the test. Another objective of the research is to examine the prestress losses in concrete with and without air entrainment. This portion of the study will also include investigating the allowable release stress of prestressed concrete girders. The complete list of tasks is shown below.

Research Objectives

1. Develop HPC without AEA.

2. Develop HPC with AEA.
3. Investigate the freeze-thaw durability of HPC to determine the necessity of air entrainment in HPC.
4. Investigate the RCIP of HPC with and without AEA.
5. Determine prestress losses of HPC with and without AEA.
6. Investigate the allowable compressive stresses at release to determine whether current allowable stress requirements can be relaxed.

1.3 SCOPE

1.3.1 Developing High Performance Concrete

This portion of the research program developed HPC without air entrainment. Several HPC mixtures were developed, batched, and tested. With one of the mixtures developed, two prestressed/precast concrete girders were produced. Strength and durability were not the only concerns; the concrete mixtures required sufficient workability and placeability to be used at a local prestressed concrete facility. The mixtures also had to contain the materials and admixtures that were currently in use at the facility. The results of the study showed that a change in the cement source affected the performance of the concrete mixtures, but HPC was still developed using materials native to Oklahoma.

1.3.2 Developing High Performance Concrete with Entrained Air

This portion of the research program developed HPC with entrained air. Like the earlier research, two prestressed/precast concrete girders were cast with a mixture developed in the research. A goal of the research program was to produce HPC with entrained air and also to examine the effects of entrained air on the properties of the fresh and hardened concrete, specifically the compressive strength. The research results showed that air entrained HPC is possible using materials native to Oklahoma. However, the research results also showed that trial batching is necessary to determine the appropriate dosage of AEA at w/cm less 0.34.

1.3.3 Freeze-Thaw Durability of High Performance Concrete

This section of the experimental program was designed to examine two criteria focused on two objectives. The first objective was to determine if air entrainment is necessary in HPC. The second objective was to determine the maximum w/cm where air entrainment is not needed. Concrete mixtures with varying w/cm were subjected to ASTM C 666 (Procedure A). The variables for the mixtures were total air content and w/cm. The targeted total air contents for the mixtures were 2 percent (entrapped air, no AEA), 4 percent, and 6 percent. The freeze-thaw tests continued until the specimen deteriorated, or the specimen reached 300 freeze thaw cycles. The results from the study show that air entrainment is not required below a specific maximum w/cm for the materials employed.

1.3.4 Permeability of High Performance Concrete

This section of the research program investigated the permeability of HPC with and without entrained air. This section of the research program had two tasks. The first task was to examine if there were any differences in the permeability of concrete with and without entrained air. The second task was to provide further information on the validity of the Rapid Chloride Ion Penetrability Test (ASTM C 1202). Concrete mixtures with varying w/cm were subjected to ASTM C 1202. The variables for the mixtures were total air content and w/cm. The targeted total air contents for the mixtures were 2 percent (no air entraining agent), 4 percent, and 6 percent. The results from the research show that entrained air has no noticeable effect on the RCIP of the specimens. The results also further support the view that the RCIP test may not be an adequate test for measuring the permeability of concrete mixtures containing mineral admixtures.

1.3.5 Prestress Losses and Allowable Compressive Stresses at Release

This portion of the research examined the prestress losses of non-air entrained and air entrained bridge girders. The main task was focused on determining if the addition of entrained air had any noticeable effects on prestress losses. The measured losses were then compared with the losses calculated using two prevalent methods used for estimating prestress losses. Another objective of the research was to provide additional data with regard to increasing the allowable compressive stress at release. The results of the research show that the addition of entrained air had no noticeable effect on the prestress losses and that increasing the allowable compressive stresses at release may not be beneficial.

1.4 SUMMARY

The dissertation is divided into seven chapters. The first chapter includes an introduction to the six research objectives. The next five chapters are focused on each of the research objectives with the last two research objectives combined into one chapter. Each chapter contains its own literature review, experimental procedures, presentation and discussion of results, and conclusions. The final chapter contains a summary of all the conclusions and recommendations for further research.

CHAPTER 2

DEVELOPING HPC WITHOUT ENTRAINED AIR

2.1 INTRODUCTION

High Performance Concrete (HPC) does not differ significantly from normal strength concrete (NSC). Both types of concrete are composed from the same basic materials. The differences lie mainly in the proportions and the quantities of the materials. These differences allow HPC to have higher strengths and better durability than NSC. This chapter provides a brief history HPC and its uses, and describes the development of HPC incorporating materials that are native to and commonly used in Oklahoma. The quantity of cement, the water to cementitious material ratio (w/cm), and the type and dosage of admixtures are the variables that were examined. This chapter includes discussion of the literature review, laboratory experiments, and analysis of results.

2.2 LITERATURE REVIEW

2.2.1 Definitions

The definition of HPC has changed throughout the years. In 1979 the American Concrete Institute (ACI) Committee 363 on High Strength Concrete defined HPC as any concrete having a compressive strength of over 6000 psi (Derucher et al, 1994). In 1997, A.M. Neville in his book *Properties of Concrete* defined HPC as any concrete with a compressive strength of over 12,000 psi. The Strategic Highway Research Program (SHRP) defined HPC by three requirements. The requirements were a maximum w/cm of 0.35, a minimum durability factor of 80, and a minimum compressive strength of either 3000-psi within four hours of placement, 5000 psi at one day, or 10,000 psi at 28 days (H. Russell, 1999). In 1999, ACI redefined HPC as "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." As one can see, the definition of HPC has changed over the years, and it will continue to change as our knowledge of concrete increases.

2.2.2 Background

High Performance Concrete is not necessarily different from normal strength concrete. Both HPC and NSC contain the same basic constituent materials. Those materials are cement, fine aggregate (sand), coarse aggregate (rock), and water. The differences are in the quantities of those materials. HPC may contain as much as or more than 900 lb/yd^3 of cement, whereas NSC may contain approximately 500 lb/yd^3 of cement. Also, HPC may have a w/cm as low as 0.20 (Neville, 1997) where NSC may have a w/cm of 0.60. However, it should be noted that HPC can contain as little as 600 lb/yd^3 of cement.

Another difference between the two types of concretes is the usual addition of chemical and mineral admixtures in HPC that further enhance the fresh and hardened properties of concrete. Chemical admixtures are soluble chemicals that are added to concrete to improve its properties (Mindess et al, 1981). Chemical admixtures can reduce the water demand, increase or decrease the set time, entrain air, waterproof, and inhibit corrosion (Kosmatka et al, 1988). The advent of the superplasticizer or High Range Water Reducers (HRWR) in 1964 had a dramatic impact on the future of HPC (Derucher 1994). HRWR can reduce the amount of water in a given mixture by as much as 35 percent (Neville, 1997). With HRWR a mixture with a w/cm of about 0.25 can have the same slump as a mixture with a w/cm of about 0.50. The reduction in water content increases the strength and also increases the density of the concrete, which results in concrete with lower permeability.

Mineral admixtures are solids that are added to the concrete to improve its fresh and hardened properties. There are basically two types of mineral admixtures, cementitious and pozzolanic. Cementitious mineral admixtures are solids that have hydraulic cementing properties (along with water the materials will set and harden). Cementitious mineral admixtures include but are not limited to blast furnace slag, natural cements (clayey limestone), and hydraulic lime (Kosmatka et al, 1988). The cementitious mineral admixtures undergo hydration and contribute to concrete strength (Neville, 1997).

Pozzolanic admixtures include silica fume, fly ash, volcanic ash, metakaolin, and rice husk ash. Pozzolanic mineral admixtures (or pozzolans) are composed of natural or artificial materials that contain reactive silica. Pozzolans improve the characteristics of concrete by reacting with products formed during the hydration of cement. Equations 2.1 and 2.2 show the hydration of PC.

$$2C_3S + 6H_2O \rightarrow C_3S_2H_3 + 3CH \qquad (2.1)$$

$$2C_2S + 4H_2O \rightarrow C_3S_2H_3 + CH \tag{2.2}$$

$$CH + S + H_2O \rightarrow C_3S_2H_3 \tag{2.3}$$

The calcium silicate hydrate ($C_3S_2H_3$), which is a dense crystalline structure, is the primary contributor to the strength of concrete. The calcium hydroxide (CH) that is formed is less dense than the $C_3S_2H_3$, and therefore it is not a major contributor to strength. However, pozzolans contain amorphous silica (S), which reacts (Equation 2.3) with CH to form additional $C_3S_2H_3$, increasing the density of the cement paste matrix thereby improving the strength and permeability of the concrete.

2.2.3 History of HPC

The compressive strength and other properties of concrete have improved throughout the 20th Century. In 1903 Van Ornum conducted fatigue tests on concrete. The strength of the concrete specimens tested was between 1200 and 1580 psi. In 1927 McMillan et al published the compressive strengths of several types of mixtures with varying quantities of water. The compressive strengths ranged from a low of approximately 2000 psi to a high of approximately 5300 psi. Then in the 1930s Probst tested concrete with a compressive strength slightly greater than 2000 psi. In 1946 Le Camus tested concrete specimens with compressive strengths of 4600 psi (Bennett et al, 1967). Then Bennett et al tested concrete with a compressive strength of 8700 psi in 1967.

By the early 1960's, HPC was beginning to be used in building projects in the U.S. In 1962, the Outer Drive East Building in Chicago was built using concrete with a compressive strength of 6000 psi. Then in 1976 another Chicago building, the River Plaza Building, contained concrete with compressive strengths of 11,000 psi. One of the latest Chicago buildings to use HPC is the 225 W. Wacker Drive Project. Eighteen of the columns that support the 31-story building were constructed with concrete with compressive strengths of 14,000 psi with one experimental column with a compressive strength in excess of 17,000 psi (Moreno, 1990).

2.2.4 Examples of HPC

As previously shown, HPC has been used in the construction of many buildings. The uses of HPC stretch far beyond the construction of skyscrapers. When the entrances to a McDonald's in Quebec needed repairing, HPC was used. HPC was needed to reduce the number of days that the entrances were closed. A compressive strength of 3000 psi was required to allow the entrances to be opened to traffic. If normal strength concrete was used, seven days might have been necessary for the concrete to reach a compressive strength of 3000 psi. Two of the three entrances to the restaurant were to be replaced simultaneously. The entrances were scheduled to be opened to traffic two days later. Due to the high early compressive strength of the concrete, the entrances were reopened 24 hours later. A concrete mixture with a w/cm of 0.30 was used for both entrances. The mixtures attained a compressive strength of over 3000 psi by 24 hours of age (Lessard et al, 1994).

Most structures that utilize HPC do so because of the need for high compressive strengths, which cannot be attained through the use of NSC. However, the increase in modulus of elasticity and decreased permeability also make HPC appealing to designers and engineers. The Interfirst Plaza in Dallas and Two Union Square in Seattle used HPC not because of compressive strength requirements but because of the high modulus of elasticity requirement. The Two Union Square required concrete with a compressive strength of 14,000 psi to carry the loads, but to achieve a modulus of elasticity of 6000 ksi, a concrete mixture with a compressive strength of 19,000 psi was used. The decreased permeability of HPC led the designers of the Lacey V. Murrow bridge in Seattle to choose HPC over NSC (H. Russell, 1997).

Another major use of HPC is in the manufacture of prestressed concrete bridge girders. The use of HPC in bridges can increase the girder spacing and bridge span, produce shallower girders, and increase bridge durability (B. Russell, 1994). Many states have begun using HPC in their bridge girders. HPC bridges have been constructed in Nebraska, New Hampshire, Texas, Virginia, and Washington (H. Russell, 1997).

2.3 EXPERIMENTAL PROCEDURES

2.3.1. Scope

This portion of the research program sought to develop HPC without air entrainment. Several HPC mixtures were developed, batched, and tested. With one of the mixtures developed, two prestressed/precast concrete girders were produced. Strength and durability were not the only concerns; the concrete mixtures required sufficient workability and placeability to be used at a local prestressed concrete facility. The mixtures also had to contain the materials and admixtures that were currently in use at the facility.

The quantity of cement, the w/cm, and the type and dosage of admixtures were varied in the research. Earlier research (OCAST research) at OU had developed HPC without air entrainment (Bush et al, 1998), but a change in cement source required that additional testing be done to determine the effects of the change in cement when Holnam ceased production of their Type III Cement during the summer of 1998. This production change resulted in the need for a batching program to identify another suitable cement source to determine if the two cements produced concrete with similar properties. In addition to using a different cement source, earlier research had also used a different HRWR, Daracem 19. During this same time period, many local prestressed concrete facilities were changing from Daracem 19 to a different HRWR. One of the different HRWR is ADVA Flow, which was used in this research. The new HRWR required lower dosages, which reduced retarding effects.

The OCAST research had produced concrete mixtures with one-day compressive strengths of 10,000 psi and 28 day compressive strengths of 15,000 psi using Holnam Type III Cement. A goal of the research was to produce concrete mixtures with similar results. The results of the study showed that changing the cement source negatively affected the performance of the concrete mixtures, but HPC was still developed using materials native to Oklahoma.

The research program was divided into six sections with the following goals:

1) Determine if the change in cement had an affect on the properties of the concrete mixtures that had been developed through previous research.
2) Determine the effect of the change in HRWR.
3) Determine the optimum cement content and w/cm that produces concrete with most desirable qualities (strength and workability).
4) Determine the optimum dosage of HRWR.
5) Examine the influence of a WR.
6) Examine the influence of an accelerator.

2.3.2 Materials

The fine and coarse aggregates were constant for all mixtures. A fine aggregate from Dover, Oklahoma was used. The coarse aggregate was 3/8-in., crushed limestone from Davis, Oklahoma. The gradations for the coarse and fine aggregates are located in Appendix A. The material properties of the fine and coarse aggregates are shown in Table 2.1. Ash Grove Type III Cement from Chanute, KS was used in all but five mixtures. In these five mixtures, Holnam Type III Cement from Midlothian, TX was used. The chemical compositions and blaine fineness of the two cements are shown in Table 2.2. When needed, a water reducer, WRDA with Hycol, and a high range water reducer, ADVA Flow, were added to the mixtures to provide adequate workability. To increase early age strength, an accelerator, DCI Corrosion Inhibitor, was also added to some mixtures.

Table 2.1 Aggregate Properties

Property	Fine Aggregate	Coarse Aggregate
Absorption (SSD)	0.68 %	0.86 %
Specific Gravity	2.63	2.68
Dry Rodded Unit Weight (lb/ft^3)	-	101.2

Table 2.2 Cement Properties

	Ash Grove	Holnam
Chemical Compositions (%)		
SiO_2	20.56	19.70
Al_2O_3	4.74	5.80
Fe_2O_3	3.06	2.76
CaO	64.10	61.40
MgO	2.49	0.90
SO_3	3.14	4.15
Compound Compositions (%)		
C_3S	63.00	57.10
C_2S	12.00	13.80
C_3A	6.00	10.50
C_4AF	9.00	-
	Blaine Air Fineness	
Blaine Fineness (cm^2/g)	4740	5240

2.3.3 Variables

2.3.3.1 Water to Cementitious Material Ratio. One of the first variables to be examined in the project was the w/cm. The w/cm for the project ranged from 0.24 to 0.30. Mixtures were cast at the following w/cm; 0.24, 0.25, 0.26, 0.27, 0.28, 0.29, and 0.30. Since the goal of the study was to develop HPC, the mixtures were cast at w/cm's between 0.24 and 0.30

2.3.3.2 Cement Quantity. Another variable examined was the quantity of cement in each mixture. The quantity of cement was either 800, 900, or 1000 lb/yd^3. Ash Grove Type III Cement was used in all but five mixtures. The five mixtures containing Holnam Type III Cement were used to compare the differences, if any, between the two sources of cement.

2.3.3.3 Admixtures and Dosage. Since the HRWR might react differently with the two types of cements (Ash Grove and Holnam), the dosages of the HRWR were varied to determine the appropriate quantity. Too much HRWR causes the mixture's set to be delayed, and too little HRWR causes a mixture with poor workability. The HRWR dosage ranged from 5 to 22.5 fluid ounces per 100 lb. of cement (fl oz/cwt). Another variable was the quantity of the water reducer (WR). The dosage rate of the WR was zero, three, or six fl oz/cwt. An accelerator was added to three mixtures to increase the early age strength of the concrete. The dosage of the accelerator was nine gal/yd^3. Two mixtures contained Type C Fly Ash.

2.3.4 Mixtures

The mixtures proportions were developed to investigate and accomplish the six goals of the research program previously stated. The mixtures proportions examined the effects of the cement source and HRWR source along with the quantity of cement, w/cm, HRWR dosage, WR dosage, and accelerator dosage in the mixtures.

2.3.4.1 Cement Source. In the first section of the research, concrete mixtures consisting of Ash Grove Type III Cement were cast using the same mixture designs that had performed well with Holnam Type III Cement. The HRWR and WR used were Daracem 19 and WRDA with Hycol, respectively. The mixtures that were repeated using Ash Grove Cement are shown in Table 2.3. Mixtures 3 and 10 contained only portland cement. In Mixtures 30, 31, and 45, 15 percent of the cement was replaced with fly ash.

Table 2.3. Mixture Proportions from Prior Research Batched w/ Ash Grove Cement.

Materials	Mixture Designations				
	3	10	30	31	45
Cement (lb/yd^3)	1096	927	809	742	674
Fly Ash (lb/yd^3)	0	0	202	186	169
Coarse Aggregate (lb/yd^3)	1790	1790	1790	1790	2066
Fine Aggregate (lb/yd^3)	783	1102	996	1129	1081
Water (lb/yd^3)	319	247	253	232	228
w/cm	0.30	0.28	0.26	0.26	0.28
HRWR (fl oz/cwt)	22	28	20	17	15

2.3.4.2 High Range Water Reducer Source. In this section of the research, a different HRWR, ADVA Flow, was used with Mixture 3 to determine if the Ash Grove Type III Cement was in fact reacting differently with Daracem 19. For each batch, all variables were held constant except for the HRWR dosage. Mixture 3 was cast four times with HRWR dosages of 6, 8, 10, and 20 fl oz/cwt. The mixture proportions are shown in Table 2.4. The mixtures are designated by the amount of HRWR contained in the mixture. For example, Mixture 3-20 is Mixture 3 with a HRWR dosage rate of 20 fl oz/cwt.

Table 2.4. Mixture Proportions for Examining Effects of HRWR.

Materials	Mixture Designations			
	3-6	3-8	3-10	3-20
Cement (lb/yd^3)	1096	1096	1096	1096
Fly Ash (lb/yd^3)	0	0	0	0
Coarse Aggregate (lb/yd^3)	1790	1790	1790	1790
Fine Aggregate (lb/yd^3)	783	783	783	783
Water (lb/yd^3)	319	319	319	319
w/cm	0.30	0.30	0.30	0.30
HRWR (fl oz/cwt)	6	8	10	20

2.3.4.3 Cement Quantity and Water to Cementitious Material Ratio. The results from the first and second sections of the research showed that there were differences in the two HRWR. The next step was to determine the optimum cement content per cubic yard and w/cm. For all mixtures the coarse aggregate volume was held constant at 65 percent, and the HRWR (ADVA Flow) dosage was 22.5 fl. oz./cwt. The variables in the matrix were cement content and w/cm. The w/cm varied from 0.24 to 0.30, and the cement content varied from 800 to 1000 pounds per cubic yard. These mixtures are listed in Table 2.5 and Table 2.6. The mixtures are designated by the quantity of cement in each mixture and the w/cm of the mixture. For example, Mixture 9-24 had a w/cm of 0.24 and contained 900 lb. of cement.

Table 2.5. Mixture Proportions for Optimizing w/cm and Cement Content.

Materials	Mixture Designations					
	9-24	10-24	8-26	9-26	10-26	8-28
Cement (lb/yd^3)	900	1000	800	900	1000	800
Coarse Aggregate (lb/yd^3)	1790	1790	1790	1790	1790	1790
Fine Aggregate (lb/yd^3)	1264	1118	1368	1217	1065	1326
Water (lb/yd^3)	216	240	208	234	260	224
w/cm	0.24	0.26	0.26	0.26	0.26	0.28
HRWR (fl oz/cwt)	22.5	22.5	22.5	22.5	22.5	22.5

Table 2.6. Mixture Proportions for Optimizing w/cm and Cement Content.

Materials	Mixture Designations				
	9-28	10-28	8-30	9-30	10-30
Cement (lb/yd^3)	900	1000	800	900	1000
Coarse Aggregate (lb/yd^3)	1790	1790	1790	1790	1790
Fine Aggregate (lb/yd^3)	1169	1013	1284	1122	960
Water (lb/yd^3)	252	280	240	270	300
w/cm	0.28	0.28	0.30	0.30	0.30
HRWR (fl oz/cwt)	22.5	22.5	22.5	22.5	22.5

2.3.4.4 High Range Water Reducer Dosage. Since the majority of mixtures cast had sufficient workability, the next step was to determine if reducing the HRWR dosage would improve one-day compressive strength without sacrificing workability. The mixture designs are shown in Tables 2.7, 2.8, and 2.9. For most mixtures, the coarse aggregate volume was again held constant at 65 percent, and the cement content was 900 lb/yd^3. One mixture contained 800 lb/yd^3 of cement while another contained 1000 lb/yd^3. The variables were w/cm and HRWR dosage. The w/cm was 0.26, 0.28, and 0.30, and the HRWR dosage ranged from 5 to 18 fl. oz/cwt. These mixtures are also designated by the HRWR in addition to the cement content and w/cm. The last number in the mixture designation is the HRWR dosage.

Table 2.7. Mixture Proportions for Optimizing HRWR Dosage.

Materials	Mixture Designations				
	9-26-18	9-28-18	9-30-18	9-26-15	9-28-15
Cement (lb/yd^3)	900	900	900	900	900
Coarse Aggregate (lb/yd^3)	1790	1790	1790	1790	1790
Fine Aggregate (lb/yd^3)	1217	1169	1122	1217	1169
Water (lb/yd^3)	234	252	270	234	252
w/cm	0.26	0.28	0.30	0.26	0.28
HRWR (fl oz/cwt)	18	18	18	15	15

Table 2.8. Mixture Proportions for Optimizing HRWR Dosage.

Materials	Mixture Designations				
	9-30-15	9-26-10	9-28-10	9-26-5	9-28-5
Cement (lb/yd^3)	900	900	900	900	900
Coarse Aggregate (lb/yd^3)	1790	1790	1790	1790	1790
Fine Aggregate (lb/yd^3)	1122	1217	1169	1217	1169
Water (lb/yd^3)	270	234	252	234	252
w/cm	0.30	0.26	0.28	0.26	0.28
HRWR (fl oz/cwt)	15	10	10	5	5

Table 2.9. Mixture Proportions for Optimizing HRWR Dosage.

Materials	Mixture Designations		
	8-28-15	9-28-15	10-28-15
Cement (lb/yd^3)	800	900	1000
Coarse Aggregate (lb/yd^3)	1790	1790	1790
Fine Aggregate (lb/yd^3)	1326	1169	1013
Water (lb/yd^3)	224	252	280
w/cm	0.28	0.28	0.28
HRWR (fl oz/cwt)	15	15	15

2.3.4.5 High Range Water Reducer and Water Reducer. In an attempt to decrease HRWR dosage without compromising workability, a water reducer (WR) was introduced into the mixtures. The WR used in all mixtures was WRDA with Hycol. The WR was chosen based on its minimal retarding effects. For these mixtures, the coarse aggregate was again held constant at 65 percent, and the cement content was 900 lb/yd^3. The variables were HRWR dosage, WR dosage, and w/cm. The HRWR varied from 5 to 15 fl. oz/ cwt, and the WR was either 3 or 6 fl. oz/cwt. The w/cm for the mixtures was 0.26, 0.28, and 0.30. The mixtures are shown in

Tables 2.10, 2.11, and 2.12. The same system of identification is used except the last numbers represent to the HRWR and WR dosage. For example, Mixture 9-26-5/6 had a w/cm of 0.26, contained 900 lb/yd^3 of cement, and the dosages of HRWR and WR were 5 and 6 fl oz/cwt, respectively.

Table 2.10. Mixture Proportions for Optimizing HRWR and WR Dosage.

Materials	Mixture Designations				
	9-26-5/6	9-26-5/3	9-26-8/6	9-26-10/6	9-26-10/3
Cement (lb/yd^3)	900	900	900	900	900
Coarse Agg. (lb/yd^3)	1790	1790	1790	1790	1790
Fine Agg. (lb/yd^3)	1217	1217	1217	1217	1217
Water (lb/yd^3)	234	234	234	234	234
HRWR (fl oz/cwt)	5	5	8	10	10
WR (fl oz/cwt)	6	3	6	6	3
w/cm	0.26	0.26	0.26	0.26	0.26

Table 2.11. Mixture Proportions for Optimizing HRWR and WR Dosage.

Materials	Mixture Designations				
	9-26-10/3	9-26-15/3	9-28-5/3	9-28-10/6	9-28-10/3
Cement (lb/yd^3)	900	900	900	900	900
Coarse Agg. (lb/yd^3)	1790	1790	1790	1790	1790
Fine Agg. (lb/yd^3)	1217	1217	1169	1169	1169
Water (lb/yd^3)	234	234	252	252	252
HRWR (fl oz/cwt)	10	15	5	10	10
WR (fl oz/cwt)	3	3	3	6	3
w/cm	0.26	0.26	0.28	0.28	0.28

Table 2.12. Mixture Proportions for Optimizing HRWR and WR Dosage.

Materials	Mixture Designations			
	9-28-10/3	9-28-15/3	9-30-10/6	9-30-15/3
Cement (lb/yd^3)	900	900	900	900
Coarse Agg. (lb/yd^3)	1790	1790	1790	1790
Fine Agg. (lb/yd^3)	1169	1169	1122	1122
Water (lb/yd^3)	252	252	270	270
HRWR (fl oz/cwt)	10	15	10	15
WR (fl oz/cwt)	3	3	6	3
w/cm	0.28	0.28	0.30	0.30

2.3.4.6 Accelerator. To improve one-day compressive strength, an accelerator was introduced into several mixtures. For all mixtures the coarse aggregate was held constant at 65 percent, and the dosage rate for the accelerator was nine gallons per cubic yard. There were many variables in this portion of the research. The variables were w/cm, HRWR dosage, WR dosage, and cement content. The w/cm was 0.26, 0.28, or 0.30. The cement content was 800, 900, or 1000 lb/yd^3. The HRWR dosage was 10 or 15 fl oz/cwt. The WR dosage was 3 fl oz/cwt. The same system of identification is used as before except for the addition of a "D" to the designation to signify the addition of an accelerator. The mixture proportions are shown in Table 2.13 and 2.14.

Table 2.13. Mixture Proportions for Examining Accelerator Dosage.

Materials	Mixture Designations				
	8-28-15D	9-28-15D	10-28-15D	9-26-10D	9-28-10D
Cement (lb/yd^3)	800	900	1000	900	900
Coarse Agg. (lb/yd^3)	1790	1790	1790	1790	1790
Fine Agg. (lb/yd^3)	1326	1169	1013	1217	1169
Water (lb/yd^3)	224	252	280	234	252
HRWR (fl oz/cwt)	15	15	15	10	10
WR (fl oz/cwt)	0	0	0	0	0
Accelerator (gal/yd^3)	9	9	9	9	9
w/cm	0.28	0.28	0.28	0.26	0.28

Table 2.14. Mixture Proportions for Examining Accelerator Dosage.

Materials	Mixture Designations		
	9-30-10D	9-28-15/3D	10-26-10/6D
Cement (lb/yd^3)	900	900	1000
Coarse Agg. (lb/yd^3)	1790	1790	1790
Fine Agg. (lb/yd^3)	1122	1169	1065
Water (lb/yd^3)	270	252	260
HRWR (fl oz/cwt)	10	15	10
WR (fl oz/cwt)	0	3	6
Accelerator (gal/yd^3)	9	9	9
w/cm	0.30	0.28	0.26

2.3.5 Batching

Prior to the batching of each mixture, the moisture content of the aggregates was determined. This was accomplished by obtaining representative samples of the aggregates, which were stored in stockpiles outside of the laboratory. The samples were then weighed and oven dried to a constant weight (ASTM C 566). While obtaining the aggregate samples, sufficient quantities of the coarse and fine aggregates were separately placed into 5 gallon plastic buckets. Each bucket contained 50 lb. of coarse or fine aggregate. Lids were then placed on each bucket to prevent any loss of moisture from the time of sampling to the time of batching.

The batching was done in a rotating drum mixer with a 6 ft^3 capacity. All of the batching conformed to ASTM C 192 except for the mixing times. The mixtures were mixed until they achieved a uniform consistency. The order and procedure of the addition of the constituent materials remained the same for all mixtures. For all batches, all of the coarse aggregate and one-half of the mixing water were added to the mixer first, before other ingredients. Then the remaining materials were gradually introduced into the mixer. The WR was added to the mixing water whereas the HRWR was added once all the materials had been introduced into the mixer.

For the research program, the desired fresh concrete temperature was between 60 and 70°F. Since the majority of batching was conducted during the summer, crushed ice was added to the mixing water. For those mixtures cast in cooler months, warm tap water was used to increase the fresh concrete temperature.

2.3.6 Curing

All the concrete specimens were cured at 73°F and 50 percent relative humidity (RH) for the first 24 hours. The specimens were then removed from their molds and moist cured in lime-saturated water at 73°F until time of testing.

2.3.7 Tests

The mixtures were subjected to several tests. The fresh properties examined were concrete temperature and slump (ASTM C 143). The hardened property tested was compressive strength. The compressive strength (ASTM C 39) was tested at one and 28 days of age. At each age, three 4 x 8 in. cylinders were tested. The durability and permeability of the mixtures are examined in Chapters 4 and 5.

2.4 PRESENTATION OF RESULTS

2.4.1 Fresh Concrete Results

The fresh concrete results are shown below in Tables 2.15, 2.16, 2.17, 2.18, 2.19, and 2.20. The results are separated by the six different sections of the research program. The concrete temperature was between 60 and 70 F for most mixtures. The slumps for the mixtures ranged from zero to 12 inches. Some mixtures would not mix. Those mixtures that did not mix are designated with DNM.

Table 2.15. Fresh Concrete Properties from Examining Cement Source.

Mixtures	Concrete Temp. (F)	Slump (in.)
3	62	8.75
10	65	8.50
30	61	5.00
31	63	5.50
45	61	8.00

Table 2.16. Fresh Concrete Properties from Examining Effects of HRWR.

Mixtures	Concrete Temp. (F)	Slump (in.)
3-6	60	1.00
3-8	62	2.00
3-10	61	12.00
3-20	61	12.00

Table 2.17. Fresh Concrete Properties from Optimizing Cement Quantity.

Mixtures	Concrete Temp. (F)	Slump (in.)
9-24	60	7.50
10-24	63	6.00
8-26	68	2.50
9-26	63	7.00
10-26	62	8.25
8-28	58	7.50
9-28	59	8.25
10-28	58	9.50
8-30	71	7.50
9-30	72	9.00
10-30	70	10.0

Table 2.18. Fresh Concrete Properties from Optimizing HRWR Dosage.

Mixtures	Concrete Temp. (F)	Slump (in.)
9-26-18	60	8.25
9-28-18	60	10.0
9-30-18	62	11.0
9-26-15	63	6.50
9-28-15	61	7.75
9-30-15	60	9.00
9-26-10	65	0.50
9-28-10	68	2.00
9-26-5	-	DNM
9-28-5	61	7.75
8-28-15	77	6.25
9-28-15	77	8.25
10-28-15	77	7.25

Table 2.19. Fresh Concrete Properties from Optimizing HRWR and WR Dosage.

Mixtures	Concrete Temp. (F)	Slump (in.)
9-26-5/6	60	DNM
9-26-5/3	60	DNM
9-26-8/6	59	2.00
9-26-10/6	59	3.25
9-26-10/3	55	1.50
9-26-10/3	63	4.00
9-26-15/3	65	7.00
9-28-5/3	60	0.50
9-28-10/6	65	4.00
9-28-10/3	58	2.50
9-28-10/3	65	6.00
9-28-15/3	71	8.25
9-30-10/6	65	8.25
9-30-15/3	65	9.50

Table 2.20. Fresh Concrete Properties from Examining Accelerator Dosage.

Mixtures	Concrete Temp. (F)	Slump (in.)
8-28-15D	72	0.00
9-28-15D	71	4.25
10-28-15D	72	6.00
9-26-10D	-	DNM
9-28-10D	-	0.25
9-30-10D	73	0.25
9-28-15/3D	74	7.00
10-26-10/6D	-	DNM

2.4.2 Hardened Concrete Results.

The compressive strength at one and 28 days was the only hardened concrete property tested for this portion of the research. Each compressive strength result is the average of three 4 x 8 in. cylinders. The individual cylinder tests, standard deviations, and confidence intervals are shown in Appendix A. The durability and permeability are examined in later chapters. The test results are shown in Table, 2.21, 2.22, 2.23, 2.24, 2.25 and 2.26.

Table 2.21. Compressive Strength Results from Examining Cement Source.

Mixtures	Average One Day (psi)[1]	Average 28 Day (psi)[1]
3	5500	12,110
10	3690 (2 d)	6040
30	5550	12,630
31	4640	12,890
45	7670	12,120

1) Average compressive strength of three tests.
2) Individual tests and statistical data are shown in Appendix A.

Table 2.22. Compressive Strength Results from Examining Effects of HRWR.

Mixtures	Average One Day (psi)[1]	Average 28 Day (psi)[1]
3-6	6370	10,920
3-8	7240	10,920
3-10	6350	11,280
3-20	4220	11,800

1) Average compressive strength of three tests.
2) Individual tests and statistical data are shown in Appendix A.

Table 2.23. Compressive Strength Results from Optimizing Cement Quantity.

Mixtures	Average One Day (psi) [1]	Average 28 Day (psi) [1]
9-24	7190	11,805
10-24	8390	13,750
8-26	8210	12,450
9-26	7450	11,950
10-26	6840	12,050
8-28	7440	12,300
9-28	6940	11,410
10-28	5250	11,550
8-30	4750	10,480
9-30	3890	10,050
10-30	2780	9510

1) Average compressive strength of three tests.
2) Individual tests and statistical data are shown in Appendix A.

Table 2.24. Compressive Strength Results from Optimizing HRWR Dosage.

Mixtures	Average One Day (psi) [1]	Average 28 Day (psi) [1]
9-26-18	6660	11,540
9-28-18	6980	12,210
9-30-18	5460	12,840
9-26-15	7930	12,140
9-28-15	6180	12,540
9-30-15	5710	11,575
9-26-10	8990	11,930
9-28-10	8280	12,390
9-26-5	DNM	DNM
9-28-5	6990	PC*
8-28-15	7430	13,010
9-28-15	7595	13,540
10-28-15	6240	10,090

1) Average compressive strength of three tests.
2) Individual tests and statistical data are shown in Appendix A.
3) PC = Poor Cylinders, due to poor workability the remaining cylinders were not properly consolidated.
4) DNM = concrete mixtures that did not mix.

Table 2.25. Compressive Strength Results from Optimizing HRWR and WR Dosage.

Mixtures	Average One Day (psi) [1]	Average 28 Day (psi) [1]
9-26-5/6	DNM	DNM
9-26-5/3	DNM	DNM
9-26-8/6	8560	13,250
9-26-10/6	9000	14,120
9-26-10/3	8830	PC
9-26-10/3	9290	13,150
9-26-15/3	8520	13,370
9-28-5/3	7900	PC
9-28-10/6	9100	12,870
9-28-10/3	8460	12,570
9-28-10/3	7920	11,650
9-28-15/3	7420	13,500
9-30-10/6	7830	11,980
9-30-15/3	7850	12,150

1) Average compressive strength of three tests.
2) Individual tests and statistical data are shown in Appendix A.
3) PC = Poor Cylinders, due to poor workability the remaining cylinders were not properly consolidated.
4) DNM = concrete mixtures that did not mix.

Table 2.26. Compressive Strength Results from Examining Accelerator Dosage.

Mixtures	Average One Day (psi) [1]	Average 28 Day (psi) [1]
8-28-15D	7920	PC
9-28-15D	8960	13,240
10-28-15D	9100	13,870
9-26-10D	DNM	DNM
9-28-10D	8780	PC
9-30-10D	8400	PC
9-28-15/3D	8630	12,880
10-26-10/6D	DNM	DNM

1) Average compressive strength of three tests.
2) Individual tests and statistical data are shown in Appendix A.
3) PC = Poor Cylinders, due to poor workability the remaining cylinders were not properly consolidated.
4) DNM = concrete mixtures that did not mix.

2.5 DISCUSSION OF RESULTS

2.5.1 Fresh Concrete Results

2.5.1.1 Cement Source. The source of cement had little effect on the workability of the mixtures. The slumps of the mixtures are shown in Figure 2.1. This is expected because there was little difference between the two cements. Both cements had finenesses within 500 cm^2/g of each other. Higher cement fineness increases the amount of surface area, which would increase the water demand, but the difference in fineness was not that great.

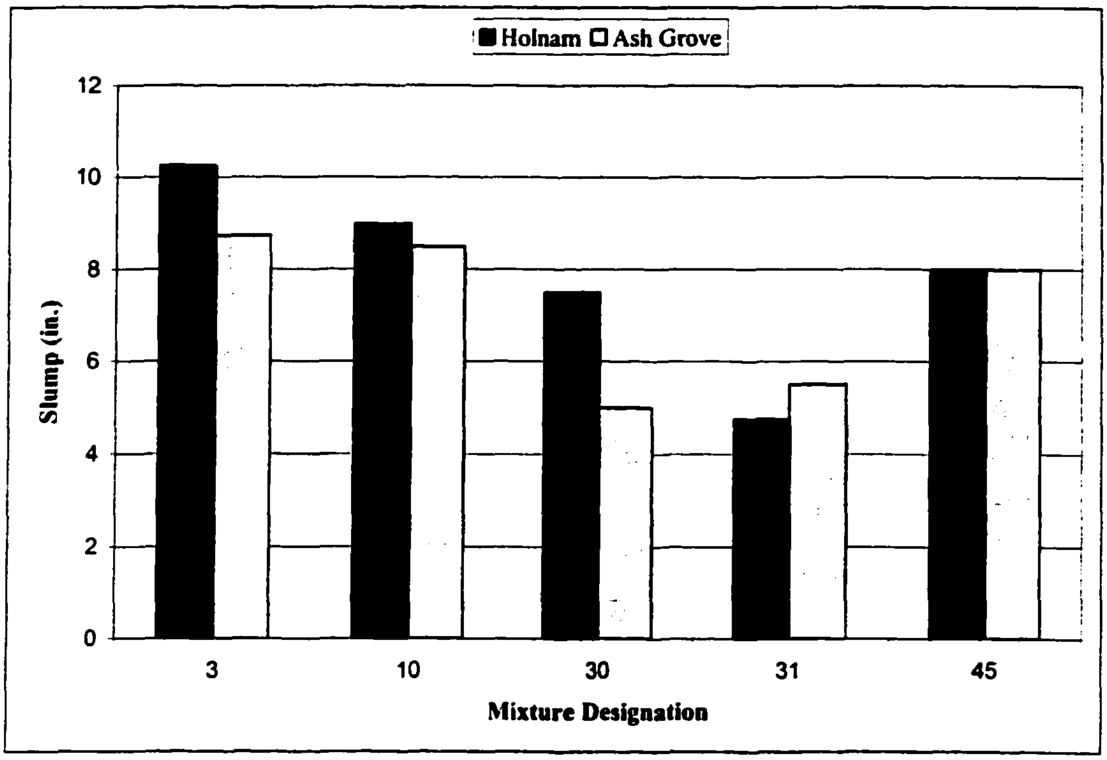

Figure 2.1 Slumps of Ash Grove and Holnam Mixtures.

2.5.1.2 High Range Water Reducer Source. For this portion of the research, Mixture 3 was batched five times with either Daracem 19 or ADVA Flow. All mixtures contained Ash Grove Type III Cement. The results of the slump tests are shown in Figure 2.2. Once again, the last number in the mixtures designations is the dosage rate of HRWR in fl oz/cwt. From the figure, it is evident that the dosage rates for the two HRWR are different. At equal dosage rates, mixtures containing ADVA Flow had greater slumps than mixtures cast with Daracem 19. Furthermore, to produce concrete with similar slumps the dosage rate for mixtures containing ADVA Flow was less than half of that for mixtures cast with Daracem 19.

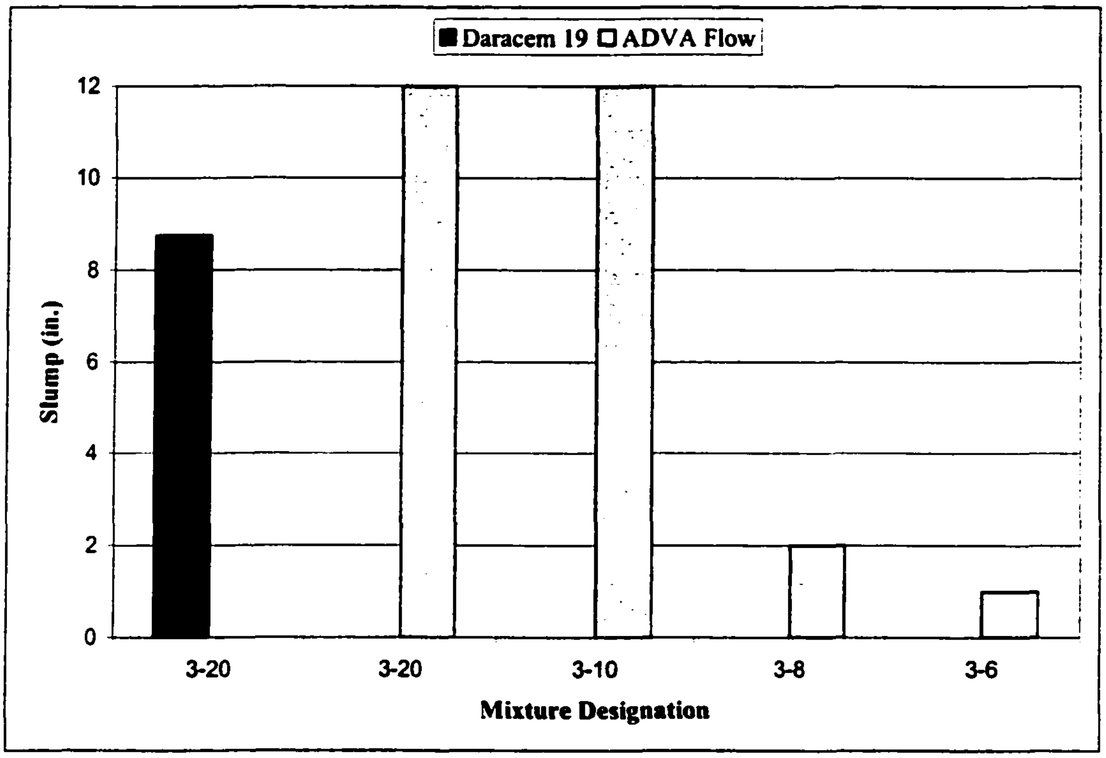

Figure 2.2 Comparisons of ADVA Flow and Daracem 19.

2.5.1.3 Cement Quantity and Water to Cementitious Material Ratio. The goal of the next phase in the research program was to determine the optimum cement content and w/cm. The results are shown in Figure 2.3. All mixtures were cast with Ash Grove Type III Cement and contained ADVA Flow at a dosage rate of 22.5 fl oz/cwt. The cement content for the mixtures was 800, 900, or 1000 lb/yd^3. The w/cm for the mixtures was 0.24, 0.26, 0.28, or 0.30. For all but one mixture, increases in w/cm resulted in increases in slumps, which was expected. Also, for all mixtures except one, increases in cement content resulted in greater slumps for mixtures of the same w/cm, which was also expected. Since adequate workability was a requirement, mixtures with a w/cm greater than 0.26 and cement content equal to and greater than 900 lb/yd^3 were further examined in the research program. Also, the results from the slump tests showed that reducing the dosage of ADVA Flow might be possible due to the fact that most mixtures had slumps greater than seven inches. From the slump data, a preliminary design aid (Table 2.27) was developed to show the relationship of w/cm, cementitious material content, and slump. When combined with additional research results, the table could be used as a design aid to assist engineers in developing concrete mixtures.

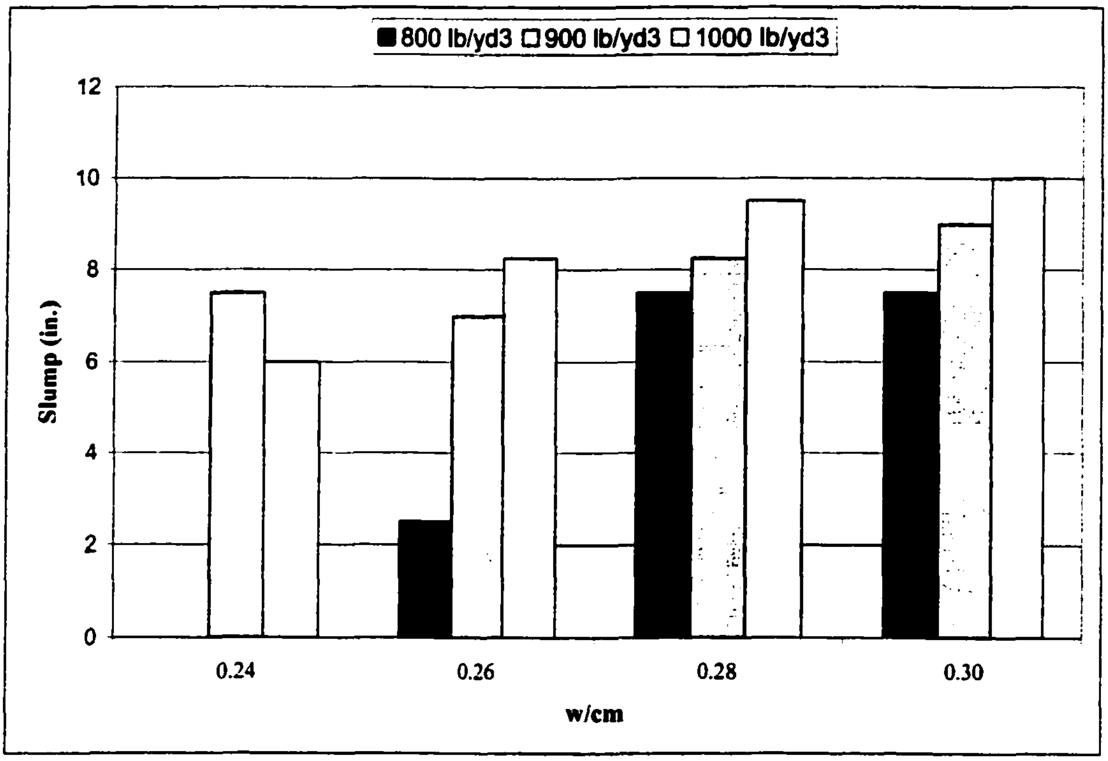

Figure 2.3. Slumps for Determining Cement Content and w/cm.

Table 2.27. Design Aid for Determining Slumps (in.) Based on Mixture Properties.

w/cm	Cementitious Material Content		
	800	900	1000
0.26	2.50	7.00	8.25
0.28	7.50	8.25	9.50
0.30	7.50	9.00	10.00

2.5.1.4 High Range Water Reducer Dosage. The goal of this phase of the research was to determine the optimum HRWR dosage. The HRWR dosage was varied from 5 to 18 fl oz/cwt. All mixtures contained 900 lb/yd^3 of cement, and the HRWR used was ADVA Flow. The results of the study are shown in Figure 2.4. As expected, the HRWR dosage did affect the slumps of the mixtures. Increases in the quantity of HRWR resulted in increases in slump and workability. Also, increases in w/cm resulted in increases in slumps, which was also expected. For mixtures containing 900 lb/ft^3 of cement, the results shown in Figure 2.4 indicate that a minimum HRWR dosage of 15 fl oz/cwt is necessary to produce concrete with adequate workability.

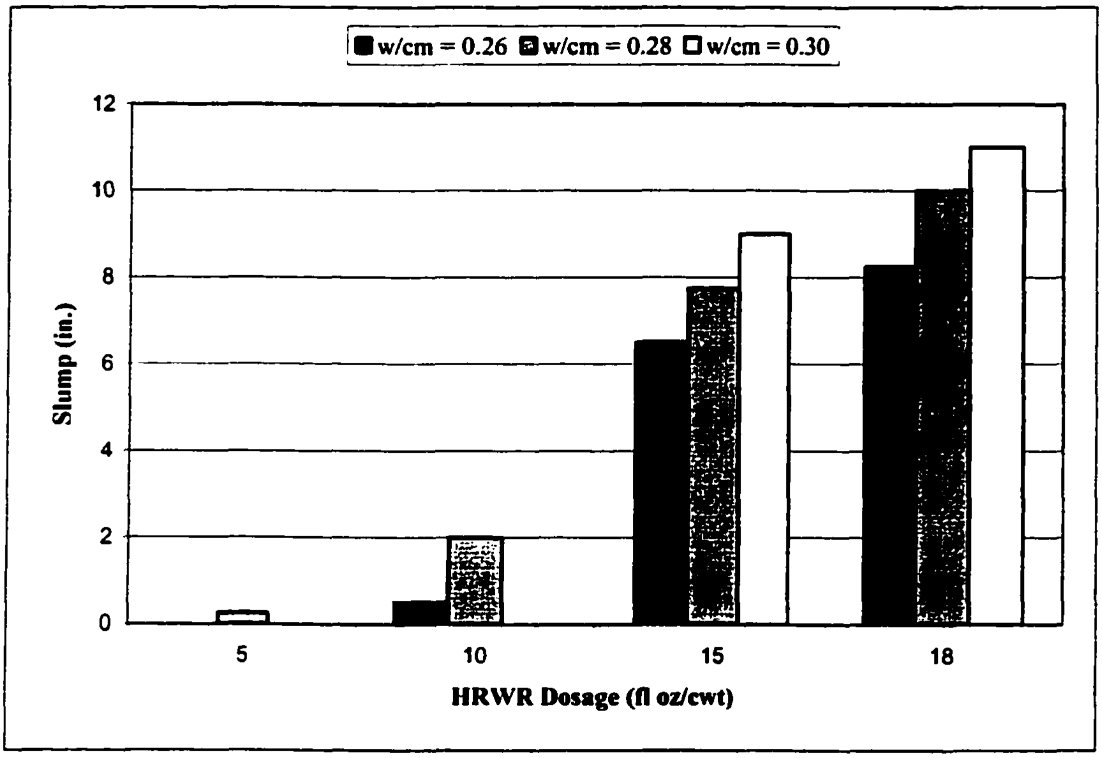

Figure 2.4. Slumps for Determining HRWR Dosage.

2.5.1.5 High Range Water Reducer and Water Reducer. The next phase of the research examined whether the addition of a WR could reduce the HRWR dosage. All mixtures contained 900 lb/yd^3 of cement. The WR and HRWR used was WRDA with Hycol and ADVA Flow, respectively. The results are shown in Figure 2.5. To produce concrete with sufficient workability, a HRWR dosage of 15 fl oz/cwt was still necessary for concrete mixtures with a w/cm of 0.26. Lower dosages of HRWR were possible at higher w/cm, but the increase in w/cm decreased compressive strength which will be shown later in the chapter.

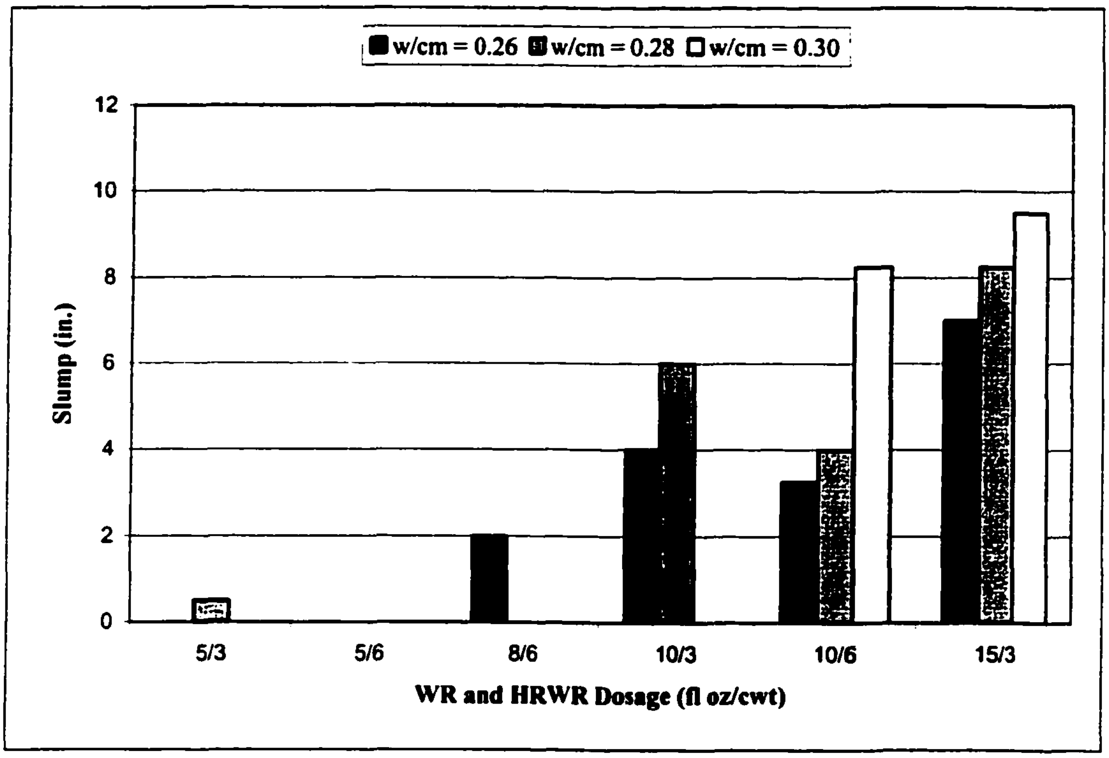

Figure 2.5. Slumps for Determining WR and HRWR Dosage.

2.5.1.6 Accelerator. The final phase of this section investigated the addition of an accelerator to increase the compressive strength at one day of age. Accelerators are added to concrete to reduce the set times and increase early age strength. The results from the study were shown in Table 2.20. The addition of the accelerator reduced the workability of most mixtures. Most importantly, the accelerator greatly increased the slump loss. Many mixtures began to set before all the specimens could be cast. This is an important factor in the research because with the mixtures developed, beams were cast. The mixtures must be fluid for a sufficient period of time to allow the beams to be cast.

2.5.2 Hardened Concrete Results

2.5.2.1 Cement Source. Concrete mixtures containing Ash Grove Type III Cement were cast using the same mix designs that performed well with Holnam Type III Cement. Other than cement source, the remaining quantities and types of materials remained the same for each pair of mixtures. The HRWR used for all mixtures was Daracem 19. Comparisons of the compressive strengths of the two cements are shown in Table 2.28.

With the exception of mixture 45, concrete mixtures made with Ash Grove cement did not perform as well as concrete mixtures made with Holnam Type III Cement. Not only did the Ash Grove mixtures have significantly less strength (3000 to 5000 psi at one day), the cement appeared to react differently with the HRWR, Daracem 19. For instance, the set time was delayed for Mixture 10. Mixture 10 had a one-day compressive strength of 10,050 psi when made with Holnam III, whereas the same mixture made with Ash Grove III had not reached final set at 24 hours and had a two-day compressive strength of only 3690 psi. This evidence along with the decrease in one-day compressive strength suggested that Daracem 19 might have been reacting differently with Ash Grove III.

Table 2.28. Compressive Strengths of Ash Grove and Holnam Mixtures.

Mixture	1 Day (psi)		28 Day (psi)	
	Holnam	Ash Grove	Holnam	Ash Grove
3	9690	5500	14,240	12,110
10	10,050	3690 (2d)	14,600	6040
30	7930	5550	14,670	12,630
31	7990	4640	14,210	12,890
45	7350	7670	13,750	12,120

2.5.2.2 High Range Water Reducer Source. A different HRWR, ADVA Flow, was used with Mixtures 3 and 10 to determine if Ash Grove III was in fact reacting differently with Daracem 19. The results of the one-day compressive strength tests are shown in Figure 2.6. Also shown in Figure 2.6 are the mixtures cast with Holnam III and Daracem 19 and then subsequent mixtures cast with Ash Grove III and various dosages of either Daracem 19 or ADVA Flow. The dosages of the HRWR are listed directly below the type of HRWR.

The use of ADVA Flow produced concrete with greater strengths and greater slumps than like mixtures cast with Daracem 19. However, the best performing Ash Grove mixtures had strengths over 2000 psi less than similar Holnam mixtures at one day of age. Mixture 10, which had not reached final set at 24 hours when made with Daracem 19, achieved a one day compressive strength of 7650 psi when cast with ADVA Flow at a dosage rate of 12 fl oz/cwt. Even with the increase in strength, Mixture 10 when made with ADVA Flow was still significantly less (2500 psi) than the same Holnam III mixture at one day of age. The results from these tests suggest that Ash Grove Cement may have been reacting differently with Daracem 19, but it appears that Holnam Cement was of a higher quality than the other cement.

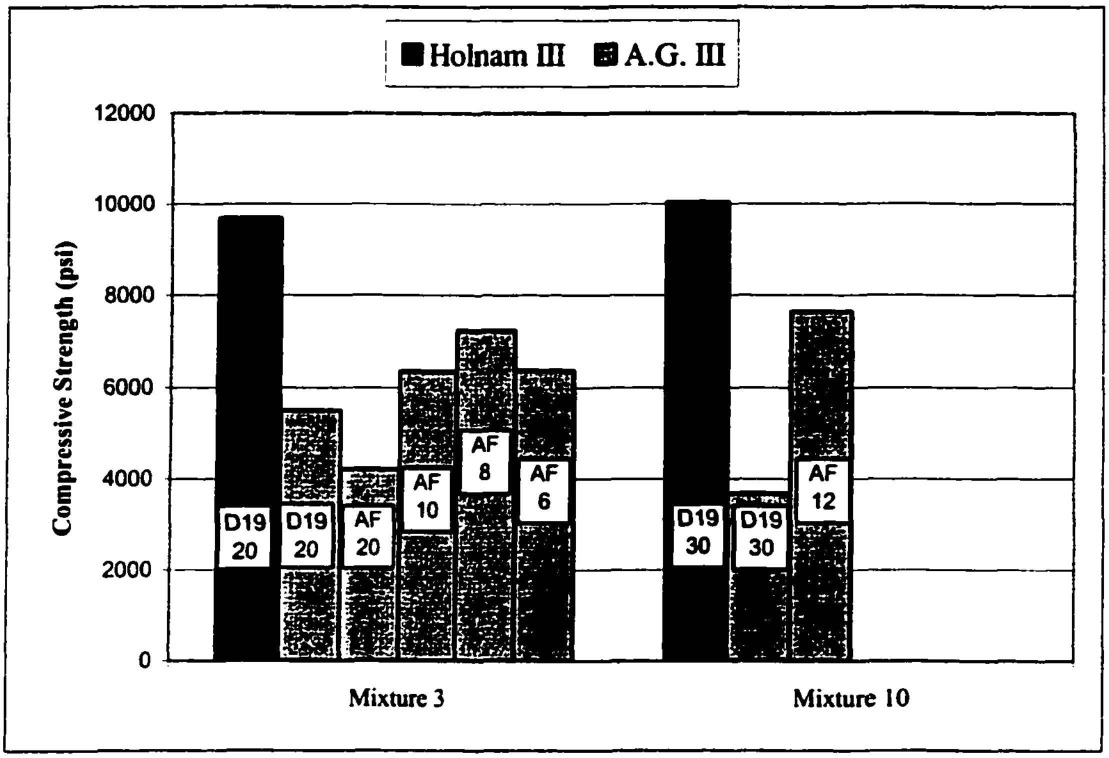

Figure 2.6. One-Day Compressive Strengths of AG III and Holnam III Mixtures.

2.5.2.3 Cement Quantity and Water to Cementitious Material Ratio. The goal of the next phase in the research program was to determine the optimum cement content and w/cm. The results are shown in Figures 2.7 and 2.8. All mixtures were cast with Ash Grove Type III Cement and contained ADVA Flow at a dosage rate of 22.5 fl oz/cwt. The cement content for the mixtures was 800, 900, or 1000 lb/yd^3. The w/cm for the mixtures was 0.24, 0.26, 0.28, or 0.30.

The results from the tests showed that increases in w/cm decreased the compressive strength at both ages, which was expected. Also, increases in cement content decreased one-day strengths, which are statistically different at 90 percent confidence intervals (Appendix A, Figure A.1). By 28 days of age the differences in the compressive strength of mixtures with like w/cm were becoming less pronounced.

Except for mixtures with a w/cm of 0.24, examination of the 90 percent confidence intervals in Figure A.2 (Appendix A) shows that mixtures with identical w/cm did not have statistically different strengths at 28 days of age. The mixtures with w/cm of 0.24 did not follow the same trend as the other mixtures. It is not clear why the 0.24 mixtures did not follow the same trend. The mixtures had sufficient workability to be mixed and placed properly.

The one-day strengths from these mixtures ranged from 2780 psi to 8390 psi. The slumps of the mixtures were between 3.5 in. to 10.0 in. The results from these mixtures showed that a cement content of 900 pounds per cubic yard of concrete showed the most promise because of its workability (slumps greater than 7 in.) and compressive strength (one day strengths greater than 7000 psi).

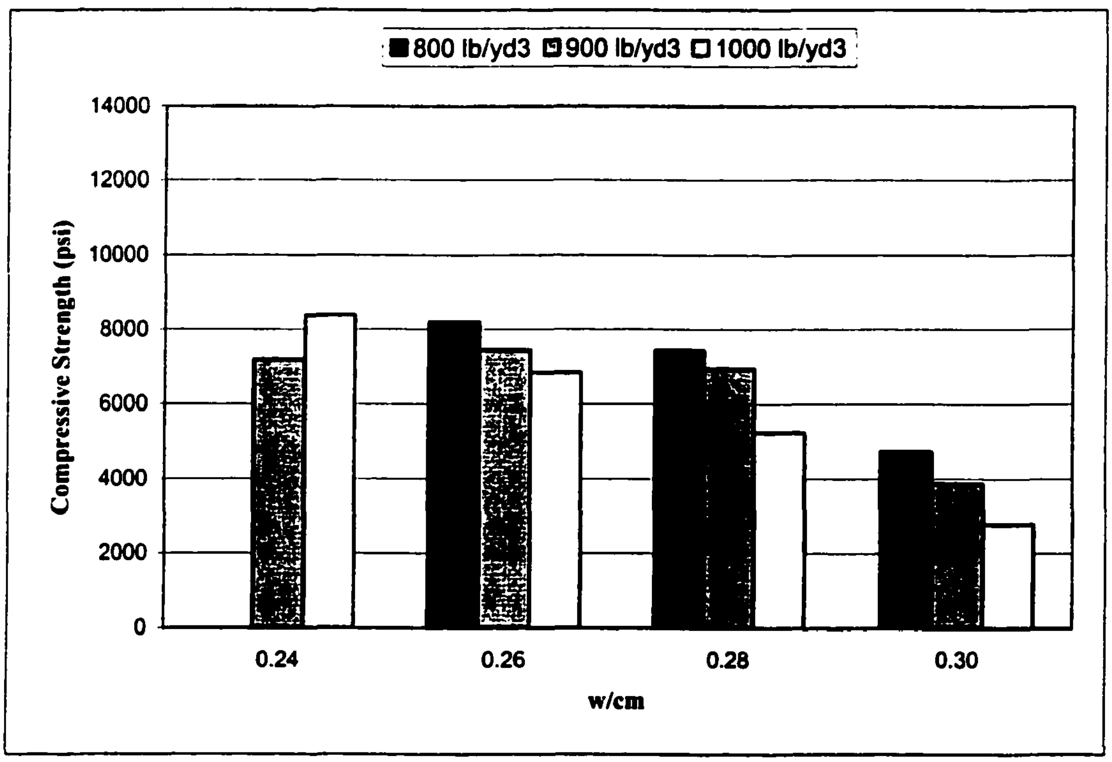

Figure 2.7. One Day Compressive Strengths for Optimizing Cement Content & w/cm.

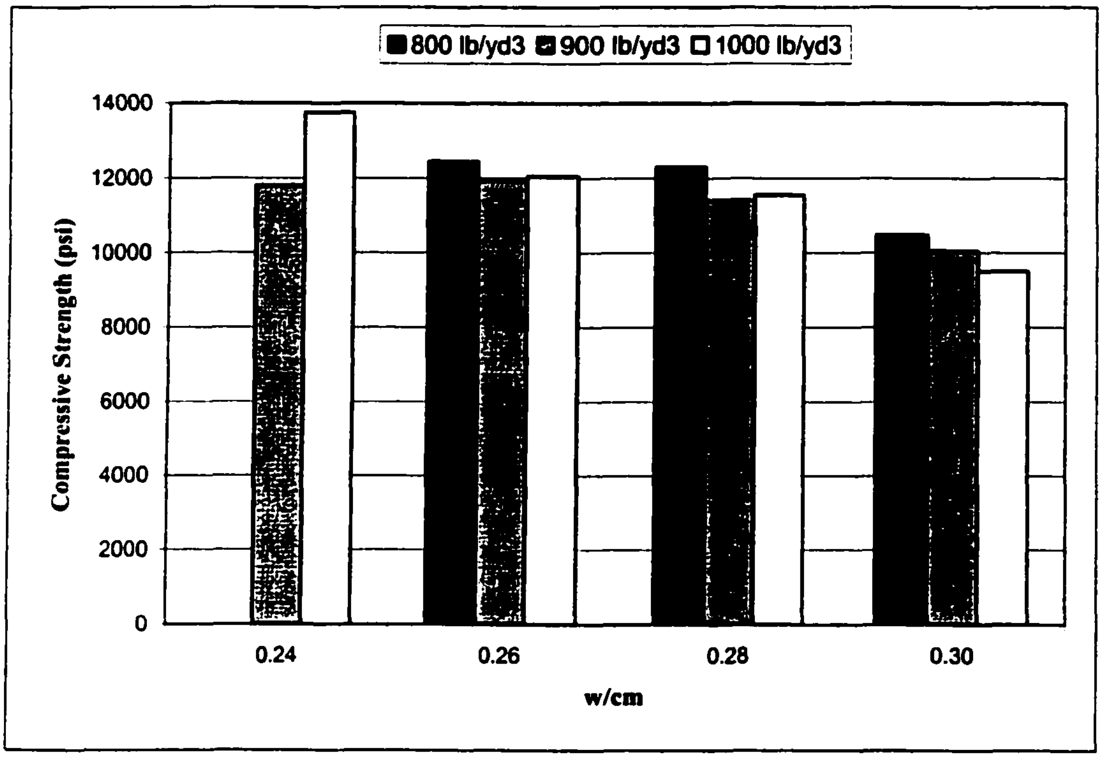

Figure 2.8. 28-Day Compressive Strengths for Optimizing Cement Content & w/cm.

2.5.2.4 High Range Water Reducer Dosage. As stated earlier, most mixtures previously cast had slumps of at least six inches. The objective of this phase of the research program was to determine the optimum HRWR dosage. The HRWR dosage was varied from 5 to 18 fl oz/cwt. All mixtures contained 900 lb/yd^3 of cement, and the HRWR used was ADVA Flow. The results from the compressive strength tests at one day are shown in Figure 2.9.

Based on compressive strength results alone, a HRWR dosage of 10 fl oz/cwt produced concrete with highest compressive strength at one day and near the highest compressive strength at 28 days of age. However, a HRWR dosage of 10 fl oz/cwt did not provide enough workability to allow a beam to be cast. At one day, the retarding effects of the HRWR can be seen in Figure 2.9. With the exception of the one mixture cast with 5 fl oz/cwt of HRWR, the mixtures containing smaller dosages of HRWR had higher compressive strengths. Examination of the 90 percent confidence intervals in Figure A.3 (Appendix A) shows that there are significant differences between the one-day compressive strengths of mixtures with w/cm of 0.26 and 0.28 at HRWR dosages of 10 and 15 fl oz/cwt. This trend is also evident between mixtures with HRWR dosages of 10 and 18 fl oz/cwt. For example, reducing the HRWR dosage from 18 fl oz/cwt to 10 fl ox/cwt increased the one-day compressive by 2330 psi for mixtures with a w/cm of 0.26.

By 28 days of age, the difference in compressive strength of all mixtures was less than 1500 psi. The compressive strengths for five of the eight mixtures were not statistically different (Figure A.4 in Appendix 4). This was expected because by 28 days of age the retarding effects of the HRWR are minimized. Based on workability and compressive strength, mixtures with a w/cm of 0.26 and HRWR of 15 fl oz/cwt displayed the most promise of producing concrete with the highest compressive with sufficient workability.

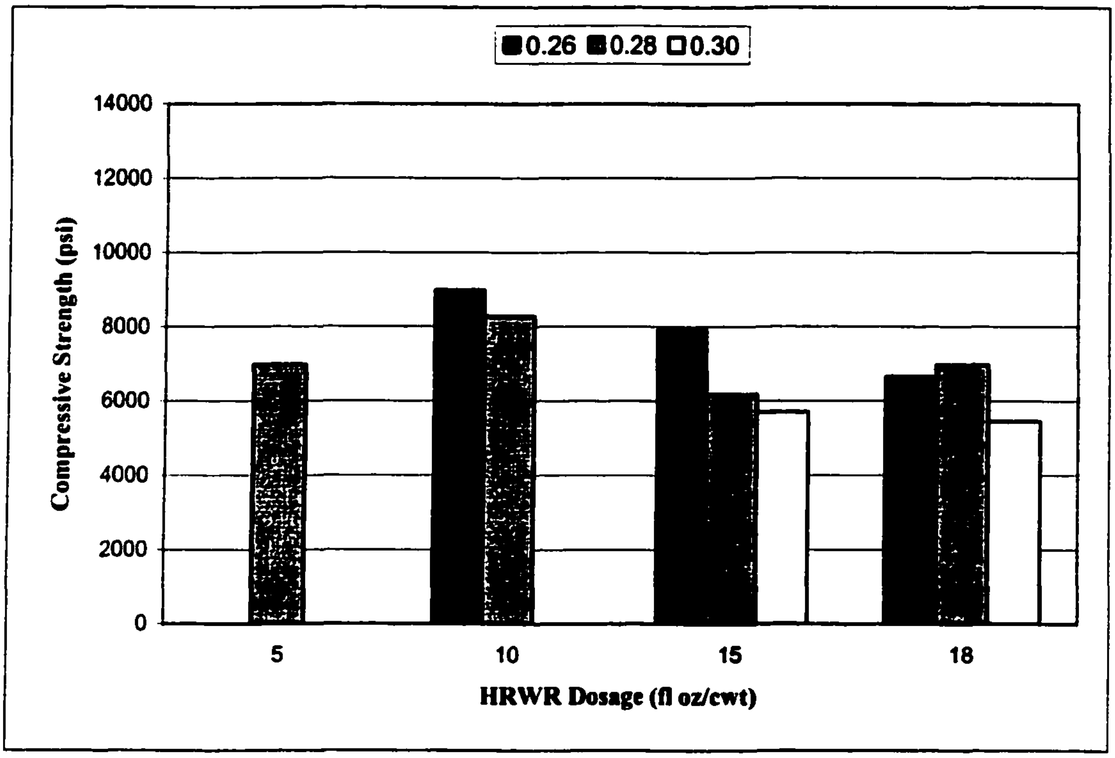

Figure 2.9. One-Day Compressive Strength for Optimizing HRWR Dosage.

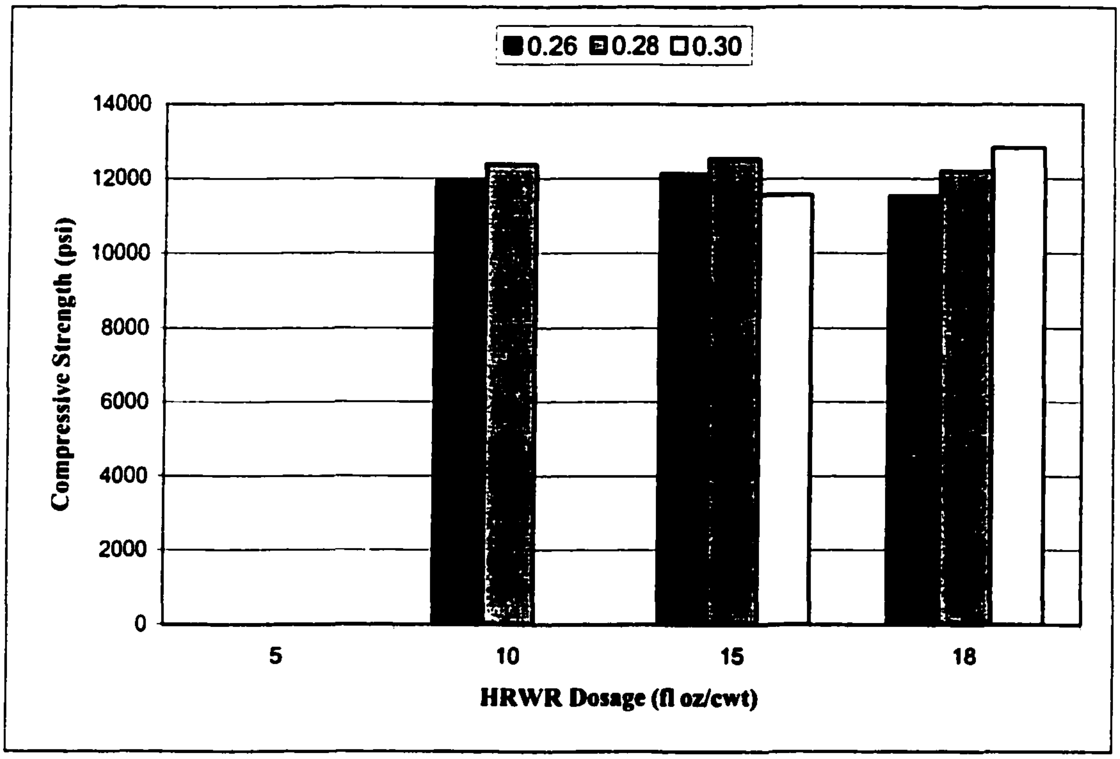

Figure 2.10. 28-Day Compressive Strength for Optimizing HRWR Dosage.

2.5.2.5 High Range Water Reducer and Water Reducer. The next phase of the research examined whether the addition of a WR could reduce the HRWR dosage. All mixtures contained 900 lb/yd^3 of cement. The compressive strength results at one day of age are shown in Figure 2.11. Once again, the mixtures with a HRWR dosage of 10 fl oz/cwt produced concrete with the greatest compressive strength at one day of age (9250 psi), which is statistically different through examination of the 90 percent confidence intervals (Figure A.5 in Appendix A). Even though the addition of a WR allowed for the reduction in the HRWR dosage, the slump or workability was too low to reduce the HRWR dosage below 15 fl oz/cwt. Lower dosages of HRWR were possible at higher w/cm, but the increase in w/cm decreased compressive strength at one day and 28 days of age.

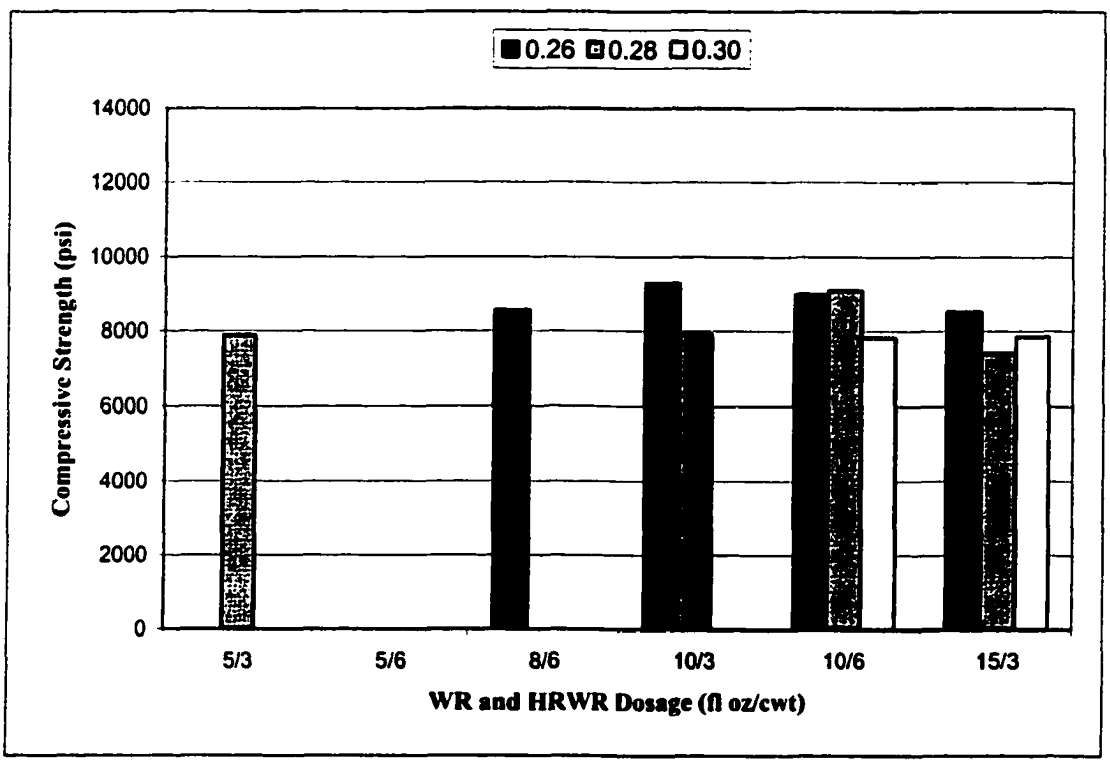

Figure 2.11. One-Day Compressive Strength for Optimizing HRWR and WR Dosage.

2.5.1.6 Accelerator. The final phase of this section of the research investigated the addition of an accelerator to increase the compressive strength at one day of age. The results from the study were listed in Table 2.26 and are shown in Figure 2.12. For all mixtures, the addition of the accelerator increased the one-day compressive strength. The differences for all mixtures except for 9-28-10 and 9-28-10D were statistically significant (Figure A.7 in Appendix A). For most mixtures, the increase in compressive strength was between 500 and 1500 psi. Even though all mixtures benefited from the addition of the accelerator, the substantial slump loss proved to be too great to benefit from the use of an accelerator.

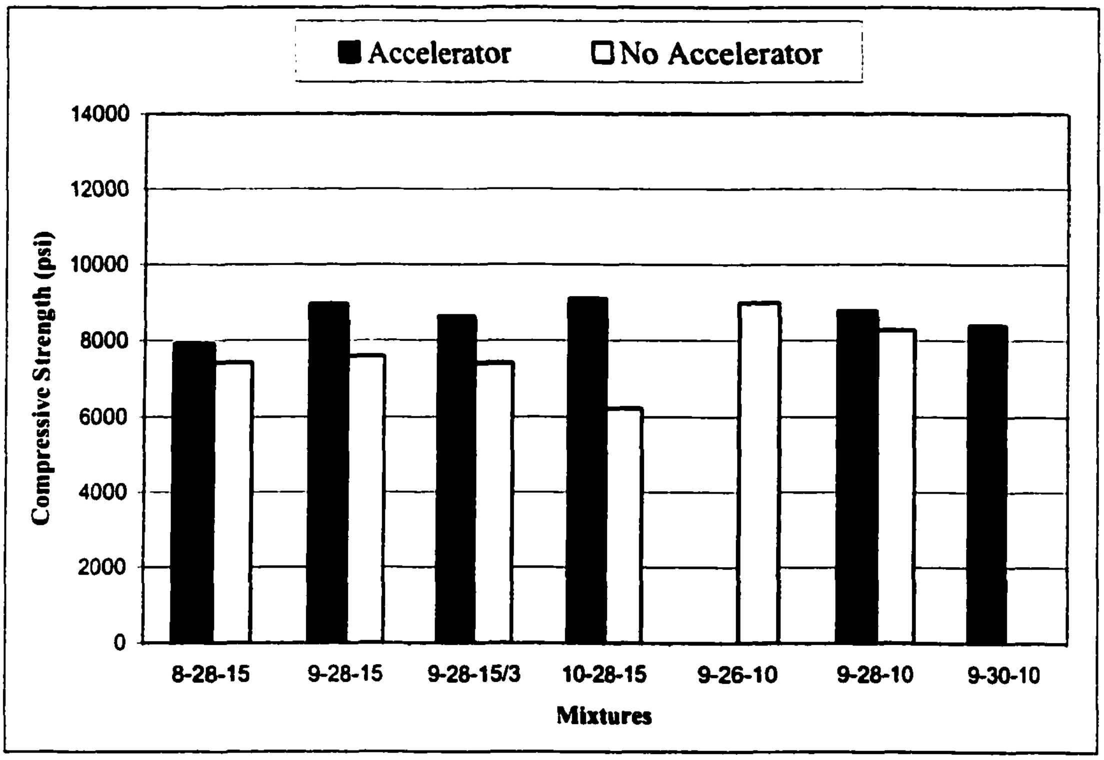

Figure 2.12. One-Day Compressive Strength Results for Examining Accelerator Dosage.

2.6 CONCLUSIONS

The results of the research program showed that HPC can be produced with materials that are native to Oklahoma. All materials used in the research program are currently being used by batch plants and prestressing facilities in the area. The research program determined that Holnam Type III Cement produced higher strengths than Ash Grove Type III Cement. However, concrete mixtures were cast with Ash Grove cement that achieved a one-day compressive strength of more than 9000 psi and a 28-day compressive strength of more than 14,000 psi, but the compressive strength of the Ash Grove mixture was still approximately 1000 psi less at both ages than the best performing Holnam mixture. A minimum w/cm of 0.26 and a maximum HRWR dosage of 15 fl oz/cwt were established as the lower and upper limits needed to produce concrete mixtures with sufficient workability. Extensive slump loss negated the use of an accelerator. The findings of this section of the research program are summarized below.

1. The cement source had little effect on the workability of the mixtures, which was expected because both cements had similar fineness. However, there were differences in the compressive strength. Concrete mixtures made with Ash Grove cement did not perform as well as concrete mixtures made with Holnam cement. Not only did the Ash Grove mixtures have significantly less strength (3000 to 5000 psi at one day), the cement appeared to react differently with the HRWR, Daracem 19. It is not clear

as to why Holnam cement produced concrete with greater compressive strengths.

2. For most mixtures, increases in cement content resulted in greater slumps for mixtures with like w/cm. The results showed that a minimum cement content of 900 lb/yd^3 was necessary for a concrete mixture with a w/cm equal to and less than 0.26 to have sufficient workability. For the mixtures tested, cement contents of 800 lb/yd^3 and greater produced workable concrete at w/cm of 0.28 and greater. Results from these mixtures showed that a cement content of 900 pounds per cubic yard of concrete showed the most promise because of its workability (slumps greater than 7 in.) and compressive strength (one day strengths greater than 7000 psi).
3. The addition of HRWR increased workability. Too much HRWR delayed set times and decreased early age strength. At w/cm between 0.26 to 0.30, a dosage rate of 15 fl oz/cwt was necessary to produce concrete with sufficient workability. Lower dosages of HRWR were possible with the use of a WR but only at higher w/cm. Based on workability and compressive strength, mixtures with a w/cm of 0.26 and HRWR of 15 fl oz/cwt displayed the most promise of producing concrete with the highest compressive strength with sufficient workability.
4. The addition of an accelerator increased the one-day compressive strength. For most mixtures, the increase in compressive strength was between 500 and 1500 psi. Even though all mixtures benefited from the addition of the

accelerator, the substantial slump loss proved to be too great to benefit from the use of an accelerator.

5. Based on compressive strength and workability, the results of research program indicated that for the aggregates and cements tested, the optimum cement content was 900 lb/yd^3. The optimum w/cm was 0.26 and the optimum coarse aggregate content was 1790 lb/yd^3. This mix proportion will be used to cast the precast/prestressed bridge girders that will be examined in later phases of the research program.

CHAPTER 3

DEVELOPING HPC WITH ENTRAINED AIR

3.1 INTRODUCTION

The second phase in the research program developed High Performance Concrete (HPC) with entrained air. It is necessary entrain air in some concrete structures that are subjected to freezing and thawing cycles. Entrained air is added to concrete through the use of air entraining agents (AEA). Current building codes require varying amounts of entrained air depending on the severity of the exposure. Entrained air voids provide air pockets where water can expand and water pressure can be relieved. Without these voids, continuous freeze/thaw cycles will eventually degrade and damage the concrete. A total air content between four and eight percent is generally considered adequate to provide resistance to the freeze/thaw action (Mindess et al, 1981).

The chapter will show the development of HPC with entrained air and also the effects of entrained air on the fresh and hardened properties of concrete. As with the mixtures developed in Chapter 2, mixtures will be cast that incorporate materials that are native to and commonly used in Oklahoma. Based on this material work, two prestressed concrete girders were cast with one of the mixtures developed during this portion of the research. The water to cementitious material ratio (w/cm), air content, and the type and dosage of admixtures are all variables that were examined.

3.2 LITERATURE REVIEW

There are two types of air voids in concrete, entrapped air and entrained air. Entrapped air voids become entrapped in the concrete during the mixing of the concrete. These voids typically have a diameter of 0.04 in. or larger (Kosmatka et al, 1994). They are randomly spaced throughout out the concrete. Entrapped air voids normally make up approximately one to two percent of the total volume of a concrete mixture. Entrained air voids are introduced into the concrete through the use of chemical admixtures called air-entraining agents (AEA). The AEA are commercial, chemical admixtures that when added to the concrete during mixing produce tiny air bubbles in the concrete. There may be as many as 300 billion bubbles in a cubic yard of concrete with a total air content of four to six percent by volume (Kosmatka et al, 1994). Entrained air voids generally have a diameter of about 0.002-in. (Neville, 1997) and are evenly distributed throughout the concrete. Entrained air voids are typically spaced within 0.008 in. of each other (Derucher et al, 1994).

It is necessary to add AEA to some concrete structures that are subjected to freezing and thawing. The Building Code Requirements for Structural Concrete and Commentary requires varying amounts of entrained air depending on the severity of exposure and nominal maximum aggregate size. Entrained air voids provide air pockets where water can expand and water pressure can be relieved. Without these voids, continuous freeze/thaw cycles will eventually degrade and damage the concrete. For all concrete structures, a total air content between four and eight percent is generally considered adequate to provide resistance to the freeze/thaw action (Mindess et al, 1981).

Although the addition of entrained air improves the concrete's freeze/thaw durability, entrained air has a negative effect on strength. Entrained air decreases the compressive strength of both NSC and HPC. As a rule of thumb, an increase in the total air content of one percent decreases compressive strength two to five percent (Mindess et al, 1981). For example, a concrete mixture with a total air content of seven percent may have a compressive strength that is 25 percent less than an identical mixture with a total air content of two percent.

3.3 EXPERIMENTAL PROCEDURES

3.3.1 Scope

This portion of the research program developed HPC with entrained air. With one of the mixtures developed, two prestressed/precast concrete girders were produced. Therefore, strength and durability were not the only concern; the concrete mixtures must have sufficient workability. The mixtures required enough workability to be used at a local prestressed concrete facility. Another necessity was the need for the mixtures to contain the materials and admixtures that were currently in use at the facility. The total air content, w/cm, and the type and dosage of admixtures were varied in the research. A goal of the research program was to produce HPC with entrained air and also to examine the effects of entrained air on the properties of the fresh and hardened concrete, specifically the compressive strength. The research results showed that air entrained HPC is possible using materials native to Oklahoma. However, the research results also showed that trial batching is necessary to determine the appropriate dosage of AEA at w/cm less 0.34.

3.3.2 Materials

The type of fine and coarse aggregate was constant for all mixtures. A fine aggregate from Dover, Oklahoma was used. The coarse aggregate was 3/8-in., crushed limestone from Davis, Oklahoma. The material properties of the fine and coarse aggregates are shown in Table 2.1 of Chapter 2. Ash Grove Type III Cement from Chanute, KS was used in all mixtures. The chemical compositions and blaine fineness of the cement are shown in Table 2.2 of Chapter 2. When needed, a water reducer, WRDA with Hycol, and a high range water reducer, ADVA Flow, were added to the mixtures to provide adequate workability. To achieve the necessary amounts of entrained air, an air-entraining agent (AEA), DARAVAIR 1000 was added to the mixtures.

3.3.3 Variables

The variables examined in the research were the w/cm, admixture dosage, and the total air content. The w/cm examined were 0.26, 0.28, 0.30, 0.34, 0.36, 0.42, and 0.50. When needed a WR and/or a HRWR were added to the mixtures to provide sufficient workability. The targeted total air content was 2, 4, and 6 percent at each w/cm.

3.3.4 Mixtures

The mixture designs are shown in Tables 3.1 through Table 3.9. Each mixture is designated by the quantity of cement in the mixture, the w/cm of the mixture, and the measured total air content of the fresh concrete. For example, Mixture 9-26-1.8 contains 900 lb/yd^3 of cement and has a w/cm of 0.26. The measured air content of the fresh concrete was 1.8 percent.

The targeted total air contents for the mixtures were 2, 4, and 6 percent. Many mixtures required several trial batches in order to achieve the desired air content. Mixtures were cast until the measured air content was within ± 0.5 percent of the desired air content. Mixtures with air contents out of this range were discarded.

Table 3.1. Mixture Proportions.

Materials	Mixture Designations				
	9-26-1.8	9-26-2.1	9-26-2.4	9-26-3.1	9-26-3.3
Cement (lb/yd^3)	900	900	900	900	900
Coarse Agg. (lb/yd^3)	1790	1790	1790	1790	1790
Fine Agg. (lb/yd^3)	1217	1128	1217	1040	1040
Water (lb/yd^3)	234	234	234	234	234
HRWR (fl oz/cwt)	15	15	15	15	15
WR (fl oz/cwt)	3	3	3	3	3
AEA (fl oz/cwt)	0	4	0	10	15
Calculated Air Content (%)	2	4	2	6	6
w/cm	0.26	0.26	0.26	0.26	0.26

Table 3.2. Mixture Proportions Continued.

Materials	Mixture Designations				
	9-26-3.5	9-26-3.8	9-26-3.9	9-26-4.5	9-26-5.6
Cement (lb/yd^3)	900	900	900	900	900
Coarse Agg. (lb/yd^3)	1790	1790	1790	1790	1790
Fine Agg. (lb/yd^3)	1040	1128	1128	1040	1040
Water (lb/yd^3)	234	234	234	234	234
HRWR (fl oz/cwt)	15	15	15	15	15
WR (fl oz/cwt)	3	3	3	3	3
AEA (fl oz/cwt)	15	10	10	17.5	15
Calculated Air Content (%)	6	4	4	6	6
w/cm	0.26	0.26	0.26	0.26	0.26

Table 3.3. Mixture Proportions Continued.

Materials	Mixture Designations			
	9-26-5.9	9-26-9.1	9-26-13	9-26-13
Cement (lb/yd^3)	900	900	900	900
Coarse Agg. (lb/yd^3)	1790	1790	1790	1790
Fine Agg. (lb/yd^3)	1040	1040	1040	1040
Water (lb/yd^3)	234	234	234	234
HRWR (fl oz/cwt)	15	15	15	15
WR (fl oz/cwt)	3	3	3	3
AEA (fl oz/cwt)	15	20	30	17
Calculated Air Content (%)	6	6	6	6
w/cm	0.26	0.26	0.26	0.26

Table 3.4. Mixture Proportions Continued.

Materials	Mixture Designations				
	9-26-17	9-28-1.9	9-28-4.1	9-28-5.8	9-30-1.1
Cement (lb/yd^3)	900	900	900	900	900
Coarse Agg. (lb/yd^3)	1790	1790	1790	1790	1790
Fine Agg. (lb/yd^3)	1040	1169	1081	992	1122
Water (lb/yd^3)	234	252	252	252	270
HRWR (fl oz/cwt)	15	15	15	12	10
WR (fl oz/cwt)	3	3	3	3	3
AEA (fl oz/cwt)	20	0	8	12	0
Calculated Air Content (%)	6	2	4	6	2
w/cm	0.26	0.28	0.28	0.28	0.30

Table 3.5. Mixture Proportions Continued.

Materials	Mixture Designations				
	9-30-2.1	9-30-4.5	9-30-4.9	9-30-5.7	9-30-6.8
Cement (lb/yd^3)	900	900	900	900	900
Coarse Agg. (lb/yd^3)	1790	1790	1790	1790	1790
Fine Agg. (lb/yd^3)	1122	1034	1034	945	1034
Water (lb/yd^3)	270	270	270	270	270
HRWR (fl oz/cwt)	12	8	4	10	10
WR (fl oz/cwt)	3	3	3	3	3
AEA (fl oz/cwt)	0	0.25	1	1	6
Calculated Air Content (%)	2	4	4	6	4
w/cm	0.30	0.30	0.30	0.30	0.30

Table 3.6. Mixture Proportions Continued.

Materials	Mixture Designations				
	9-30-9.1	9-30-11	9-30-15	9-34-2.2	9-34-3
Cement (lb/yd^3)	900	900	900	900	900
Coarse Agg. (lb/yd^3)	1790	1790	1790	1790	1790
Fine Agg. (lb/yd^3)	1034	1034	1034	1028	939
Water (lb/yd^3)	270	270	270	306	306
HRWR (fl oz/cwt)	12	8	8	8	2
WR (fl oz/cwt)	3	3	3	3	3
AEA (fl oz/cwt)	4	4	2	0	0.25
Calculated Air Content (%)	4	4	4	2	4
w/cm	0.30	0.30	0.30	0.34	0.34

Table 3.7. Mixture Proportions Continued.

Materials	Mixture Designations				
	9-34-5.1	9-34-11	9-34-13	9-34-17	9-36-2.6
Cement (lb/yd^3)	900	900	900	900	900
Coarse Agg. (lb/yd^3)	1790	1790	1790	1790	1790
Fine Agg. (lb/yd^3)	939	939	939	939	980
Water (lb/yd^3)	306	306	306	306	324
HRWR (fl oz/cwt)	2	8	5	5	2.5
WR (fl oz/cwt)	3	3	3	3	3
AEA (fl oz/cwt)	0.5	4	1	3	0
Calculated Air Content (%)	4	4	4	4	2
w/cm	0.34	0.34	0.34	0.34	0.36

Table 3.8. Mixture Proportions Continued.

Materials	Mixture Designations				
	9-36-3.8	9-36-6.2	9-42-2.1	9-42-1.8	9-42-3.5
Cement (lb/yd^3)	900	900	900	900	900
Coarse Agg. (lb/yd^3)	1790	1790	1790	1790	1790
Fine Agg. (lb/yd^3)	892	803	839	839	750
Water (lb/yd^3)	324	324	378	378	378
HRWR (fl oz/cwt)	3	3	0	0	0
WR (fl oz/cwt)	3	3	3	0	3
AEA (fl oz/cwt)	1.0	1.5	0	0	0.25
Calculated Air Content (%)	4	6	2	2	4
w/cm	0.36	0.36	0.42	0.42	0.42

Table 3.9. Mixture Proportions Continued.

Materials	Mixture Designations				
	9-42-4.4	9-42-5.9	9-50-0.9	9-50-3.6	9-50-6.6
Cement (lb/yd^3)	900	900	900	900	900
Coarse Agg. (lb/yd^3)	1790	1790	1790	1790	1790
Fine Agg. (lb/yd^3)	750	662	650	561	473
Water (lb/yd^3)	378	378	450	450	450
HRWR (fl oz/cwt)	0	0	0	0	0
WR (fl oz/cwt)	0	0	0	0	0
AEA (fl oz/cwt)	1.0	1.25	0	0.5	0.75
Calculated Air Content (%)	4	6	2	4	6
w/cm	0.42	0.42	0.50	0.50	0.50

3.3.5 Batching

The batching of the mixtures followed the same procedures and sequences that were described in Chapter 2. For these mixtures, the AEA was added to the mixtures once they achieved a uniform consistency

For the research program, the desired fresh concrete temperature was 70°F. Since the majority of batching was conducted during the summer, crushed ice was added to the mixing water. For those mixtures cast in cooler months, warm tap water was used to increase the fresh concrete temperature.

3.3.6 Curing

All the concrete specimens were cured at 73°F and 50 percent relative humidity (RH) for the first 24 hours. The specimens were then removed from their molds and moist cured in lime-saturated water at 73°F until time of testing.

3.3.7 Tests

The mixtures were subjected to several tests. The fresh properties examined were concrete temperature (ASTM C 1064), air content (ASTM C 231) and slump (ASTM C 143). The compressive strength was the hardened property tested. The compressive strength (ASTM C 39) was tested at one, seven, 28, and 56 days of age. At each age, three 4 x 8 in. cylinders were tested. The durability and permeability of the mixtures are examined in Chapters 4 and 5.

3.4 PRESENTATION OF RESULTS

3.4.1 Fresh Concrete Results

The fresh concrete results are shown below in Tables 3.10, 3.11, and 3.12. The fresh concrete temperature was between 70 and 80 F for most mixtures. The slumps for the mixtures ranged from 1.50 to 12 inches. The measured air contents of the fresh concrete ranged from 0.9 to 17 percent.

Table 3.10. Fresh Concrete Results.

Mixtures	Concrete Temp. (F)	AEA Dosage (fl oz/cwt)	Air Content (%)	Slump (in.)
9-26-1.8	80	0	1.8	9.00
9-26-2.1	75	4.0	2.1	9.00
9-26-2.4	74	0	2.4	6.50
9-26-3.1	71	10.0	3.1	5.50
9-26-3.3	75	15.0	3.3	7.25
9-26-3.5	74	15.0	3.5	9.00
9-26-3.8	77	10.0	3.8	8.00
9-26-3.9	70	10.0	3.9	8.50
9-26-4.5	85	17.5	4.5	4.50
9-26-5.6	75	15.0	5.6	4.25
9-26-5.9	71	15.0	5.9	8.75
9-26-9.1	70	20.0	9.1	11.00
9-26-13	70	30.0	13.0	8.25
9-26-13	72	17.0	13.0	12.00
9-26-17	74	20.0	17.0	12.00

Table 3.11. Fresh Concrete Results Continued.

Mixtures	Concrete Temp. (F)	AEA Dosage (fl oz/cwt)	Air Content (%)	Slump (in.)
9-28-1.9	81	0	1.9	9.00
9-28-4.1	75	8.0	4.1	11.00
9-28-5.8	71	12.0	5.8	9.50
9-30-1.1	73	0	1.1	10.00
9-30-2.1	80	0	2.1	8.00
9-30-4.5	77	0.25	4.5	1.50
9-30-4.9	74	1	4.9	2.25
9-30-5.7	72	1.0	5.7	10.75
9-30-6.8	71	6.0	6.8	9.75
9-30-9.1	75	4.0	9.1	11.50
9-30-11	69	4.0	11.0	9.50
9-30-15	70	2.0	15.0	10.25
9-34-2.2	81	0	2.2	8.50
9-34-3	78	0.25	3.0	4.25
9-34-5.1	74	0.5	5.1	4.50
9-34-11	75	4.0	11.0	11.00
9-34-13	69	1.0	13.0	11.00
9-34-17	70	3.0	17.0	11.00

Table 3.12. Fresh Concrete Results Continued.

Mixtures	Concrete Temp. (F)	AEA Dosage (fl oz/cwt)	Air Content (%)	Slump (in.)
9-36-2.6	75	0	2.6	2.50
9-36-3.8	71	1.0	3.8	3.50
9-36-6.2	75	1.5	6.2	2.00
9-42-2.1	71	0	2.1	5.00
9-42-1.8	71	0	1.8	3.25
9-42-3.5	74	0.25	3.5	9.25
9-42-4.4	80	1.0	4.4	3.75
9-42-5.9	70	1.25	5.9	4.00
9-50-0.9	65	0	0.9	6.25
9-50-3.6	75	0.5	3.6	9.50
9-50-6.6	71	0.75	6.6	10.00

3.4.2 Hardened Concrete Results

The compressive strength was tested at one, seven, 28, and 56 days. Each compressive strength result is the average of three 4 x 8 in. cylinders tests. The individual tests, standard deviations, and confidence intervals are reported in Appendix B. Mixtures with air contents out of the specified range were not tested. The durability and permeability are examined in later chapters. The test results are shown in Tables 3.13, 3.14, and 3.15.

Table 3.13. Compressive Strength Results.

Mixtures	Average Compressive Strength (psi)			
	1 Day[1]	7 Day[1]	28 Day[1]	56 Day[1]
9-26-1.8	6010	11,630	12,520	13,900
9-26-2.1	6470	10,790	11,590	-
9-26-2.4	6830	11,040	12,190	12,960
9-26-3.1	-	-	-	-
9-26-3.3	-	-	-	-
9-26-3.5	-	-	-	-
9-26-3.8	4830	10,440	11,270	12,090
9-26-3.9	6740	10,590	12,160	12,030
9-26-4.5	-	-	-	-
9-26-5.6	4560	9270	10,560	11,290
9-26-5.9	6670	9920	11,380	12,080
9-26-9.1	-	-	-	-
9-26-13	-	-	-	-
9-26-13	-	-	-	-
9-26-17	-	-	-	-

1) Average compressive strength of three tests.
2) Individual tests and statistical data are shown in Appendix B.

Table 3.14. Compressive Strength Results Continued.

Mixtures	Average Compressive Strength (psi)			
	1 Day[1]	7 Day[1]	28 Day[1]	56 Day[1]
9-28-1.9	5540	10,650	11,870	12,460
9-28-4.1	6410	9140	10,770	11,570
9-28-5.8	6360	9520	10,080	10,940
9-30-1.1	6150	10,050	12,190	13,600
9-30-2.1	5580	9060	10,430	11,480
9-30-4.5	5610	8650	9960	11,000
9-30-4.9	4580	7290	8340	8880
9-30-5.7	6000	9130	10,270	11,030
9-30-6.8	5720	8490	9830	10,090
9-30-9.1	-	-	-	-
9-30-11	-	-	-	-
9-30-15	-	-	-	-
9-34-2.2	5160	8760	9940	10,530
9-34-3	4450	7090	8640	8840
9-34-5.1	4130	6740	7540	8120
9-34-11	-	-	-	-
9-34-13	-	-	-	-
9-34-17	-	-	-	-

1) Average compressive strength of three tests.
2) Individual tests and statistical data are shown in Appendix B.

Table 3.15. Compressive Strength Results Continued.

Mixtures	Average Compressive Strength (psi)			
	1 Day[1]	7 Day[1]	28 Day[1]	56 Day[1]
9-36-2.6	4220	7210	9180	9620
9-36-3.8	4580	6960	8340	9040
9-36-6.2	3660	6790	7700	8450
9-42-2.1	3050	5940	7480	7540
9-42-1.8	2690	6760	8691	9050
9-42-3.5	2950	5390	6490	6770
9-42-4.4	2750	4940	6470	6860
9-42-5.9	2640	4900	6110	6710
9-50-0.9	2020	4730	6220	6990
9-50-3.6	1560	5220	6340	6760
9-50-6.6	1650	4320	5560	6150

1) Average compressive strength of three tests.
2) Individual tests and statistical data are shown in Appendix B.

3.5 DISCUSSION OF RESULTS

3.5.1 Fresh Concrete Results

For mixtures with a w/cm equal to or less than 0.36, WR and HRWR were used to achieve adequate workability. As expected, higher dosages of HRWR were required at lower w/cm. The addition of AEA was observed to increase the workability of concrete. This is because the entrained air behaves as "small ball bearings" that help lubricate the mixture. The slumps of mixtures with w/cm of 0.26, 0.34, and 0.50 did increase as the total air content increased, but the slumps of the remaining mixtures did not necessarily increase with increases in air content.

At lower w/cm, higher dosages of AEA were required to achieve the targeted total air contents. Shown in Figures 3.1 and 3.2 are the dosages of AEA necessary to attain the targeted air contents of four and six percent. From the figures, it is evident that mixtures below a w/cm of 0.30 require higher dosages of AEA to achieve the desired air contents. Below a w/cm of 0.30, 12 and 15 fl oz/cwt of AEA were necessary to attain an air content of six percent. This dosage rate is significantly more than the manufacturer's recommended dosage rate of 3/4 to 3 fl. oz/cwt. At w/cm greater than 0.30, the dosage rates required were within the manufacturer's recommendation. The higher dosage rates were necessary due to insufficient amounts of water in mixtures with w/cm of 0.26 and 0.28.

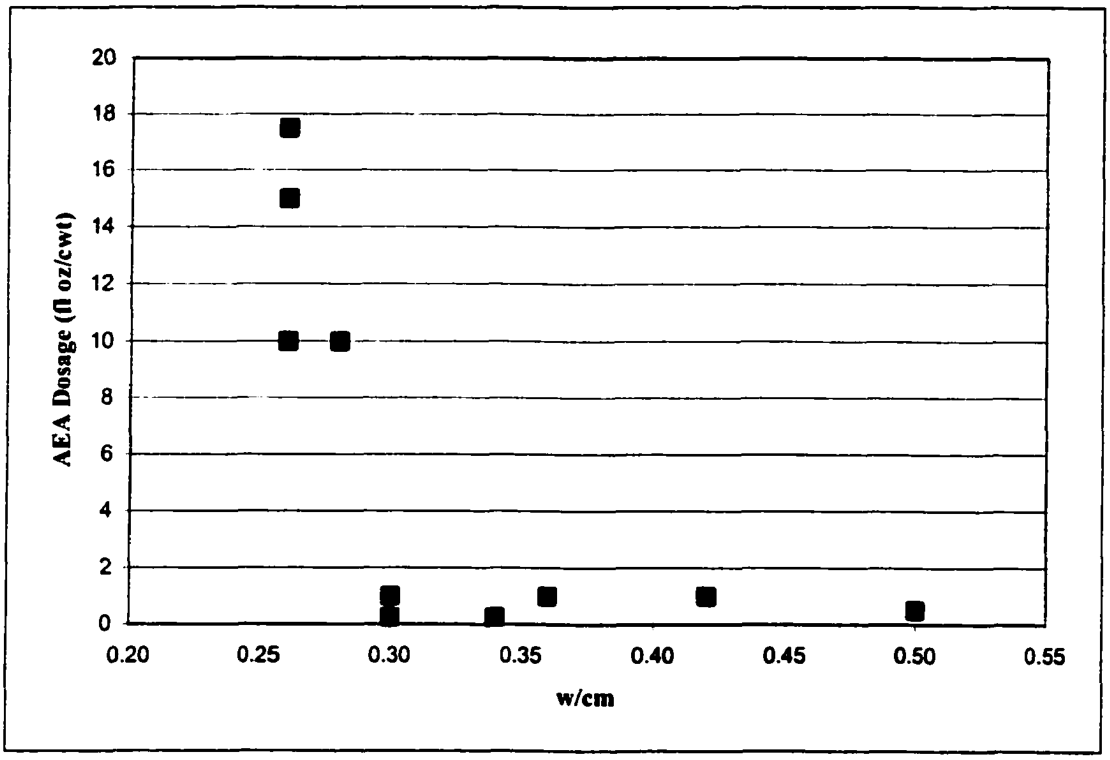

Figure 3.1. AEA Dosage for 4 percent Total Air Content.

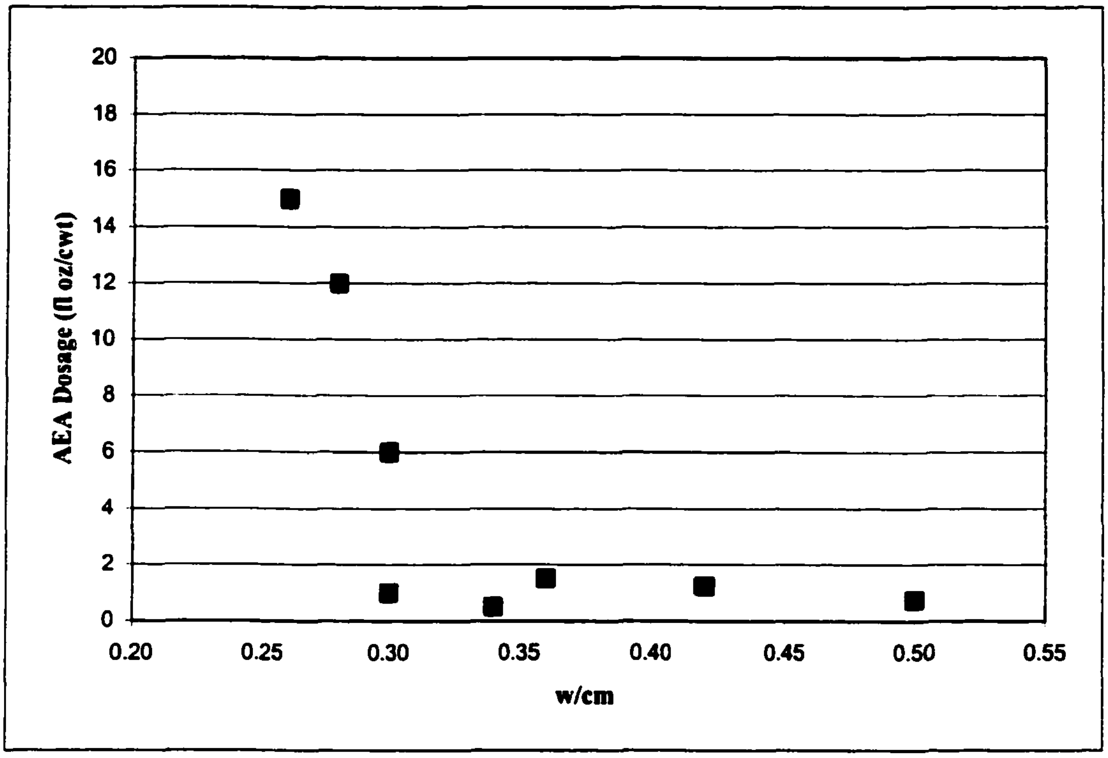

Figure 3.2. AEA Dosage for 6 percent Total Air Content.

As previously stated, many mixtures were cast several times in order to achieve the targeted total air contents of four and six percent. Mixtures with w/cm of 0.26 and 0.30 were repeated numerous times. Shown in Figure 3.3 are the dosages of AEA and the measured air contents of several mixtures. All mixtures contained the same quantity of cement and coarse aggregate, and all mixtures had a w/cm of 0.26. Also, all mixtures were mixed for approximately the same length of time in the same concrete mixer. The only variable in the mixtures was the concrete temperature. The fresh concrete temperature ranged from 70 to 85 F. The manufacturer's specifications warn that as temperatures increase, the admixture loses its air entraining abilities. They also warn that increases in mixing time cause higher air contents. These reasons may account for some of the variations in air content but not all. Some mixtures were cast twice with the same AEA dosage (17.5 and 20 fl oz/cwt) and had significant differences in measured air content (greater than 7 percent difference in air content at each w/cm). The results from the tests further reinforce the need for trial batching. At low w/cm, trial batching is necessary examine the fresh concrete properties such as workability and air content and to determine the correct dosages of admixtures.

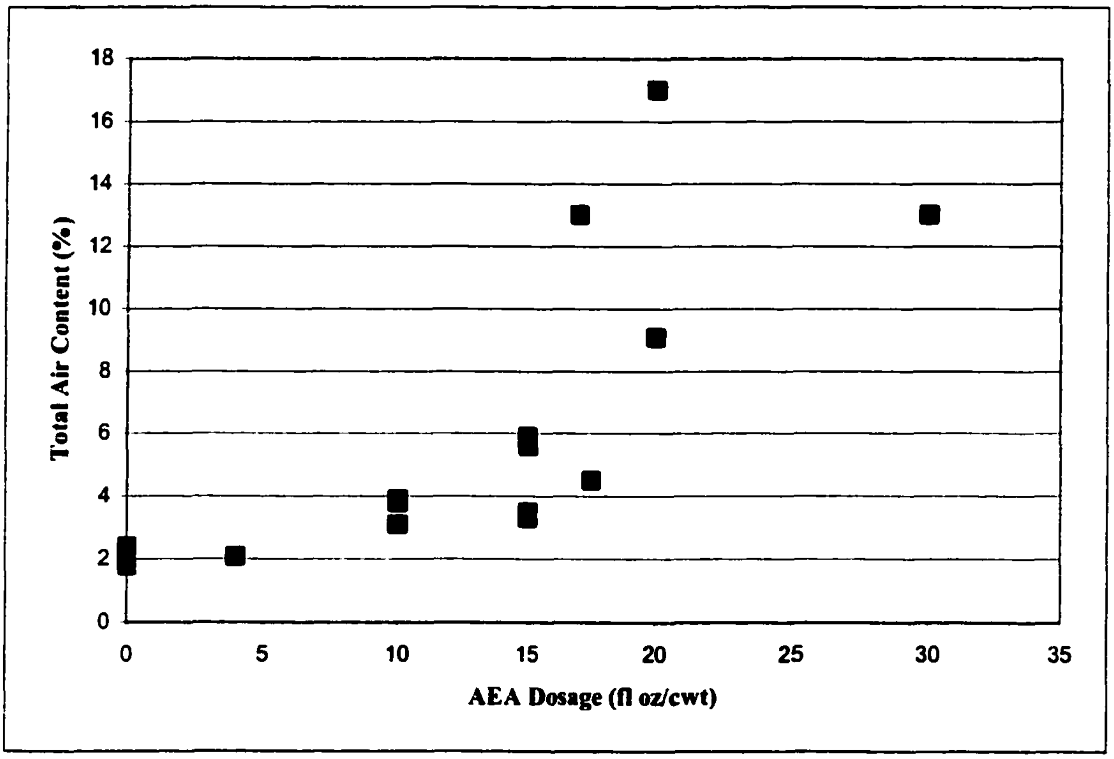

Figure 3.3. AEA Dosage versus Total Air Content for w/cm of 0.26.

3.5.2 Compressive Strength Results

The addition of entrained air decreases the compressive strength of concrete. This is because air is concrete. The decrease in compressive strength normally ranges from two to five percent for every one percent increase in entrained air. Therefore, a mixture with an air content of six percent may have a compressive strength that is 20 percent less than an identical mixture with no entrained air (2 percent air). The compressive strength results are shown in Figures 3.4, 3.5, and 3.6. Where there were multiple mixtures at a given air content, the results shown in the figures are of the mixtures whose measured air content was closest to the targeted air content.

At one day of age, the compressive strength results followed a general trend that is substantiated by examination of the 90 percent confidence intervals in Figure B.1 (Appendix B). Increases in w/cm resulted in decreases in compressive strength, which was expected. However, one would also expect that increases in air content (for mixtures with like w/cm) would decrease compressive strength. This was not the trend for most mixtures. For example, the mixture with the greatest air content had the highest compressive strength at one day for mixtures cast at a w/cm of 0.30. The confidence intervals (Figure B.1 in Appendix B) overlap for most mixtures at each w/cm, which means that the differences between mixtures are not statistically significant. It is unclear as to why the mixtures did not follow the expected trends at one day of age. But, by 28 and 56 days of age, the expected trend (the lower the air content, the greater the compressive strength) became apparent.

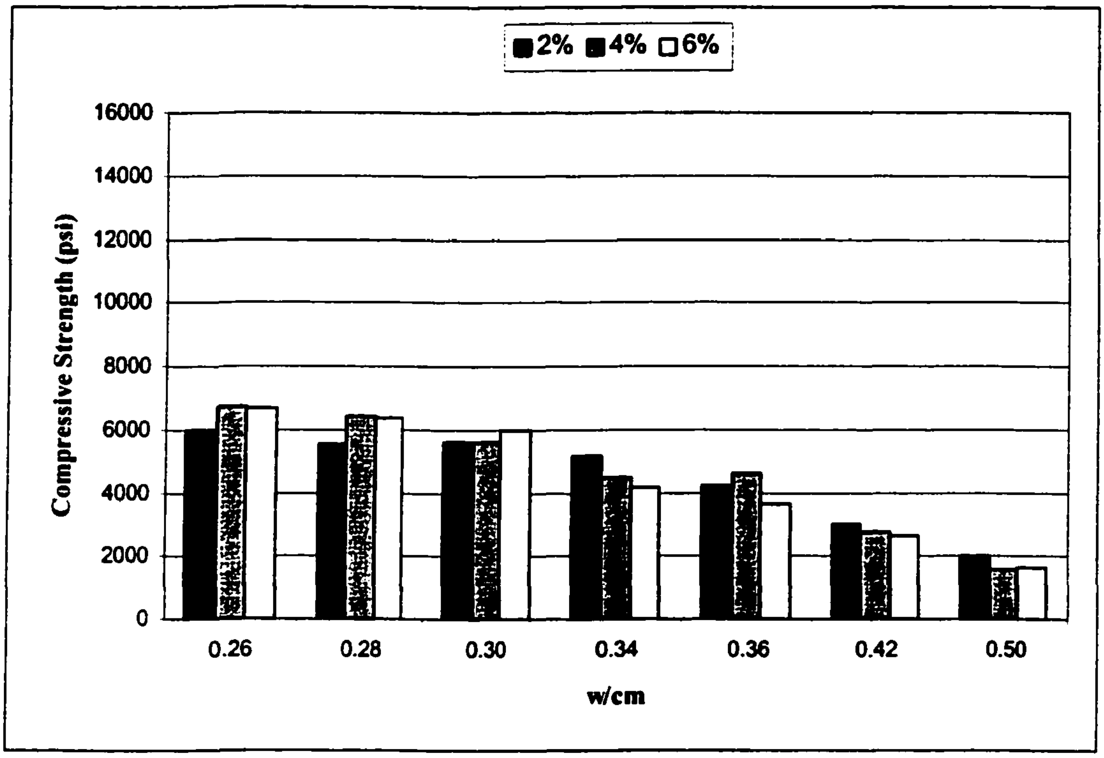

Figure 3.4. Compressive Strength Results at One Day.

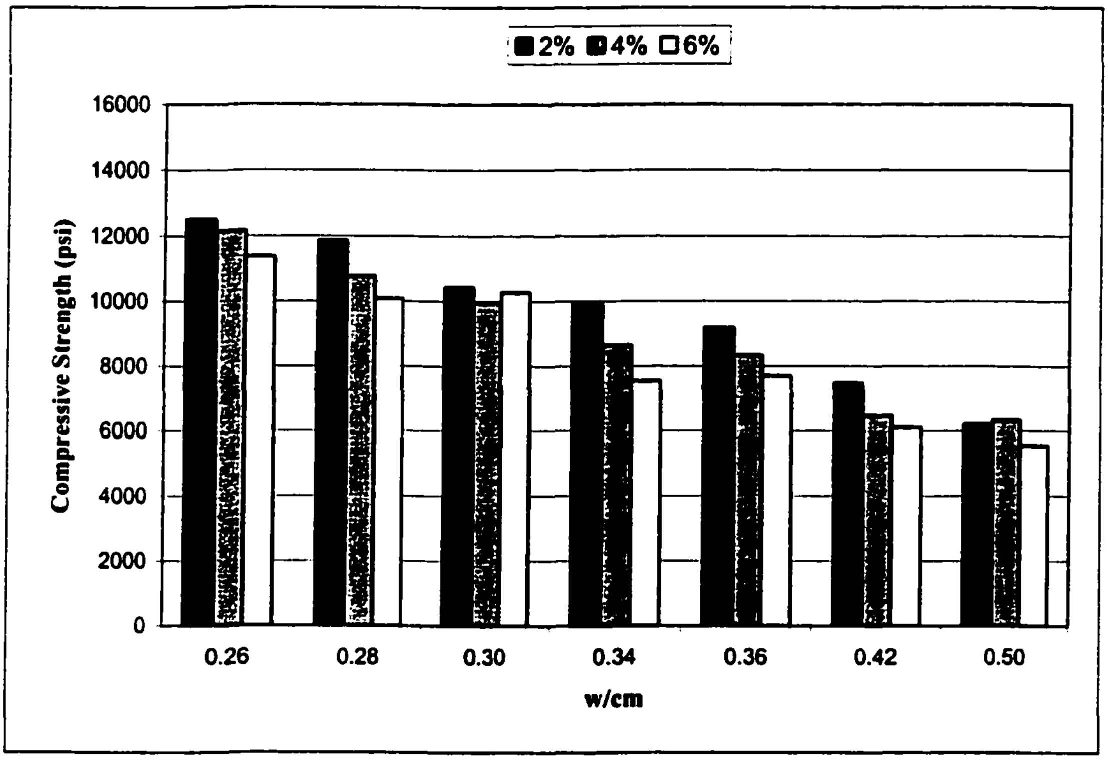

Figure 3.5. Compressive Strength Results at 28 Days.

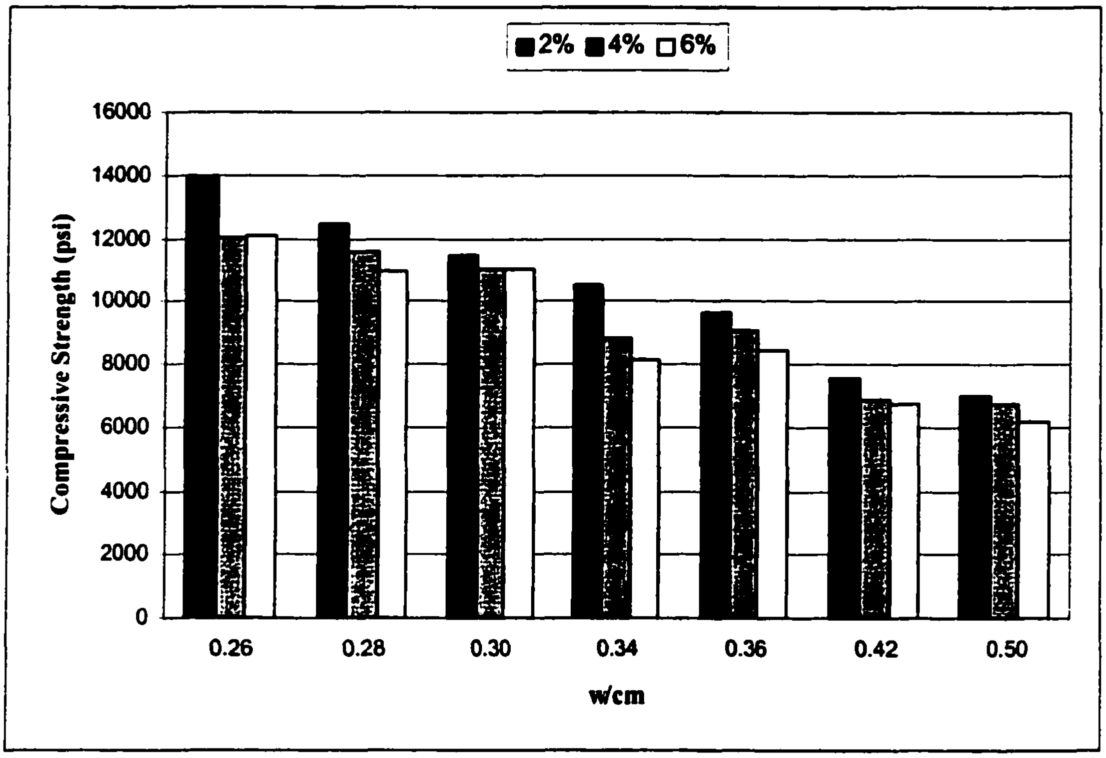

Figure 3.6. Compressive Strength Results at 56 Days.

By 56 days of age, the expected trends become visible. For mixtures of the same w/cm, mixtures without entrained air (two percent air) had the greatest compressive strength. For most mixtures, the mixtures with a nominal air content of two percent had the greater compressive strength and the six percent mixture had the least strength. This trend is validated through examination of the 90 percent confidence intervals (Figure B.3 in Appendix). Figure B.3 shows that there are significant differences between mixtures with nominal air contents of two and six percent for most w/cm, but there is some overlap between either the two and four percent mixtures or the four and six percent mixtures.

Shown in Table 3.16 is the decrease in compressive strength for every one percent increase in entrained air for mixtures with a nominal air content of six percent. At one day of age, half of the mixtures did not follow the " 2 –5 percent decrease in compressive strength for every one percent increase in entrained air" rule of thumb. But, by 28 and 56 days of age, the decrease in compressive strength was well within this range for most mixtures (Figure 3.7). The results further reinforce that the rule of thumb is an adequate predictor in the decreases in compressive strength that result from increases in entrained air.

Table 3.16. Decrease in Compressive Strength (%) per 1% Increase in Entrained Air

w/cm	Concrete Age			
	1 day	7 days	28 days	56 days
0.26	-2.7	3.6	2.2	3.2
0.28	-3.8	2.7	3.9	3.1
0.30	-2.1	-0.2	0.4	1.1
0.34	6.9	8.0	8.3	7.9
0.36	3.7	1.6	4.5	3.4
0.42	3.5	4.6	4.8	2.9
0.50	3.2	1.5	1.9	2.1

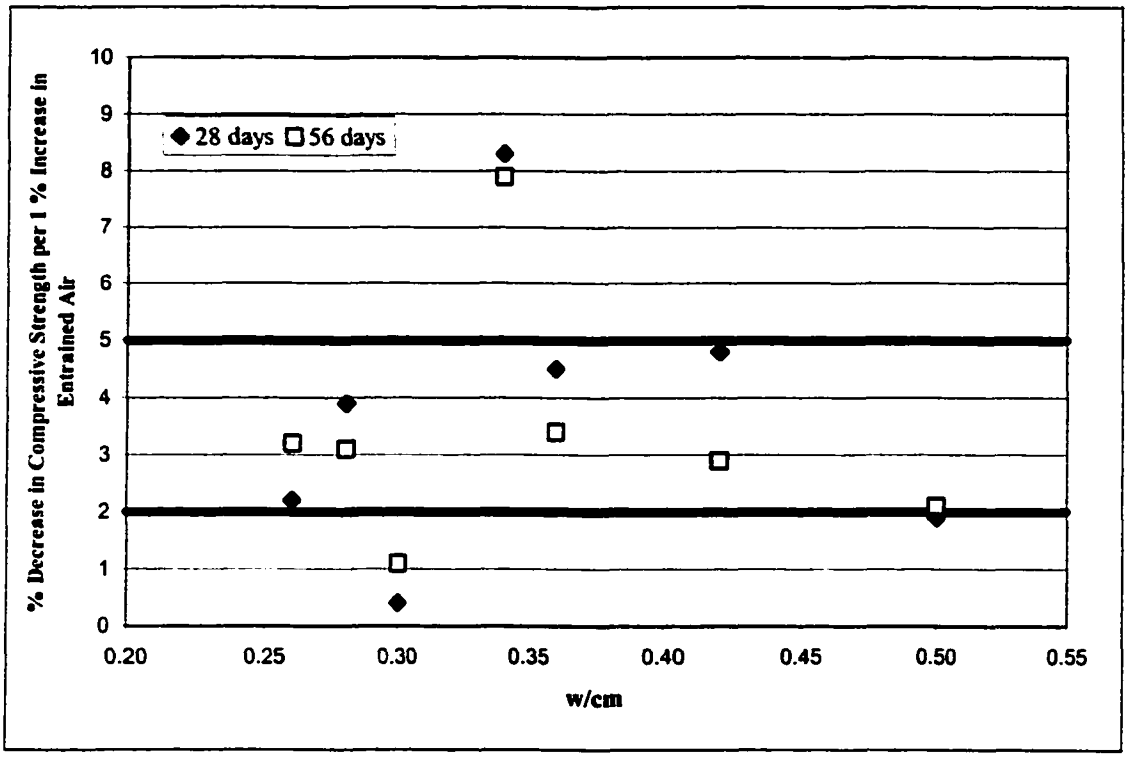

Figure 3.7. Decrease in Compressive Strength per Increase in Entrained Air.

3.6 CONCLUSIONS

This portion of the research focused on the development of HPC with air entrainment using locally available materials. With one of the mixtures developed, two prestressed concrete beams were cast. Once again, not only were the mixtures to have high compressive strength, they must also be cast with local materials and must have enough workability to be placed. Another task of the research was to document the effects of air entrainment of the compressive strength of the concrete. The findings of the research are summarized below.

1. Much higher dosages of AEA are required at w/cm below 0.30 when compared to w/cm above 0.30. At w/cm greater than 0.30, the dosage rates required were within the manufacturer's recommendation. The higher dosage rates were necessary due to insufficient amounts of water in mixtures with w/cm of 0.26 and 0.28. Trial batching is recommended at lower w/cm to determine the optimum dosages of admixtures.
2. For most w/cm, the compressive strength decreased as the total air content increased. By 56 days of age, most mixtures followed the "2 – 5 percent decrease in compressive strength for every 1 percent increase in entrained air" rule of thumb.
3. Air entrained HPC mixtures with adequate workability and high compressive strength can be developed using locally available materials.
4. Based on compressive strength and workability, the results indicated that for the aggregates and cements tested, the optimum cement content was 900 lb/yd^3. The optimum w/cm was 0.26 and the optimum coarse aggregate content was 1790 lb/yd^3. This mix proportion was used to cast the precast/prestressed bridge girders that will be examined in later phases of the research program.

CHAPTER 4

EXAMINING THE FREEZE THAW DURABILITY OF HPC

4.1 INTRODUCTION

This section of the research program investigated the need for air entrainment in high performance concrete (HPC). Entrained air is required to ensure the durability of some concretes that are subjected to freezing and thawing. Entrained air is added to concrete through the use of air entraining agents (AEA). Current building codes require varying amounts of entrained air depending on the severity of the exposure. Entrained air voids provide small air pockets where water can expand when freezing and where water pressure can be relieved. There may be as many as 300 billion entrained air voids in a cubic yard of concrete with a total air content of four to six percent by volume (Kosmatka et al, 1994). The voids generally have a diameter of about 0.002-in. (Neville, 1997) and are evenly distributed throughout the concrete. Entrained air voids are typically spaced within 0.008 in. of each other (Derucher et al, 1994). Without these voids, continuous freeze-thaw cycles will eventually degrade and damage the concrete. A total air content between four and eight percent is generally considered adequate to provide resistance to the freeze/thaw action (Mindess et al, 1981). The Oklahoma Department of Transportation requires a total air content of 5 ± 1.5 percent in bridge girders.

The experimental program was designed to examine two objectives. The first objective was to determine if air entrainment is necessary for freeze-thaw durability in HPC. The second objective was to determine the maximum w/cm where air entrainment is not needed. Concrete mixtures with varying water to cementitious material ratios (w/cm) were subjected to ASTM C 666 (Procedure A). The variables for the mixtures were total air content and w/cm. The targeted total air contents for the mixtures were 2 percent (entrapped air, no air entraining agent added), 4 percent, and 6 percent. The freeze-thaw tests continued until the specimen deteriorated, or the specimen reached 300 freeze thaw cycles. The results from the study show that air entrainment is not required below a specific maximum w/cm for the materials employed.

4.2 LITERATURE REVIEW

4.2.1 Introduction

For the purposes of this study, the researchers defined HPC using ACI's 1999 definition which states that 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." Also, normal strength concrete (NSC) is defined as all other concretes that do not fall within ACI's definition of HPC. The use of HPC in exterior structures has increased in recent years (Neville et al, 1998). Its increased strength and durability make HPC very appealing to the prestressed concrete industry, particularly in bridge girders. Due to its increased strength and durability, HPC can reduce the number of girders, increase bridge spans, decrease bridge depth, and increase bridge durability (B. Russell, 1994). The durability and permeability of NSC has been thoroughly studied over the past century, but the same properties of HPC have not been studied to the same extent. To increase the durability of normal strength concrete, an air-entraining agent is added to the concrete mixture. While the entrained air increases durability, it also decreases concrete strength. For this reason, air entrainment is not commonly used in HPC (Cohen et al, 1992). Many researchers are questioning the need for air entrainment in HPC (Cohen et al, 1992; Hooten et al, 1993; Lessard et al, 1995; Li et al, 1994; Marchand et al, 1995; Mokhtarzadeh et al, 1995; Pigeon et al, 1991). Many of these researchers have concluded that the low permeability of HPC due to its decreased water to cementitious ratio (w/cm) improves its durability (Pigeon et al, 1991; Cohen et al, 1992; Li et al, 1994; Marchand et al, 1995; and Fagerlund, 1994). Accordingly, several researches have developed durable non-air-entrained HPC mixtures (Cohen et al, 1992; Mokhtarzadeh et al, 1995; Fagerlund, 1994; and Hilsdorf, 1994). Despite these indicators, most researchers still recommend the use of air entrainment for durability (Pigeon et al, 1991; Cohen et al, 1992; Marchand et al, 1995; Fagerlund, 1994; Zia et al, 1993; and Aitcin et al, 1993). The objective of this research program is to examine the need of air entrainment in HPC made with materials commonly available.

4.2.2 Freeze-Thaw Mechanism

It is common knowledge that water freezes at 32°F, and when water freezes there is a nine percent increase in volume as water turns to ice. However, water that is trapped within the capillary pores of concrete does not necessarily freeze at 32°F. The temperature at which water freezes in the capillary pores is a function of the size of the pores and pore chemistry. As pore sizes decrease, the temperature required to freeze the water also decreases. For example, in pores with a diameter of 10 nm, the water will not freeze until 23°F, and for pores with a diameter of 3.5 nm, the water will not freeze until -4°F (Mindess et al. 1981).

While the freezing of water damages the concrete paste, it is not the main contributor to the freeze-thaw deterioration of concrete. The major cause of damage is due to the increase in hydraulic pressure within the pore spaces. As water freezes within a capillary pore, the ice that is formed compresses the unfrozen water within the pore. If the water can escape into an occupied void, the hydraulic pressure is relieved. However, if the distance to a void is too great and the hydraulic pressure cannot be relieved, the water pressure will expand the pores causing tensile stresses in the surrounding concrete paste. In saturated concrete, the tensile stresses will eventually exceed the tensile capacity of the paste and cracking will occur. Entrained air is added to the concrete to provide the voids necessary to relieve the hydraulic pressure (Mindess et al, 1981).

4.2.3 Freeze-Thaw Tests Procedure

The American Society for Testing and Materials (ASTM) developed a test procedure for examining the freeze-thaw durability of concrete, ASTM C 666 Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing. ASTM C 666 requires that the specimens are moist cured for 14 days prior to testing. The specimens are then subjected to 300 freeze-thaw cycles at temperatures ranging from 0°F to 40°F. The dynamic modulus is measured prior to testing and during testing. Changes in the dynamic modulus provide a measure of damage to the concrete. Within ASTM C 666 are two different procedures, Procedure A and Procedure B. For Procedure A, the specimens are frozen and thawed in water, and for Procedure B, the specimens are thawed in water and frozen in air. After 300 cycles, the durability factor of the specimens is reported. The durability factor is the percent change in the dynamic modulus of elasticity. For example, a concrete mixture with a durability factor of 60 has a dynamic modulus that is 60 percent of what it was prior to testing. Acceptable freeze-thaw durability is defined as concrete having a durability factor greater than 60 or a spacing factor less than 0.008 inches (Marchand et al, 1995, and Mokhtarzadeh et al, 1995).

Many researchers suggest that ASTM C 666 Procedure A does not adequately simulate field conditions (Aitcin et al, 1993; Cohen et al, 1992; Kucharska, 1994; Lessard et al, 1995; Marchand et al, 1995; Mokhtarzadeh et al, 1995; and Pigeon et al, 1991). Many concrete structures are not subjected to freeze-thaw conditions as early as 14 days after casting nor are the structures completely saturated with water which is required by ASTM C 666. Aitcin et al (1993), Lessard et al (1995), and Pigeon et al (1991) believed that ASTM C 666 would required air entrainment in concrete structures that would otherwise have adequate freeze-thaw durability for outdoor exposures. Another concern is the possible temperature gradient within individual specimens (Kucharska, 1994 and Galeota et al, 1991). Even with these faults, most researchers still recommend the use of the test to examine the freeze-thaw durability of concrete.

4.2.4 Air Content

There are two types of air voids in concrete, entrapped and entrained air voids. Entrapped air voids result from air that is trapped in the concrete during mixing. Entrapped air normally accounts for approximately two percent of the volume of the concrete mixture. Entrained air voids are intentionally placed in the concrete through the use of AEA (Mindess et al, 1981). AEA are chemical admixtures composed of animal or vegetable fats, alkali salts of wood resins, or alkali salts of sulfated and sulfonated organic compounds (Neville, 1997). Entrained air voids have a diameter of approximately 0.002 inches (0.05mm.) and are evenly distributed throughout the concrete mixture, whereas entrapped air voids are much larger (can be up to several millimeters in diameter) and are randomly distributed throughout the concrete (Neville, 1997, and Popovics, 1998).

If exposed to freezing and thawing, normal strength concrete requires a total air content of four to seven percent (Mindess et al, 1981). Of this four to seven percent air, normally two percent is entrapped air and the remaining two to five percent is entrained air. Due to the reduced permeability of HPC, researchers have suggested that the air content of HPC can be reduced (Lessard et al, 1995). Fagerlund (1995) used a closed container model to determine the minimum air contents required for HPC to have acceptable freeze-thaw durability. The model assumed a spherical shape with solid, impermeable materials along the outside of the sphere with the freezable water in the center. From his model, Fagerlund determined that an entrained air content of 0.2 percent would be sufficient in HPC, but in normal strength concrete 7.5 percent would be necessary.

Experimental results have proven that the total air content required for HPC to have acceptable freeze-thaw durability is less than that for normal strength concrete. Acceptable freeze-thaw durability is defined as concrete having a durability factor greater than 60 or a spacing factor less than 0.008 inches (Marchand et al, 1995, and Mokhtarzadeh et al, 1995). Several researchers have developed HPC mixtures with low air contents that have acceptable freeze-thaw durability (Hooten, 1993, Kriesel et al, 1995, Pigeon et al, 1991, and Pinto et al, 2001). However, several researchers have also developed HPC mixtures with low air contents that did not have acceptable freeze-thaw durability (Cohen et al, 1992, Galeota et al, 1991, Hooten et al, 1993, Kriesel et al, 1993, and Li et al, 1994).

4.2.5 Air Void System

In addition to the total air content, the specific surface and spacing factor of air voids are crucial factors that play a major role in the freeze-thaw durability of concrete. The surface area of air voids per unit volume is expressed as specific surface. The specific surface is expressed in terms of square inches per cubic inches. For concrete with acceptable freeze-thaw durability, the specific surface is typically between 400 to 600 in^2/in^3 or in^{-1}. The specific surface for concrete containing only entrapped air is normally less than 300 in^{-1} (Neville, 1997).

The spacing factor is defined as the "average maximum distance from any point in the paste to the edge of a void." A spacing factor of 0.008 inch or less is generally considered adequate to ensure freeze-thaw durability (Aitcin et al, 1994, Attigobe et al, 1992, and Mindess et al, 1981). However, Marchland et al (1995) developed durable, low w/cm concrete mixtures with spacing factors as high as 0.030 inch.

4.2.6 Water to Cementitious Material Ratio

In 1956 Hubert Woods suggested that as an alternative to air entrainment, concrete mixtures could be cast at or below a water to cement ratio (w/c) of 0.40. His reasoning was that at low w/c (below 0.40), well cured concrete mixtures would not contain water "that would freeze under natural conditions." He also suggested that the reduced permeability of low w/c concrete would aid in preventing the natural saturation of the concrete. Woods recommended that experimental testing be conducted to verify this theory (Woods, 1956). Years later, Francis Young and Sidney Mindess (1981) theorized that for completely hydrated cement pastes, air entrainment may not be necessary at w/c equal to or less than 0.36. Cohen et al (1992) theorized that low w/cm concrete containing silica fume is relatively impermeable to water. Because of this impermeability, the concrete would never become saturated in natural, outdoor exposures. Therefore, air entrainment would not be necessary. But if there were a possibility for saturation, air entrainment would still be required. Marchland et al (1995), Hooten (1993), and Pigeon et al (1991) all believed that at low w/cm, the concrete consumes available water during hydration, which reduces the freezable water in concrete and increases its durability.

Several researchers have developed low w/cm mixtures that had acceptable freeze-thaw durability without the aid of air entrainment. Pigeon et al (1991) developed freeze-thaw durable mixtures at w/cm of 0.30 at lower. With the use of silica fume, Hooten (1993) achieved good durability at w/cm of 0.35 and lower. Pinto and Hover (2001) achieved durability factors of 101 at w/cm of 0.35 using a modified version of ASTM C 666 (14 day drying period before testing). Mokhtarzadeh et al (1995) developed durable mixtures with w/cm of 0.29 and 0.31. Whereas Li et al (1994) had acceptable durability at w/cm of 0.24 and lower.

However, for each of the durable non-air entrained mixtures developed by the previous researchers, there are similar mixtures that did not have acceptable durability. Kriesel et al (1995) and Cohen et al (1992) examined concrete mixtures with a w/cm of 0.30 that had poor durability. Li et al (1994) tested mixtures with w/cm of 0.24 to 0.33 that were not durable. Without the aid of silica fume, concrete mixtures investigated by Hooten (1993) were not durable at a w/cm of 0.35. From these results, one cannot exclusively determine a w/cm where air entrainment is not required. Many other factors such as aggregate type, cement type, and curing conditions of the concrete also affect the durability of concrete (Mokhtarzadeh et al, 1995).

4.2.7 Aggregate Type

The type of aggregate plays an important role in the freeze-thaw durability of concrete. Certain types of aggregates are susceptible to freeze-thaw damage. The porosity of the coarse aggregates is vital to the concrete's durability. Some limestones and granites have little porosity and a low permeability. However due to their low permeability, these aggregates are not normally saturated in concrete and contain voids (since they seldom become saturated) where hydraulic pressure can be relieved. On the other hand, sandstones have a high permeability, but are very porous and may be prone to saturation. Their high porosity allows water to escape during freezing which reduces the hydraulic pressure. These aggregates are not usually susceptible to freeze-thaw damage. Problems arise when the aggregates have high porosity and low permeability. The high porosity is achieved by extremely fine pores within the aggregates. If these aggregates become saturated, the distance the water must flow to relieve the hydraulic pressure is too great. The water then freezes within the aggregate and fracture of the aggregate occurs. Aggregates that are most susceptible to freeze-thaw damage are fine-grained rocks, such as cherts or shales (Mindess et al, 1981).

Researchers have investigated the freeze-thaw durability of concrete mixtures containing various types of coarse aggregates. Pigeon et al (1991) found acceptable durability with concrete mixtures containing either limestone or granite. Aitcin et al (1994) had acceptable durability with concrete containing either crushed diorite or crushed granite. Kriesel et al (1996) and Mokhtarzadeh et al (1995) determined that concrete containing low absorption limestone performed the best. Both researchers determined that round gravel, partially crushed granite, and crushed granite were unacceptable. Even though some researchers found that limestone was the best performing aggregate, Cohen et al (1992) and Li et al (1994) had poor results with limestone.

The freeze-thaw durability of aggregates can be investigated by two main methods. The aggregates can be tested indirectly using ASTM C 666 (tests in concrete) or tested directly using ASTM C 682, Standard Practice for Evaluation of Frost Resistance of Coarse Aggregate in Air Entrained Concrete by Critical Dilation Process.

The results from the previous researchers suggest that durability of the aggregate should be known before its use in a harsh environment.

The size of the aggregate also plays a role in the required air content. Depending on the nominal maximum aggregate (NMA) size and environmental conditions, the required total air contents range from 3.5 to 7.5 percent (ACI 318-99). This is because less paste is required for concrete containing larger aggregates to be workable (Mindess and Young, 1981). For example, concretein severe exposure containing coarse aggregate with a NMA size of 3 in. requires a total air content of 4.5 percent, but concrete containing 3/8 in. NMA size requires 7.5 percent.

4.2.8 Curing

The age, curing temperature, and curing regimen all affect the freeze-thaw durability of concrete. Water within the capillary pores freeze at lower temperatures as the concrete ages. As the concrete cures, the concentration of salts within the pores increases, which lowers the freezing temperature of the solution. For example, the temperature must be lower than 30°F in the capillary pores for water to freeze at 3 days of age but at 28 days of age the freezing temperature decreases to 23°F (Neville, 1997).

Not only does the length of curing affect the freeze-thaw durability, the temperature of the curing regimen can also have an effect. In the prestressed concrete industry, steam curing is used to increase the one-day compressive strength. The steam curing increases the initial hydration. This results in a "less uniform distribution of hydration products within the paste" which alters the pore size distribution (Mindess et al, 1981). The change in pore distribution may result in a slightly higher permeability. The increase in permeability allows more water into the concrete making the specimen more susceptible to freeze-thaw damage.

Kriesel et al (1996) and Mokhtarzadeh et al (1995) determined that the curing regimen was an important factor affecting the freeze-thaw durability of concrete. Kriesel et al (1996) determined that moist cured specimens had higher durability factors than heat cured specimens. They also found that moist cured specimens absorbed less water than heat-cured specimens which indicates reduced permeability for moist cured specimens.

4.2.9 Conclusions

As the Literature Review reveals, many factors affect the freeze-thaw durability of concrete. The w/cm, air content, aggregate type, and curing regimens are some of the factors that affect the durability of concrete. While many researchers developed durable non-air entrained mixtures, other researchers were unable to produce similar results. For now, most researchers still recommend the use of air entrainment in HPC subjected to a harsh environment. The use of air entrainment can assure the freeze-thaw durability of HPC. Further research needs to be conducted on the durability of non-air entrained concrete.

4.3 EXPERIMENTAL PROCEDURES

4.3.1 Scope

The experimental program was designed to examine two objectives. The first objective was to determine if air entrainment is necessary for freeze-thaw durability in

HPC. The second objective is to determine the maximum w/cm where air entrainment is not needed. Several different concrete mixtures with varying w/cm and varying total air contents were subjected to the freeze-thaw durability test (ASTM C 666, Procedure A). The testing matrix is shown in Table 4.1. The w/cm ranged from 0.26 to 0.50. The targeted total air contents were 2, 4, and 6 percent.

Table 4.1. Testing Matrix

Target Total Air Content (%)	w/cm							
	0.26	0.30	0.32	0.34	0.36	0.42	0.45	0.50
2	X	X	-	X	X	X	-	X
4	X	X	X	-	X	X	X	X
6	X	X	-	-	X	X	-	X
X = indicates batching and testing of these variables								

4.3.2 Materials

Ash Grove Type III Cement was used in all mixtures. The coarse aggregate was 3/8 inch (ASTM # 8), crushed limestone from Davis, Oklahoma. The fine aggregate was washed river sand from Dover, Oklahoma. The material properties of the cement and aggregates are listed in Chapter 2. To provide adequate workability a water reducer (WR) and/or a high range water reducer (HRWR) was used. The WR used was WRDA with Hycol, and the HRWR used was ADVA Flow. An air entraining agent (AEA), DARAVAIR 1000, was used to attain the required target total air contents.

4.3.3 Mixtures

All mixtures contained 900 lb/yd^3 of cement and 1790 lb/yd^3 of coarse aggregate. The w/cm were 0.26, 0.30, 0.32, 0.34, 0.36, 0.42, 0.45, and 0.50. The quantity of fine aggregate was dependent on the w/cm and total air content. Mix proportioning was done by the Absolute Volume Method. In this method sand is used to replace the same volume whenever water and air are reduced. For example, as the w/cm increased, the quantity of fine aggregate decreased. Likewise, as the air contents increased for mixtures with the same w/cm, the quantity of sand decreased. The mixture proportions are shown in Tables 4.2 through Table 4.6.

Mixture designations, or batch numbers, were determined by three properties. The first number in the designation is the quantity of cement in each mixture. Since all the mixture contained 900 lb/yd^3, the first number for all the mixtures is nine. The second number is the w/cm of the mixture. The third number is the measured total air content for the mixture. For example, mixture 9-26-2.4 contained 900 lb/yd^3 of cement. The mixture also had a w/cm of 0.26 and a measured total air content of 2.4 percent.

Table 4.2 Mixture Proportions.

Materials	Mixture Designations				
	9-26-2.4	9-26-3.8	9-26-5.6	9-30-1.1	9-30-4.5
Cement (lb/yd^3)	900	900	900	900	900
Coarse Agg. (lb/yd^3)	1790	1790	1790	1790	1790
Fine Agg. (lb/yd^3)	1217	1128	1040	1122	1034
Water (lb/yd^3)	234	234	234	270	270
HRWR (fl oz/cwt)	15	15	15	10	10
WR (fl oz/cwt)	3	3	3	3	3
AEA (fl oz/cwt)	0	10	15	0	0.25
Targeted Air Content (%)	2	4	6	2	4
Calculated Unit Weight (lb/yd^3)	153.4	150.1	146.8	151.2	147.9
w/cm	0.26	0.26	0.26	0.30	0.30

Table 4.3. Mixture Proportions.

Materials	Mixture Designations				
	9-30-5.7	9-32-4.1	9-34-2.0	9-36-2.6	9-36-3.8
Cement (lb/yd^3)	900	900	900	900	900
Coarse Agg. (lb/yd^3)	1790	1790	1790	1790	1790
Fine Agg. (lb/yd^3)	945	986	1028	980	892
Water (lb/yd^3)	270	288	306	324	324
HRWR (fl oz/cwt)	10	10	8	2.5	3
WR (fl oz/cwt)	3	3	3	3	3
AEA (fl oz/cwt)	1	0.25	0	0	1.0
Targeted Air Content (%)	6	4	2	2	4
Calculated Unit Weight (lb/yd^3)	144.6	146.8	149.0	147.9	144.7
w/cm	0.30	0.32	0.34	0.36	0.36

Table 4.4. Mixture Proportions.

Materials	Mixture Designations			
	9-36-6.2	9-42-1.8	9-42-2.2FA	9-42-2.4SF
Cement (lb/yd^3)	900	900	765	855
Fly Ash (lb/yd^3)	0	0	135	0
Silica Fume (lb/yd^3)	0	0	0	45
Coarse Agg. (lb/yd^3)	1790	1790	1790	1790
Fine Agg. (lb/yd^3)	803	839	817	822
Water (lb/yd^3)	324	378	378	378
HRWR (fl oz/cwt)	3	0	0	0
WR (fl oz/cwt)	3	0	0	3
AEA (fl oz/cwt)	1.5	0	0	0
Targeted Air Content (%)	6	2	2	2
Calculated Unit Weight (lb/yd^3)	141.5	144.7	143.9	142.4
w/cm	0.36	0.42	0.42	0.42

Table 4.5. Mixture Proportions.

Materials	Mixture Designations			
	9-42-4.4	9-42-5.9	9-45-4.1	9-50-0.9
Cement (lb/yd^3)	900	900	900	900
Coarse Agg. (lb/yd^3)	1790	1790	1790	1790
Fine Agg. (lb/yd^3)	750	662	679	650
Water (lb/yd^3)	378	378	405	450
HRWR (fl oz/cwt)	0	0	0	0
WR (fl oz/cwt)	0	0	0	0
AEA (fl oz/cwt)	1	1.25	0.75	0
Targeted Air Content (%)	4	6	4	2
Calculated Unit Weight (lb/yd^3)	141.4	138.1	139.8	140.4
w/cm	0.42	0.42	0.45	0.50

Table 4.6. Mixture Proportions.

Materials	Mixture Designations			
	9-50-1.5FA	9-50-2.2SF	9-50-3.6	9-50-6.6
Cement (lb/yd^3)	765	855	900	900
Fly Ash (lb/yd^3)	135	0	0	0
Silica Fume (lb/yd^3)	0	45	0	0
Coarse Agg. (lb/yd^3)	1790	1790	1790	1790
Fine Agg. (lb/yd^3)	628	633	561	473
Water (lb/yd^3)	450	450	450	450
HRWR (fl oz/cwt)	0	0	0	0
WR (fl oz/cwt)	0	0	0	0
AEA (fl oz/cwt)	0	0	0.5	0.75
Targeted Air Content (%)	2	2	4	6
Calculated Unit Weight (lb/yd^3)	139.6	138.1	137.1	133.8
w/cm	0.50	0.50	0.50	0.50

4.3.4 Tests

All mixtures were subjected to several tests. The fresh concrete properties tested were slump (ASTM C 143), unit weight (ASTM C 138), and air content (ASTM C 231). The hardened properties tested were compressive strength (ASTM C 39) at one, seven, twenty-eight, and fifty-six days of age and freeze-thaw durability (ASTM C 666, Procedure A).

The only deviation from the ASTM test procedures occurred in the freeze-thaw tests. ASTM C 666 states that the specimens be submerged in water for 14 days and then placed in the freeze-thaw chamber. For this program, the specimens were submerged in water for 56 days and then placed in the chamber. The additional 42 days allowed the concrete mixtures to attain higher compressive strengths and greater maturity.

4.4 RESULTS

4.4.1 Fresh Properties

The results of the fresh property tests are shown in Table 4.7. The air contents for all the mixtures were near the targeted total air contents. Not all the mixtures that were cast fell into one of the targeted zones. Several mixtures had to be discarded because the total air contents were much higher than the targeted air content. If the measured total air content was outside this range (± 0.5 percent), the concrete was discarded and no tests were performed.

Table 4.7. Fresh Concrete Properties

Mixture	Slump (in.)	Total Air Content (%)	Unit Weight (lb/ft^3)
9-26-2.4	6.50	2.4	152.3
9-26-3.8	8.00	3.8	149.3
9-26-5.6	4.25	5.6	147.2
9-30-1.1	10.00	1.1	150.9
9-30-4.5	1.50	4.5	147.5
9-30-5.7	10.75	5.7	146.2
9-32-4.1	6.50	4.1	147.2
9-34-2.0	11.50	2.0	150.6
9-36-2.6	2.50	2.6	147.8
9-36-3.8	3.50	3.8	146.8
9-36-6.2	2.00	6.2	139.6
9-42-1.8	3.25	1.8	145.8
9-42-2.2FA	2.00	2.2	145.9
9-42-2.4SF	1.00	2.4	143.8
9-42-4.4	3.75	4.4	142.7
9-42-5.9	4.00	5.9	139.1
9-45-4.1	5.50	4.1	140.3
9-50-0.9	6.25	0.9	143.4
9-50-1.5FA	8.00	1.5	142.3
9-50-2.2SF	3.25	2.2	140.4
9-50-3.6	9.50	3.6	135.1
9-50-6.6	10.00	6.6	133.4

25.4 mm = 1 inch
16.01 kg/m^3 = 1 lb/ft^3

4.4.2 Hardened Properties

The results of the hardened property tests are shown in Tables 4.8 and 4.9. The results from the compressive strength tests are shown in Table 4.8. Shown in Table 4.9 is the durability factor (DF) of the specimens at 300 freeze-thaw cycles. The compressive strength results are the average of three compressive strength tests, and the durability factors are the average of four specimens. The individual results and statistical data for the compressive strength tests and durability factors are shown in Appendix C.

Table 4.8. Compressive Strength Results.

Mixtures	Average Compressive Strength (psi)			
	1 day[1]	7 days[1]	28 days[1]	56 days[1]
9-26-2.4	6830	11,040	12,190	12,960
9-26-3.8	4830	10,440	11,270	12,090
9-26-5.6	4560	9270	10,560	11,290
9-30-1.1	6150	10,050	12,190	13,600
9-30-4.5	5610	8650	9960	11,010
9-30-5.7	6000	9130	10,270	11,030
9-32-4.1	5730	8320	9650	10,700
9-34-2.0	4700	9080	10,250	11,200
9-36-2.6	4220	7210	9180	9620
9-36-3.8	4580	6960	8340	9040
9-36-6.2	3660	6790	7700	8450
9-42-1.8	2690	6760	8690	9050
9-42-2.2FA	2830	6840	8680	9100
9-42-2.4SF	4070	6290	7770	8240
9-42-4.4	2750	4940	6470	6860
9-42-5.9	2640	4900	6110	6710
9-45-4.1	2600	4900	6110	6980
9-50-0.9	2020	4730	6220	6990
9-50-1.5FA	1970	6020	7360	7870
9-50-2.2SF	2110	4390	6100	6380
9-50-3.6	1560	5220	6340	6760
9-50-6.6	1650	4320	5560	6150

1) Average compressive strength of three tests.
2) Individual tests and statistical data are shown in Appendix C.

Table 4.9. Freeze-Thaw Results.

Mixture	Freeze-Thaw	
	Number of Cycles	Average Durability Factor[1]
9-26-2.4	300	97.5
9-26-3.8	300	97.7
9-26-5.6	300	99.3
9-30-1.1	300	100.6
9-30-4.5	300	98.9
9-30-5.7	300	95.6
9-32-4.1	300	94.4
9-34-2.0	300	94.2
9-36-2.6	300	77.6
9-36-3.8	300	94.4
9-36-6.2	300	100.9
9-42-1.8	300	20.8
9-42-2.2FA	300	30.5
9-42-2.4SF	300	13.4
9-42-4.4	300	100.2
9-42-5.9	300	93.2
9-45-4.1	300	91.3
9-50-0.9	141	17.3
9-50-1.5FA	300	21.5
9-50-2.2SF	300	22.4
9-50-3.6	300	91.3
9-50-6.6	300	93.1

1) Average durability factor of four tests.
2) Individual tests and statistical data are shown in Appendix C.

4.5 DISCUSSION OF RESULTS

4.5.1 Fresh Properties

The results from the total air content test and the unit weight test directly correlated. While this result is expected, the literature does not contain a lot of information on this subject. Increases in total air content resulted in decreases in unit weight for mixtures of like w/cm, which was expected. For mixtures with the same w/cm, the unit weight decreased as the total air content increased, which was expected (Figure 4.1). Also, the unit weights of mixtures with like total air contents decreased as the w/cm increased, which was also expected (Figure 4.2).

The unit weight test served as a check for the total air content. The check was necessary to be certain that the air meter was working correctly and as an independent quality control technique. Shown in Table 4.10 is a comparison of the calculated unit weights and the measured unit weights of the mixtures. Also in Table 4.10 is the calculated unit weight based on the measured air content. The differences between the two air contents are also shown. The average differences between the measured and the calculated unit weights (for the targeted and actual air contents) were +0.67 lb.ft^3 and +0.61 lb/ft^3, respectively. The standard deviation for both differences was 1.36 lb/ft^3. Overall, the differences between the measured and calculated unit weights were not significant, and the minimal differences show that no errors occurred during the batching of the concrete mixture. The minimal differences also show that the unit weight test would be a good quality control tool to use on the job site.

Table 4.10. Comparison of Calculated and Measured Unit Weights.

Mixture Designation	Unit Weights (lb/ft^3)				
	Calculated based (2, 4, or 6 %)	Measured	Difference	Calculated based on actual air content	Difference
9-26-2.4	153.4	152.3	-1.1	152.7	-0.4
9-26-3.8	150.1	149.3	-0.8	150.4	-1.1
9-26-5.6	146.8	147.2	+0.4	147.5	-0.3
9-30-1.1	151.2	150.9	-0.3	152.7	-1.8
9-30-4.5	147.9	147.5	-0.4	147.1	+0.4
9-30-5.7	144.6	146.2	+1.6	145.1	+1.1
9-32-4.1	146.8	147.2	+0.4	146.7	+0.5
9-34-2.0	149.0	150.6	+1.6	149.0	+1.6
9-36-2.6	147.9	147.8	-0.1	147.0	+0.8
9-36-3.8	144.7	146.8	+2.1	145.0	+1.8
9-36-6.2	141.2	139.6	-1.6	141.1	-1.5
9-42-1.8	144.7	145.8	+1.1	145.0	+0.8
9-42-2.2FA	143.9	145.9	+2.0	143.6	+2.3
9-42-2.4SF	142.4	143.8	+1.4	141.8	+2.0
9-42-4.4	141.4	142.7	+1.3	140.8	+1.9
9-42-5.9	138.1	139.1	+1.0	138.3	+0.8
9-45-4.1	139.8	140.3	+0.5	139.6	+0.7
9-50-0.9	140.4	143.4	+3.0	142.2	+1.2
9-50-1.5FA	139.6	142.3	+2.7	140.3	+2.0
9-50-2.2SF	138.1	140.4	+2.3	137.8	+2.6
9-50-3.6	137.1	135.1	-2.0	137.7	-2.6
9-50-6.6	133.8	133.4	-0.4	132.8	+0.6
	Average Differences (lb/ft^3)		+0.67		+0.61
	Standard Deviation (lb/ft^3)		1.36		1.36

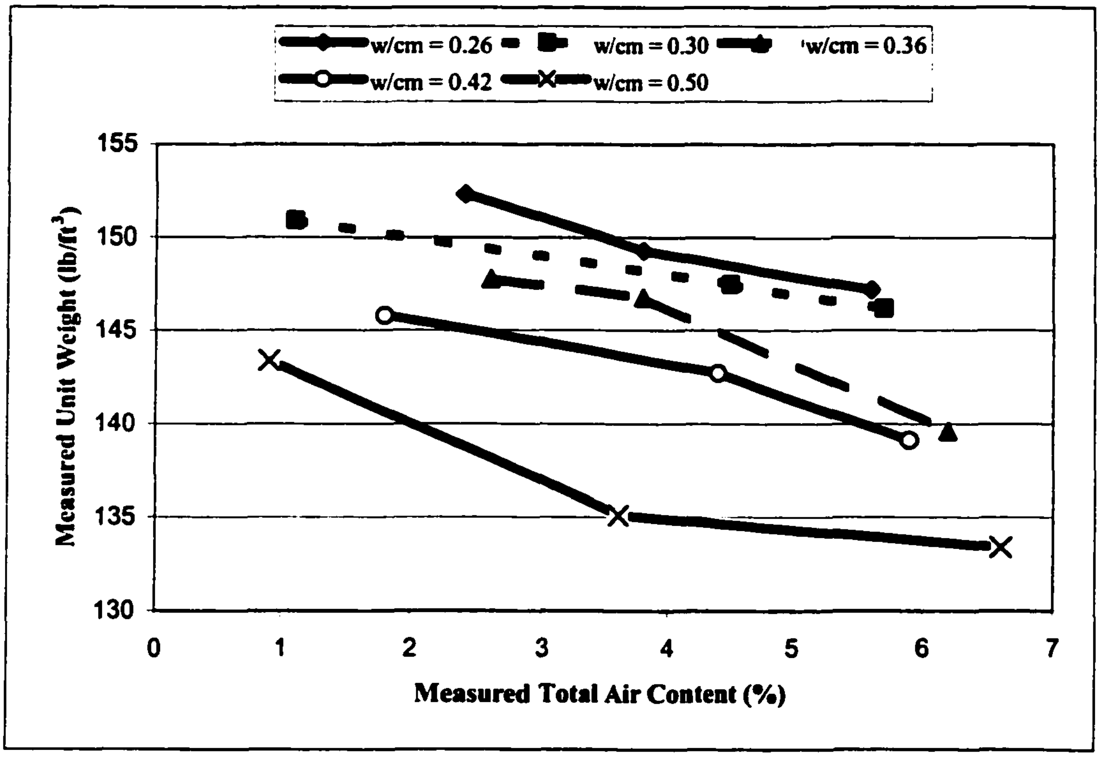

Figure 4.1. Measured Unit Weight versus Measured Air Content.

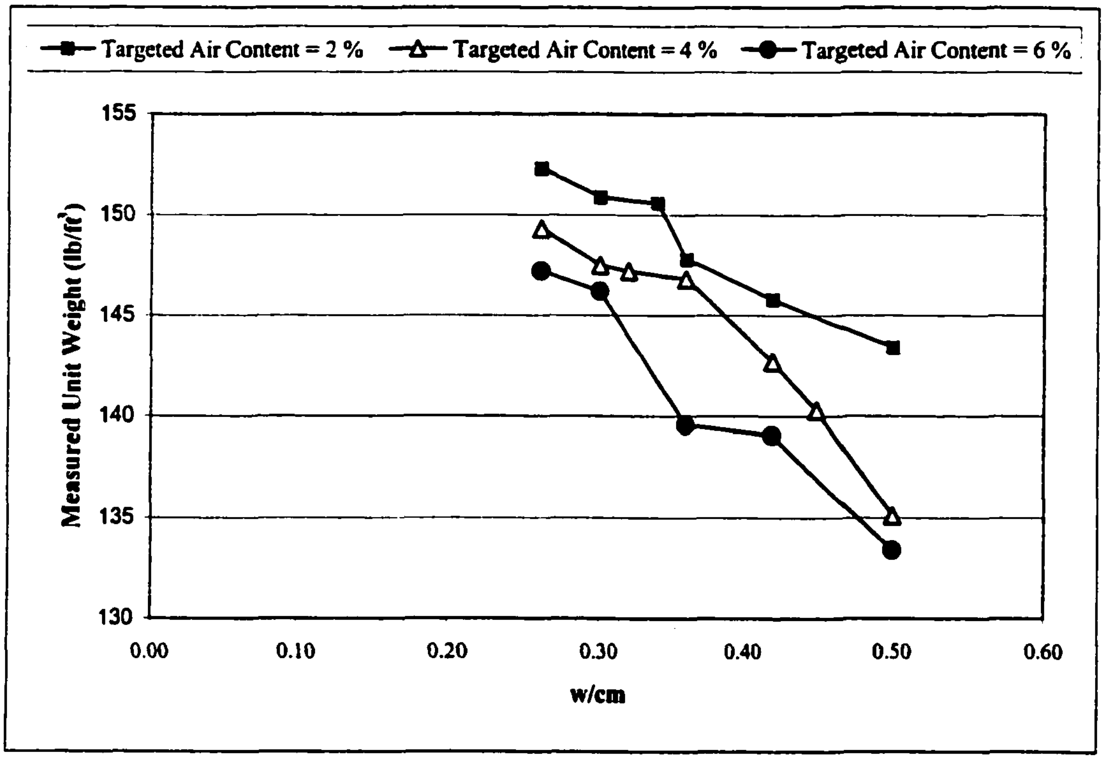

Figure 4.2. Measured Unit Weight versus w/cm.

4.5.2 Hardened Properties

4.5.2.1 Compressive Strength. The compressive strength tests for the mixtures followed an expected trend. As the w/cm decreased, the compressive strengths increased. These trends were validated through examination of the 90 percent confidence intervals shown in Figure C.4 in Appendix C. Also, for mixtures with like w/cm, the increase in total air content resulted in a decrease in compressive strength (Figure 4.3). In Chapter 3, the decrease in compressive strength per increase in air content was examined for the mixtures. The decreases in compressive strength generally followed the "2 – 5 percent decrease in compressive strength for every 1 percent increase in total air content" rule of thumb. The effects of air entrainment on the mixtures were discussed in more detail in Chapter 3.

4.5.2.2 Freeze-Thaw Durability. The results from the freeze-thaw tests indicate that air entrainment may not be needed in concrete mixtures with a w/cm less than 0.36. Also, the results show that between a w/cm of 0.36 and 0.50 only four percent air is necessary for freeze-thaw durability. The durability factors are shown in Figure 4.4. From Figure 4.4, it appears that mixtures with a w/cm less than 0.36 do not require air entrainment. Pinto and Hover (2001) also found acceptable durability at w/cm less than 0.35 for non-air entrained mixtures. At w/cm greater than 0.35, they achieved durability factors greater than 98 with a nominal air content of 4 percent. The results of the freeze-thaw tests are further discussed in the following sections.

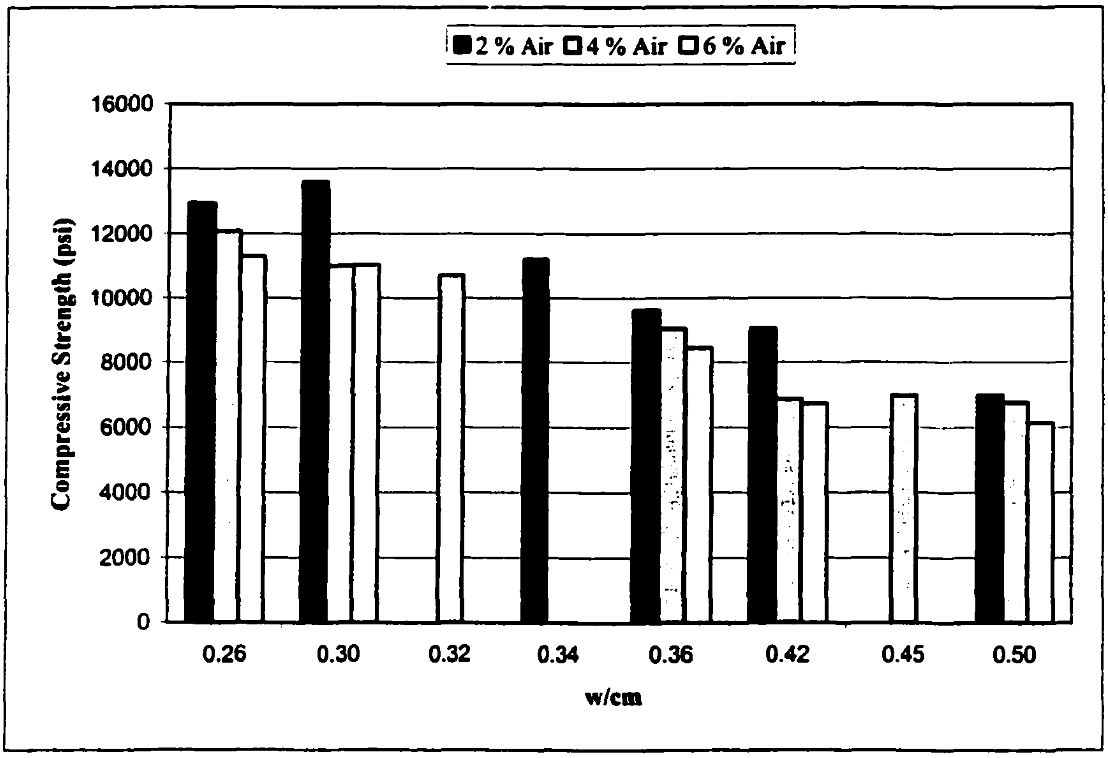

Figure 4.3. 56-Day Compressive Strength versus w/cm.

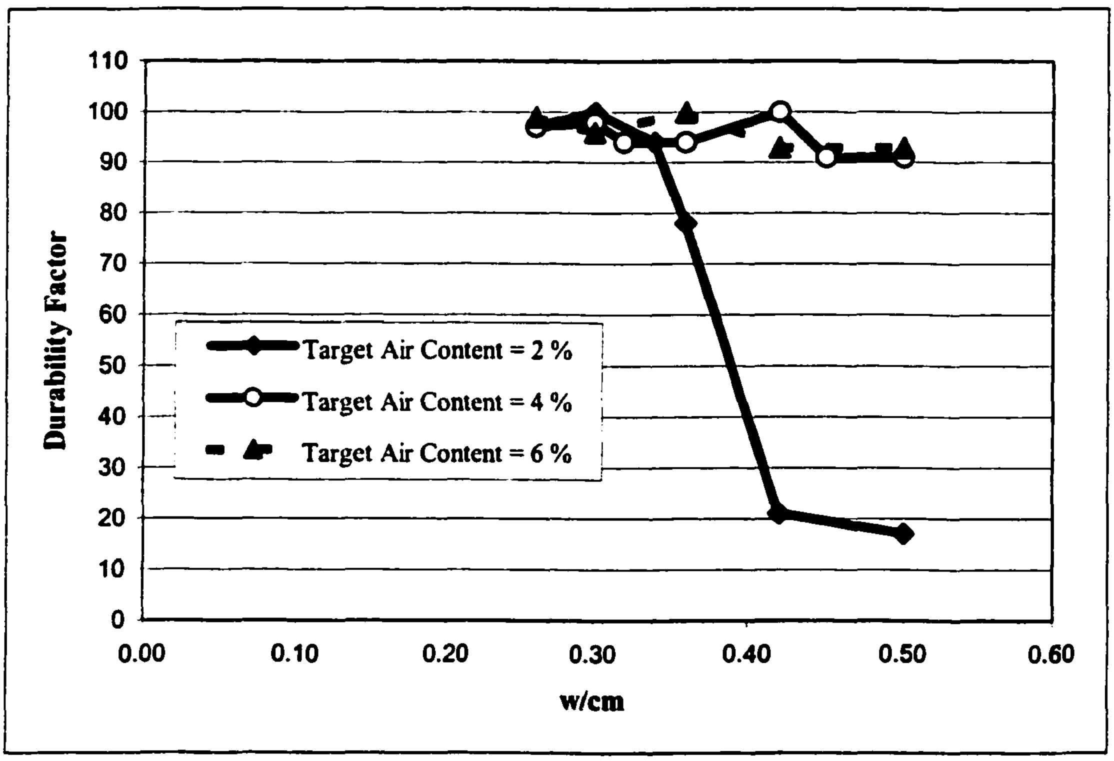

Figure 4.4. Durability Factors versus w/cm.

The mixtures that had unacceptable durability (unacceptable durability is defined as a durability factor below 60 by ASTM C 666) were the non-air entrained mixtures at w/cm of 0.42 and 0.50. Specimens from these mixtures are shown in Figures 4.5 through 4.10. Mixture 9-50-0.9, which had a total air content of 0.9, had deteriorated by 141 freeze-thaw cycles as shown in Figure 4.5. Also, mixture 9-42-1.8 (Figure 4.8), which had a total air content of 1.8 percent, had a durability factor of 21. By 300 freeze-thaw cycles all four specimens had fractured into two pieces.

In an attempt to improve their durability, fly ash or silica fume was added to the mixtures. At each w/cm, 15 percent of the cement was replaced with fly ash (Mixtures 9-42-2.2FA and 9-50-1.5FA), or 5 percent of the cement was replaced with silica fume (Mixtures 9-42-2.4SF and 9-50-2.2SF). Results show that the addition of silica fume and fly ash did not improve the mixtures' durability (Table 4.9). At a w/cm of 0.50, both mixtures (9-50-1.5FA and 9-50-2.2SF) had durability factors of 22 compared to the durability factor of 17 for the mixtures without pozzolans. However, there was a difference in the appearance of the specimens. Mixture 9-50-0.9 had deteriorated by 141 freeze-thaw cycles (Figure 4.5), but with the addition of fly ash and silica fume, the specimens did not completely deteriorate as evident in Figures 4.6 and 4.7.

Fly ash and silica fume were also added to the mixtures with a w/cm of 0.42 (Mixtures 9-42-2.2FA and 9-42-2.2SF). As with the mixtures with a w/cm of 0.50, the pozzolans did not significantly increase the concrete's freeze-thaw durability (Figures 4.9 and 4.10). Without pozzolans, Mixture 9-42-1.8 had a durability factor of 21. The addition of fly ash increased the durability factor to 31, and silica fume reduced the durability factor to 13.

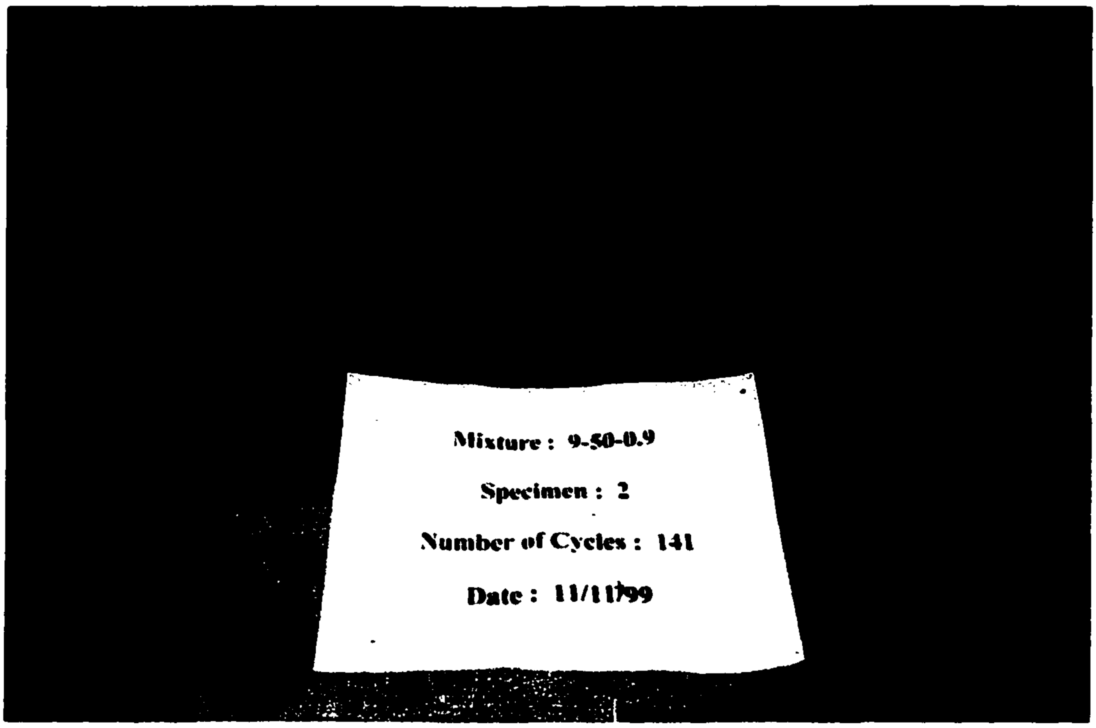

Figure 4.5. Mixture 9-50-0.9 with a Durability Factor of 17.

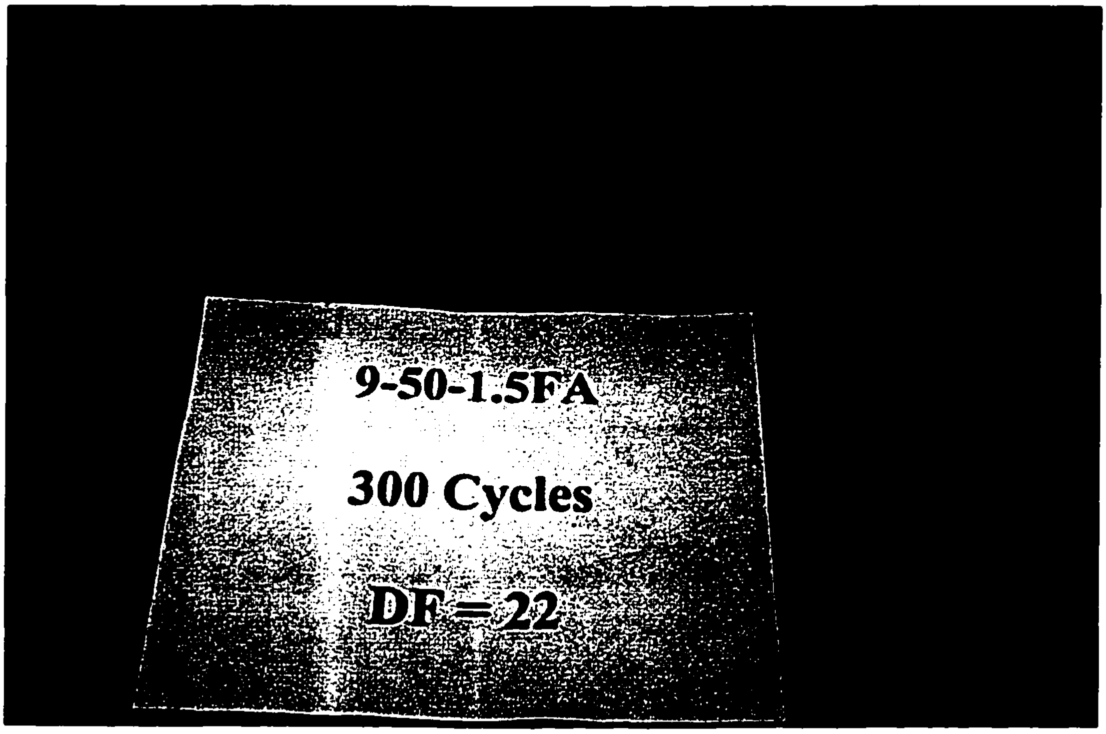

Figure 4.6. Mixture 9-50-1.5FA with a Durability Factor of 22.

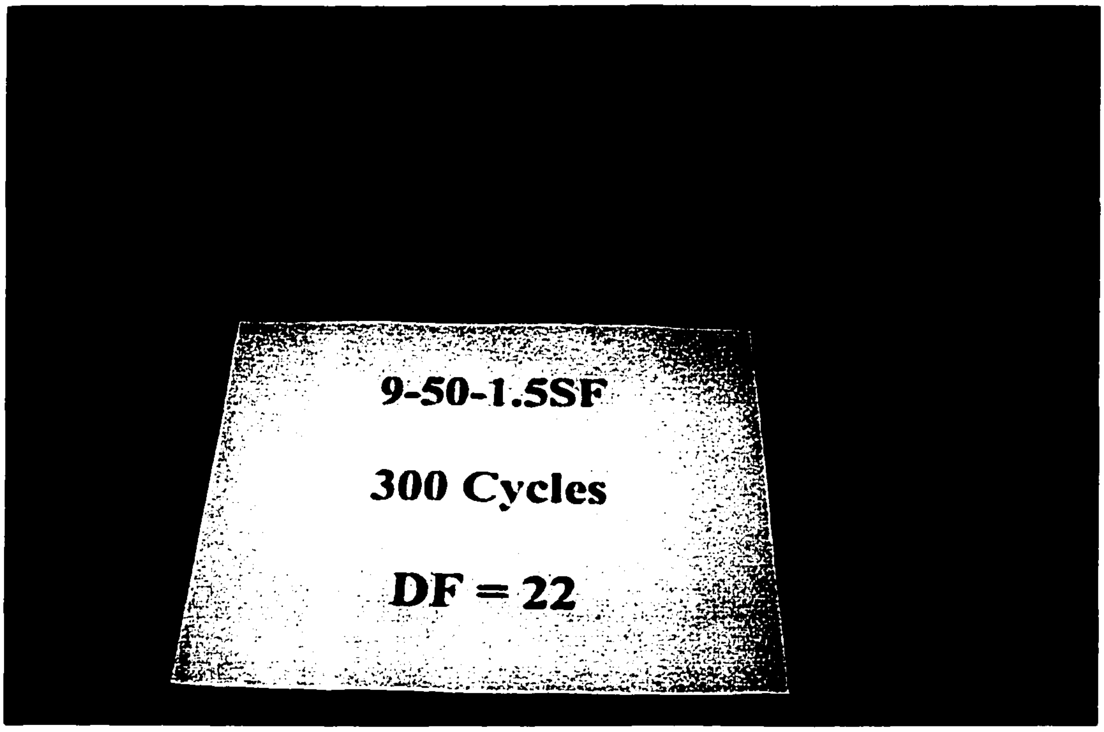

Figure 4.7. Mixture 9-50-1.5SF with a Durability Factor of 22.

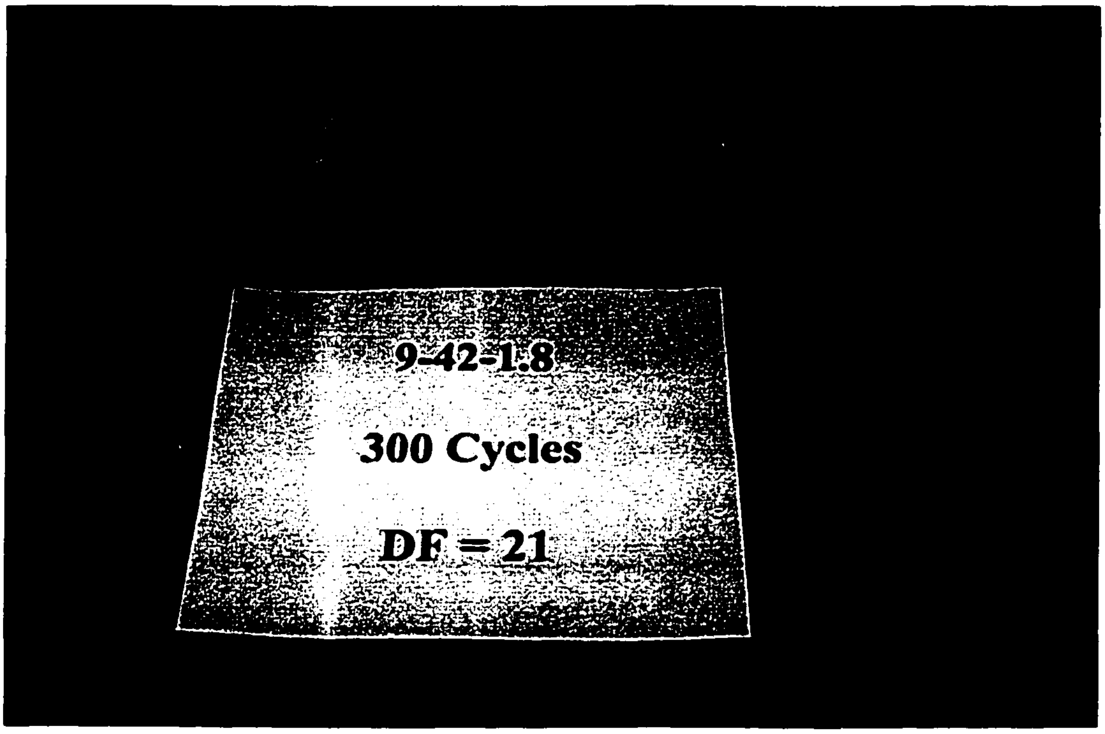

Figure 4.8. Mixture 9-42-1.8 with a Durability Factor of 21.

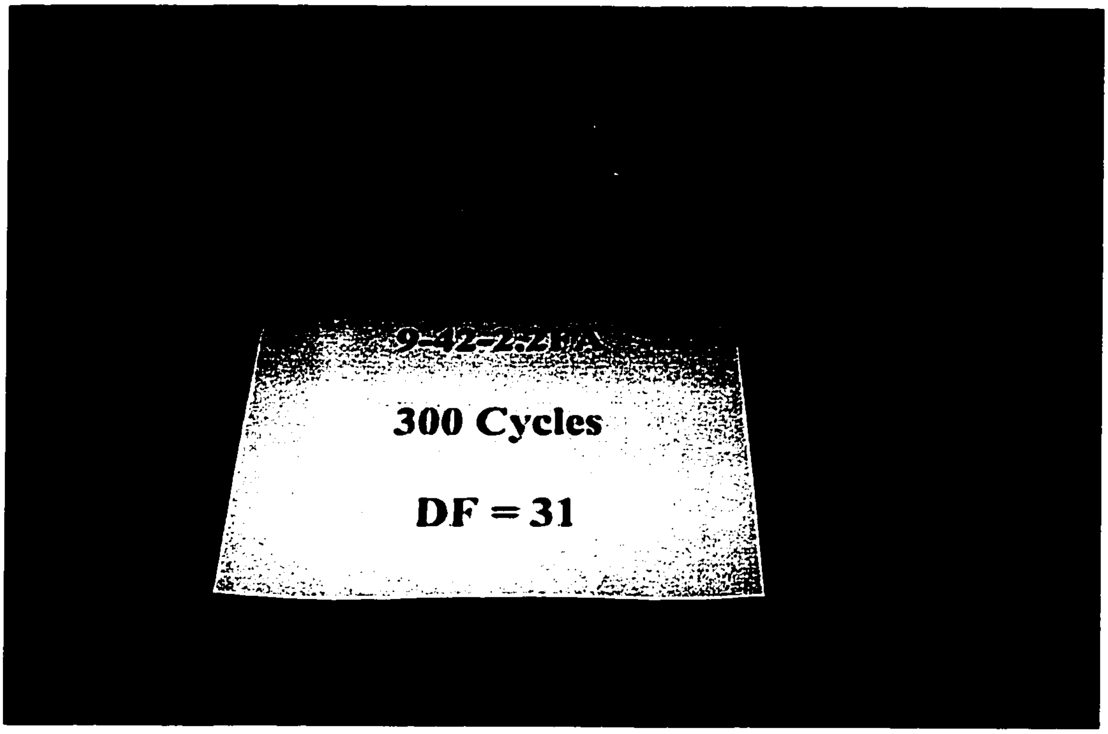

Figure 4.9. Mixture 9-42-2.2FA with a Durability Factor of 31.

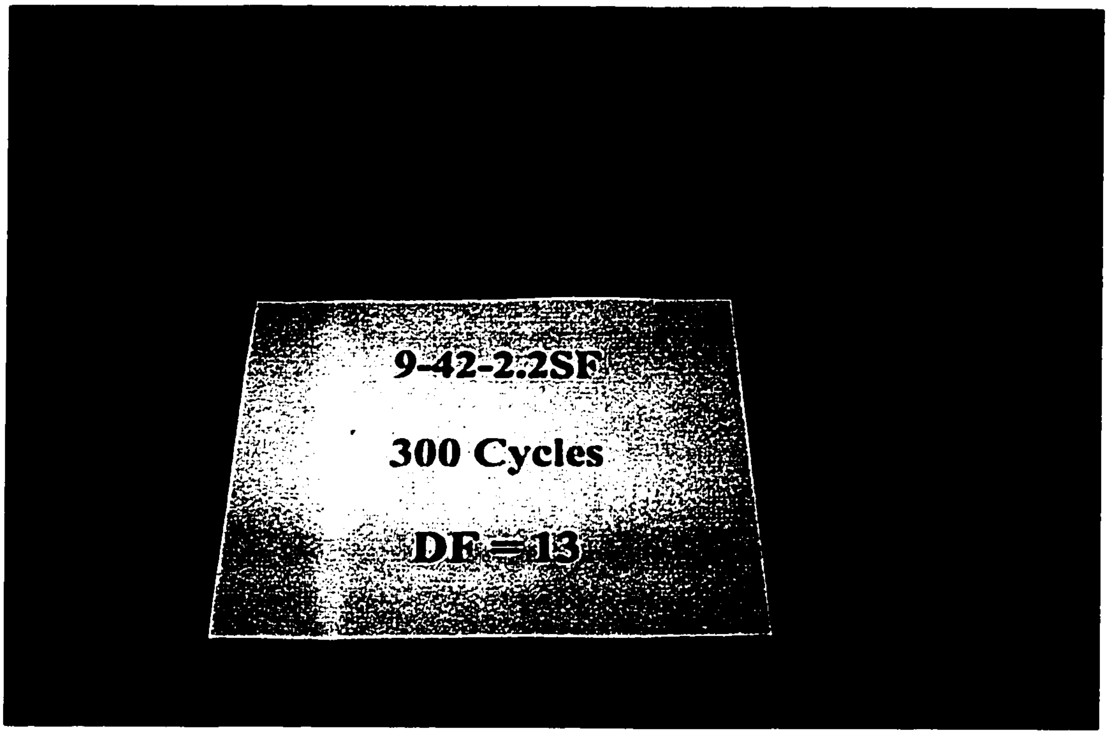

Figure 4.10. Mixture 9-42-2.2SF with a Durability Factor of 13.

The results from the research show that a total air content of four percent is sufficient to protect the concrete against freeze-thaw damage for concrete mixtures with a w/cm between 0.36 and 0.50. All mixtures containing a nominal air content of four percent had a durability factor of at least 91. Examination of the 90 percent confidence intervals (Figure C.5 in Appendix C) shows that 14 of the 15 mixtures with a targeted air content of four or six percent plot above a durability factor of 90 percent. Photographs of the tested specimens are shown in Figures 4.11 through 4.13. Without air entrainment, the mixture 9-50-0.9 had deteriorated by 141 cycles, but with addition of air entrainment, Mixture 9-50-3.7 (Figure 4.11) had a durability factor 91. At a w/cm of 0.42, mixtures had either fractured (Figure 4.8 and Figure 4.10) or had begun to deteriorate (Figure 4.9). But with 4.4 percent air, Mixture 9-42-4.4 had a durability factor of 100. Pinto and Hover (2001) also found acceptable durability with a nominal air content of 4 percent for mixtures with w/cm between 0.36 and 0.50.

For the mixtures in this study, a total air content of four percent is equal to an air content of 6.9 percent in the mortar (cement, fine aggregate, and water). This is still less than the mortar air content of 8.7 percent that is recommended for concrete containing 3/8 in. coarse aggregate (Mindess and Young, 1981). As explained in the literature review, the type, size, and quantity of coarse aggregate can affect the freeze-thaw durability of concrete (Neville, 1997). The conclusions from this research are limited to the mixtures and materials tested in the study. Decreasing the aggregate content would increase mortar content, which in turn increases the required air content of the mortar to be resistant to freeze-thaw cycles. Simply stated, as the mortar content increases, the air content required for the specimen to have sufficient freeze-thaw durability also increases.

If larger aggregates were used, additional paste would be required for workability, which would increase the required total air content when compared to a similar mix with less paste/mortar. Finally, using aggregates that are susceptible to freeze-thaw damage might also affect the need for air entrainment.

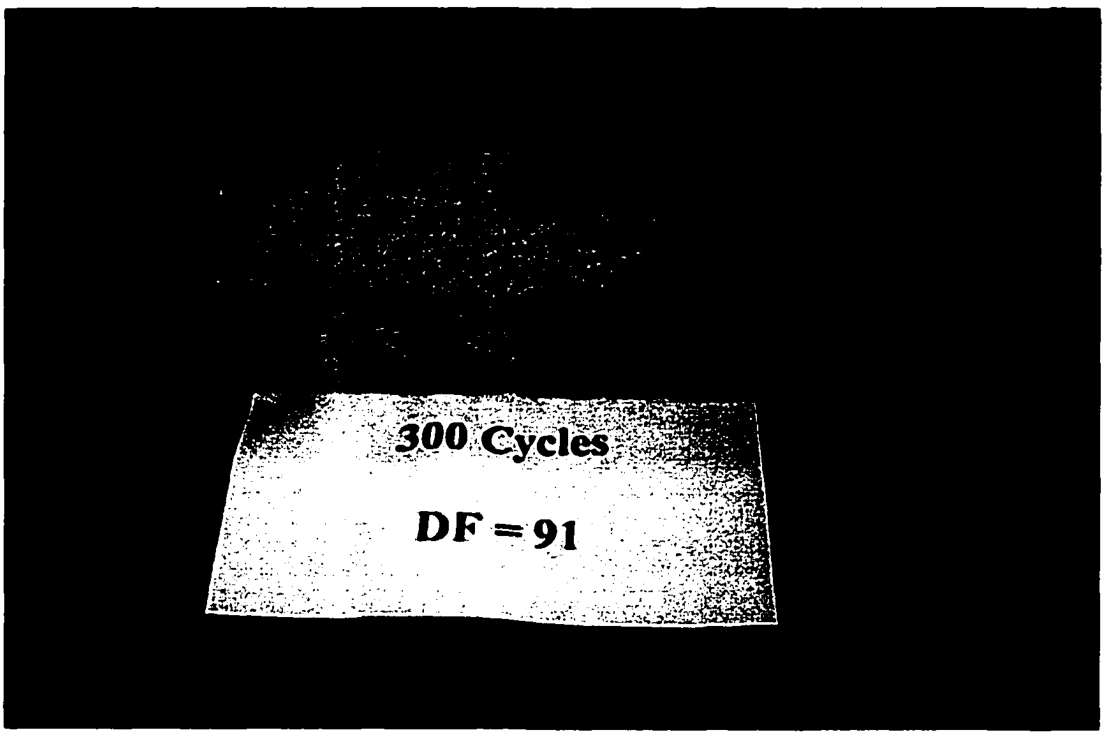

Figure 4.11. Mixture 9-50-3.7 with a Durability Factor of 91.

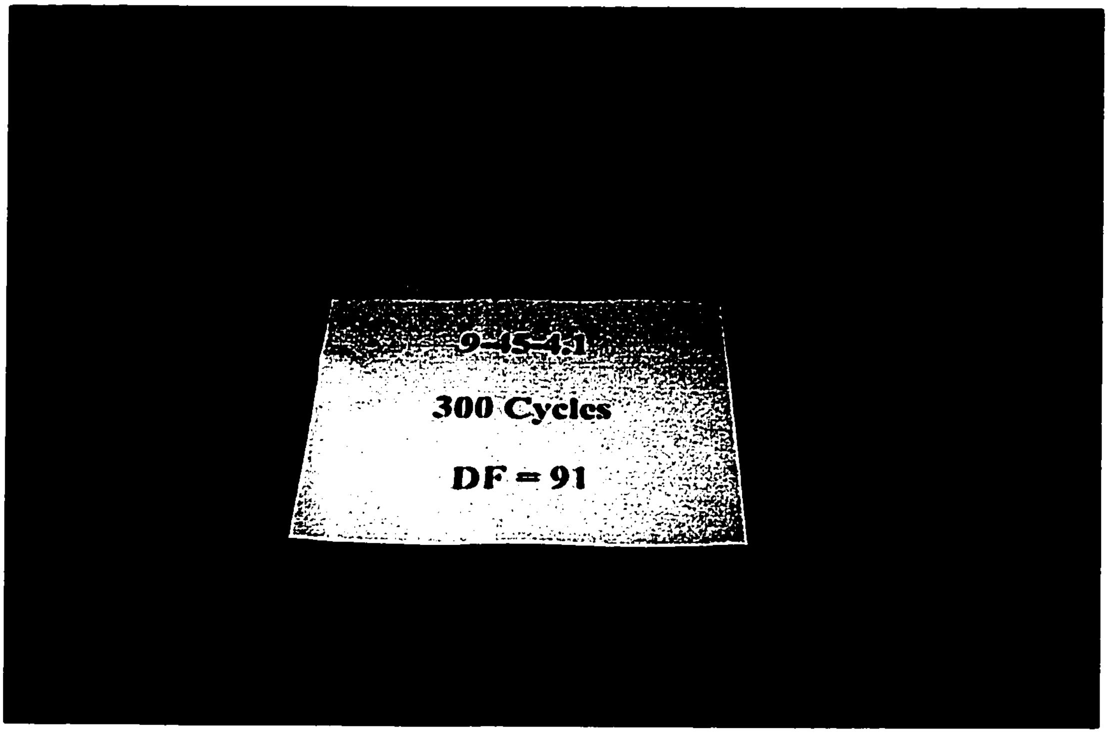

Figure 4.12. Mixtures 9-45-4.1 with a Durability Factor of 91.

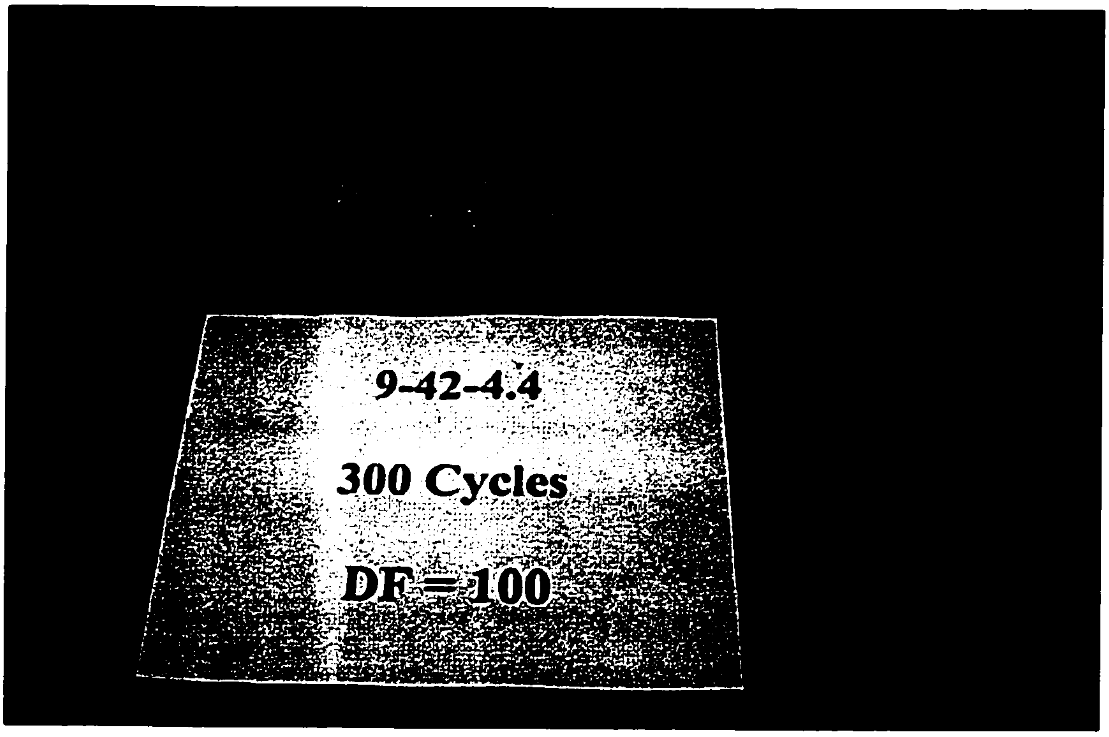

Figure 4.13. Mixtures 9-42-4.4 with a Durability Factor of 100.

At a w/cm of 0.36 and below, all non-air entrained mixtures had durability factors of 78 or greater. The freeze-thaw specimens are shown in Figures 4.14 through 4.17. The results indicate that air entrainment may not be necessary at w/cm equal to and less than 0.36. Once again these results are similar to Pinto and Hover (2001) whose non-air entrained mixture at a w/cm of 0.36 had a durability factor of 98. Also, a w/cm of 0.36 was the limit at which Mindess and Young (1981) theorized that air entrainment might not be necessary. Many researchers were able to produce durable non-air entrained mixtures, but Pinto and Hover (2001) and Hooten (1993) were the only researchers to produce durable non-air entrained mixtures at a w/cm of 0.35. Pigeon et al (1991) developed freeze-thaw durable mixtures at w/cm of 0.30 at lower. Mokhtarzadeh et al (1995) developed durable mixtures with w/cm of 0.29 and 0.31. Whereas Li et al (1994) had acceptable durability at w/cm of only 0.24 and lower.

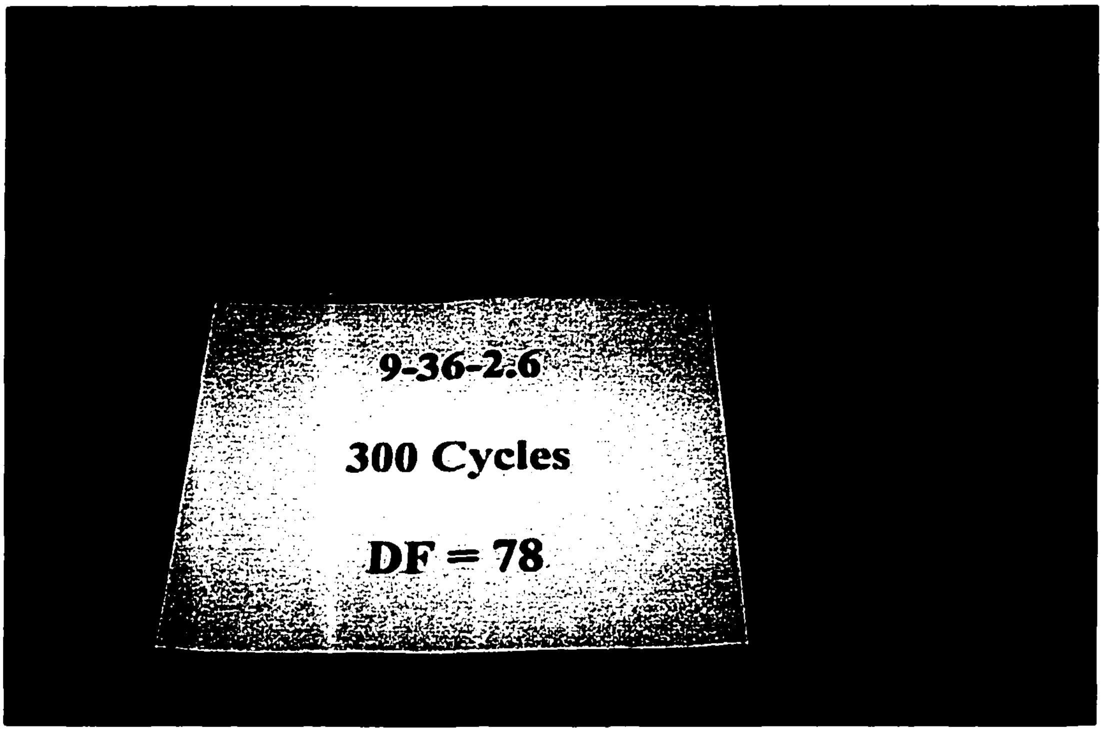

Figure 4.14. Mixture 9-36-2.6 with a Durability Factor of 78.

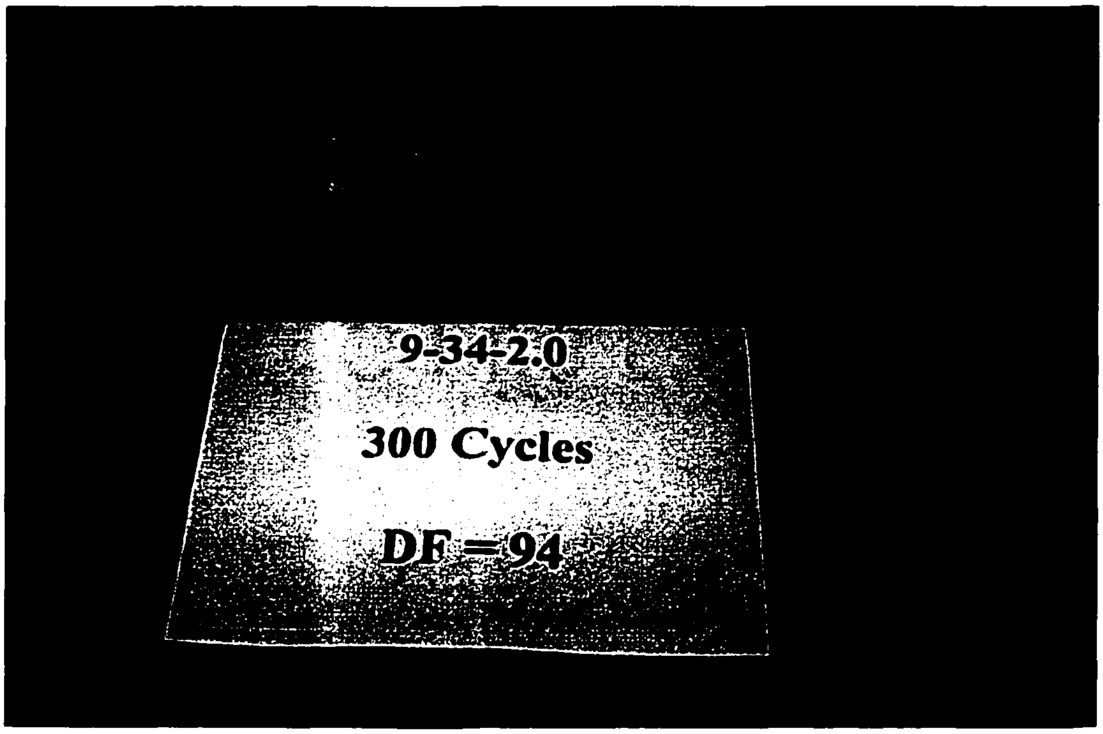

Figure 4.15. Mixture 9-34-2.0 with a Durability Factor 94.

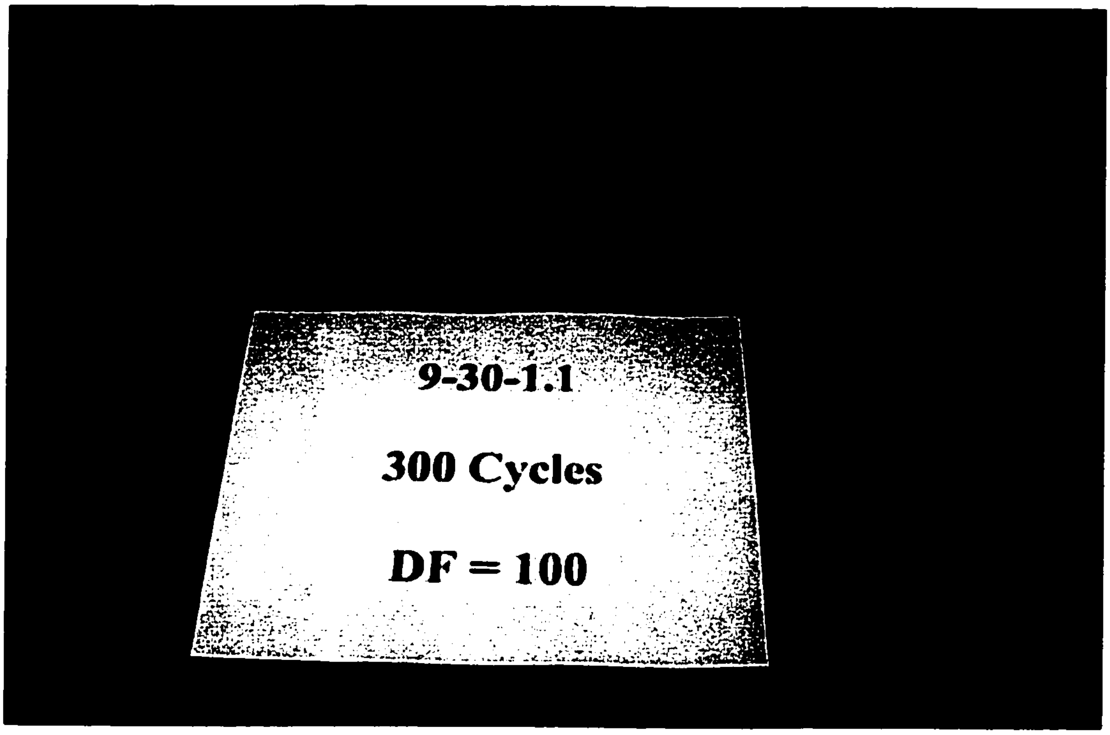

Figure 4.16. Mixture 9-30-1.1 with a Durability Factor of 100.

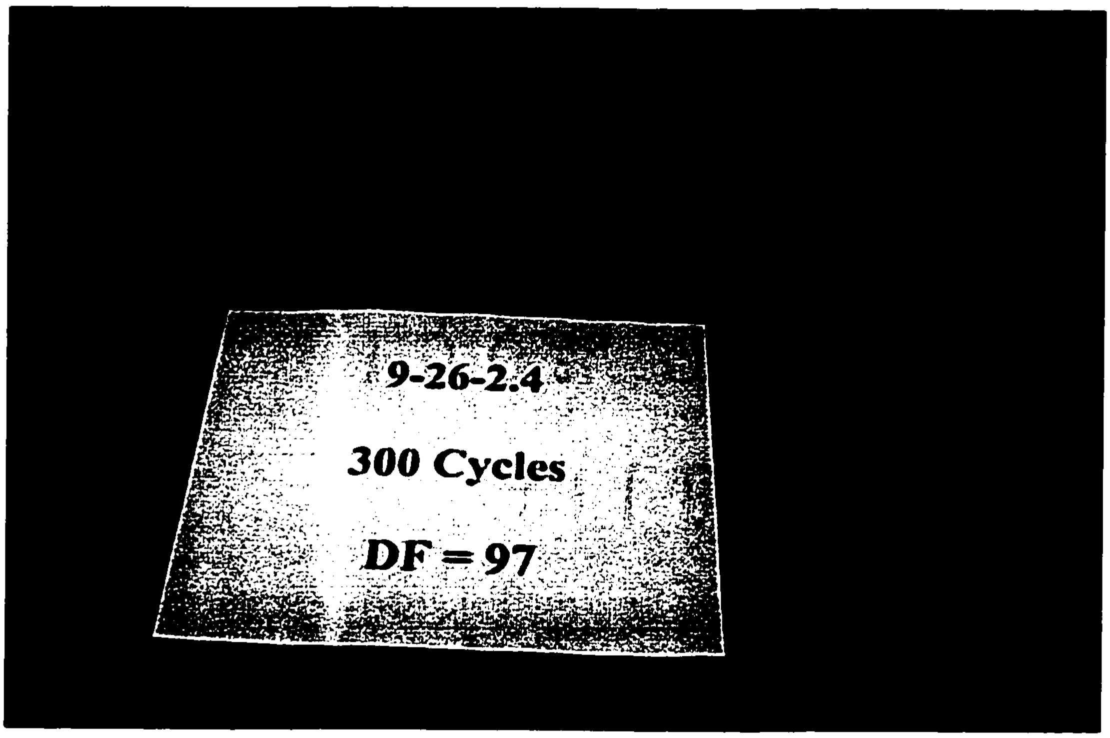

Figure 4.17. Mixture 9-26-2.4 with a Durability Factor of 97.

4.6 CONCLUSIONS

Results from the research show that air entrainment may not be necessary to achieve adequate freeze-thaw durability for concrete mixtures with w/cm less than 0.36. From the results in Table 4.9, it also appears that a total air content of 4 percent is enough to provide freeze-thaw durability for mixtures with a w/cm between 0.36 and 0.50. The findings of this research are very similar to those of Pinto and Hover (2001) who also developed non-air entrained, freeze-thaw durable mixtures at w/cm less than 0.36 and who also determined that a total air content of 4 percent was adequate for w/cm between 0.36 and 0.50.

However, all mixtures examined in the research contained the same type of coarse aggregate, which was known not to be susceptible to effects of freezing and thawing. The results from this research apply only to the materials used in the research. Mixtures containing aggregates with poor freeze-thaw durability may not perform as well as the mixtures tested in this research program. Before one can specify non-air entrained concrete in a harsh environment, specimens containing the materials to be used in the concrete should to be tested to determine its freeze-thaw durability. If freeze-thaw testing is not available, air entrainment should be employed in the concrete.

CHAPTER 5

INVESTIGATING THE PERMEABILITY OF HPC WITH AND WITHOUT ENTRAINED AIR

5.1 INTRODUCTION

This section of the research program investigated the permeability of high performance concrete (HPC) with and without entrained air. The previous chapter examined the freeze-thaw durability of HPC. Decreasing the amount of water entering a concrete structure can reduce the damage caused by freeze-thaw cycles. Therefore, reducing the permeability would increase the durability of concrete structures. This section of the research program had two tasks. The first task was to examine if there were any differences in the permeability of concrete with and without entrained air. The second task was to provide further information on the validity of the Rapid Chloride Ion Penetrability Test (ASTM C 1202) as a permeability measuring technique. Many researchers believe the RCIP test measures the concrete's conductivity not necessarily the permeability (Shi et al, 1998). Since the test procedure is relatively new, the results of the research will also provide the Oklahoma Department of Transportation (ODOT) information on the effectiveness of the test.

Concrete mixtures with varying water to cementitious material ratios (w/cm) were subjected to ASTM C 1202. The variables for the mixtures were total air content and w/cm. The targeted total air contents for the mixtures were 2 percent (no air entraining agent), 4 percent, and 6 percent. The results from the research show that entrained air has no noticeable effect on the RCIP of the specimens. The results also further support the view that the RCIP test may not be an adequate test for measuring the permeability of concrete mixtures containing mineral admixtures.

5.2 LITERATURE REVIEW

The longevity of a concrete structure is influenced by the permeability of the concrete. If the amount of water entering a concrete structure can be reduced, the life of the structure can be extended. As discussed in Chapter 4, the presence of water in hardened concrete can eventually deteriorate the concrete through continuous freeze-thaw cycles. To combat the harmful effects of freeze-thaw cycles, entrained air is added to the concrete. Entrained air voids provide air pockets where water can expand and water pressure can be relieved. Without these voids, continuous freeze-thaw cycles will eventually degrade and damage the concrete. With the help of entrained air voids, highly permeable concrete can be resistant to freeze-thaw damage. However, damage from freeze-thaw cycles is not the only concern of highly permeable concrete. Water can also carry harmful chemicals (such as deicing salts) into the structure, which can lead to the corrosion of the reinforcement and loss of the structural capacity of the member. Therefore, to produce durable concrete structures, the permeability of the concrete must be considered.

Like any other material, the permeability of concrete is influenced by the pore system of the concrete paste. The porosity of concrete is the measure of the total volume of pores within the concrete. The size, spacing, distribution, and continuity of the pores all have an effect on the permeability of concrete. Also of importance is the area around the aggregates known as the interface zone. This area is the interface between the paste and the aggregates. The interface area is known to have a different pore system than the remaining paste and is the location for early microcracking, but the "significance of the interface zone with respect to permeability remains uncertain (Neville, 1997)." To increase permeability, pores must be interconnecting to provide a path for water to flow. The addition of entrained air has little or no effect on the permeability of concrete. This is due to the fact that the entrained air bubbles are distributed evenly throughout the concrete and the pores do not connect (Mindess et al, 1981; Neville, 1997; Kosmatka et al, 1994).

Pozzolans such as fly ash, blast furnace slag, and silica fume also have an effect on the permeability of concrete. Pozzolans react with the byproducts of the cement hydration reaction to form additional calcium silicate hydrate (C-S-H). The additional C-S-H decreases the porosity of the cement paste, which in turn reduces the permeability (Mindess et al, 1981 and Neville, 1997).

There are several different standardized tests that measure the permeability of concrete. Many of the tests require a significant amount of time to measure the concrete's permeability. In 1991 a test was adopted by American Society for Testing and Materials (ASTM) that measured the concrete's permeability in six hours. This test is known as the Rapid Chloride Ion Penetrability Test (RCIP). Theoretically, the test measures the number of chloride ions that pass through a sample of concrete in a six-hour period of time. In general, the lower the permeability of the concrete the lower the amount of coulombs passed. Therefore, concrete with a high permeability will pass more coulombs.

Some researchers believe that the Rapid Chloride Ion Penetrability Test (ASTM C 1202) measures the concrete's conductivity but not necessarily the concrete's permeability (Shi et al, 1998). The addition of mineral admixtures such as fly ash, blast furnace slag, and silica fume may alter the concrete's conductivity by altering the pH of the pore solution, which could either increase or decrease the RCIP value. The pore fluid of hardened concrete consists mainly of Na, K, and OH ions. Shi et al (1998) reported that the addition of mineral admixtures might increase or decrease the concentrations of these ions. They reported that BFS decreases the concentration of the OH and K ions, but increases the Na concentration. Shi et al (1998) also reported that the replacement of cement with 5 percent silica fume "can lead to order of magnitude reductions in Na, K, and OH ion concentrations." They also reported that the addition of fly ash might either increase or decrease the Na and K concentrations.

It is theorized that these changes in concentrations of the pore fluid result in changes in the concretes' specific conductivity. When compared to the control mixture (100 percent portland cement), the addition of 50 percent BFS reduced the concrete's specific conductivity by 3.25 percent at 28 days and up to 24 percent by 730 days. A specific conductivity of a concrete mixture containing 5 percent silica saw a reduction of 70.6 percent at 28 days. Shi et al (1998) concluded that "it was not correct" to use the RCIP test to evaluate the rapid chloride penetrability of concrete containing mineral admixtures.

Zhao et al (1999) also examined the pore chemicals in concretes with mineral admixtures. They measured the concretes' resistivity and its RCIP. They also found that the concretes' resistivity increased with the addition of mineral admixtures and as the concretes' resistivity increased its RCIP values decreased. These findings are similar to the results of a study conducted by Wee et al (2000).

Pfiefer et al (1994) examined the correlation of the RCIP test to the 90-day salt ponding test (AASHTO T259). They, like Shi et al, also state that the addition of silica fume can substantially reduce the number of coulombs passed. Pfiefer et al states that typical concrete "may have a 5- to 10-fold decrease in coulombs passed when 7 percent silica fume is added, while the actual chloride ingress after 90-day ponding tests may decrease only one to two times." For selecting materials for low permeability concretes, Pfiefer et al recommended that the selection should based on the 90-day salt ponding tests (AASHTO T259) not the RCIP test (ASTM C 1202). Scanlon et al (1996) also examined the correlation between the 90-day ponding test (AASHTO T259) and the RCIP test (ASTM C 1202) for concretes containing fly ash and/or silica fume. They too concluded that there were inconsistencies between the permeability determined by the RCIP test and the permeability measured by the 90-day ponding test. Scanlon et al stated that the correlation between coulomb values and chloride ingress given in AASHTO T277 or ASTM C 1202 "appears invalid for use with concretes containing silica fume, fly ash or a HRWRA."

5.3 EXPERIMENTAL PROCEDURES

5.3.1 Scope

The research program had two tasks. The first task was to examine if there were any differences in the permeability of concrete with and without entrained air. The second task was to provide further information on the validity of the Rapid Chloride Ion Penetrability test (ASTM C 1202). Several different concrete mixtures with varying w/cm and varying total air contents were subjected to the RCIP test (ASTM C 1202). The testing matrix is shown in Table 5.1. The w/cm ranged from 0.26 to 0.50. The targeted total air contents were 2, 4, and 6 percent.

Table 5.1. Testing Matrix

Target Total Air Content (%)	w/cm							
	0.26	0.30	0.32	0.34	0.36	0.42	0.45	0.50
2	X	X	-	X	X	X	-	X
4	X	X	X	-	X	X	X	X
6	X	X	-	-	X	X	-	X
X = indicates batching and testing of these variables								

5.3.2 Materials

Ash Grove Type III Cement was used in all mixtures. The coarse aggregate was 3/8 inch, crushed limestone from Davis, Oklahoma. The fine aggregate was washed river sand from Dover, Oklahoma. The material properties of the cement and aggregates are listed in Chapter 2. To provide adequate workability, a water reducer (WR) and/or a high range water reducer (HRWR) was used. The WR used was WRDA with Hycol, and the

HRWR used was ADVA Flow. An air entraining agent (AEA), DARAVAIR 1000, was used to attain the required target total air contents. Two mixtures contained Class C fly ash, and two mixtures also contained silica fume.

5.3.3 Mixtures

The same mixtures used to examine the freeze-thaw durability of HPC in Chapter 4 were also used to examine the permeability of HPC mixtures. The mixtures proportions are located in Table 4.2 through 4.6 of Chapter 4.

5.3.4 Tests

All mixtures were subjected to several tests. The fresh concrete properties tested were slump (ASTM C 143), unit weight (ASTM C 138), and air content (ASTM C 231). The hardened properties tested were compressive strength (ASTM C 39) at one, seven, twenty-eight, and fifty-six days of age, freeze-thaw durability (ASTM C 666, Procedure A), and the rapid chloride ion penetrability test (RCIP, ASTM C 1202) at 28 and 56 days of age. The compressive strength was discussed in detail in Chapters 2 and 3. The freeze-thaw durability was discussed in Chapter 4. This chapter will mainly focus on the permeability of the mixtures measured by the RCIP test.

The RCIP was tested at 28 and 56 days of age. Four specimens were tested from each mixture. Two days prior to testing, the top two inches of the cylinders to be tested were cut off using a masonry saw. The sides of the cylinders were then covered with epoxy (Figure 5.1). Once the epoxy dried, the cylinders were placed in a vacuum of no more than 1mm of Hg for three hours. After the three hours, water was introduced into the container with the vacuum pump continuing to run for an additional hour. After the additional hour, the vacuum pump was turned off and the container was opened to atmospheric pressure. The cylinders were then ready to test once they had been submerged in water for 18 hours.

End caps, conforming to ASTM C 1202, were manufactured to test the samples. The end caps were then placed on the end of the prepared specimens (Figure 5.2). In each of the end caps, sodium hydroxide or sodium chloride was placed. A positive terminal was then attached to the cap containing 0.3 N NaOH solution and a negative terminal attached to the cap containing 3.0 percent NaCl. Then, a potential of 60 volts was applied to each specimen for six hours. A PC based data acquisition system was used to measure the total coulombs passed through each specimen for the six-hour period (Figure 5.3).

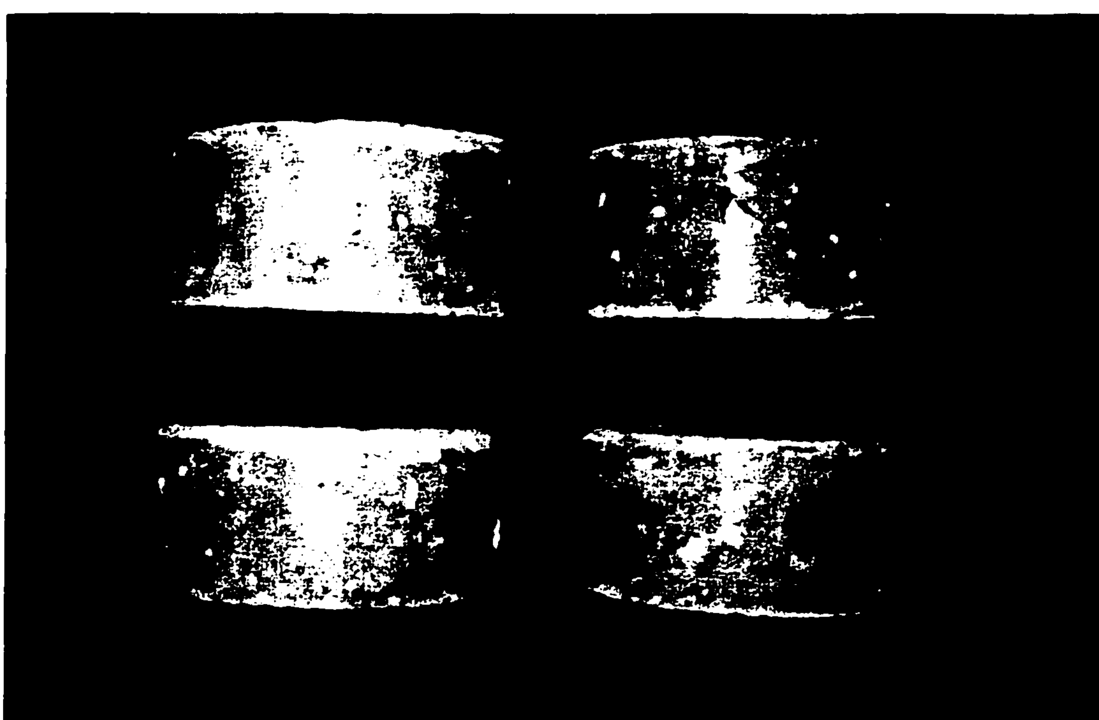

Figure 5.1. Epoxy Coated Test Specimens.

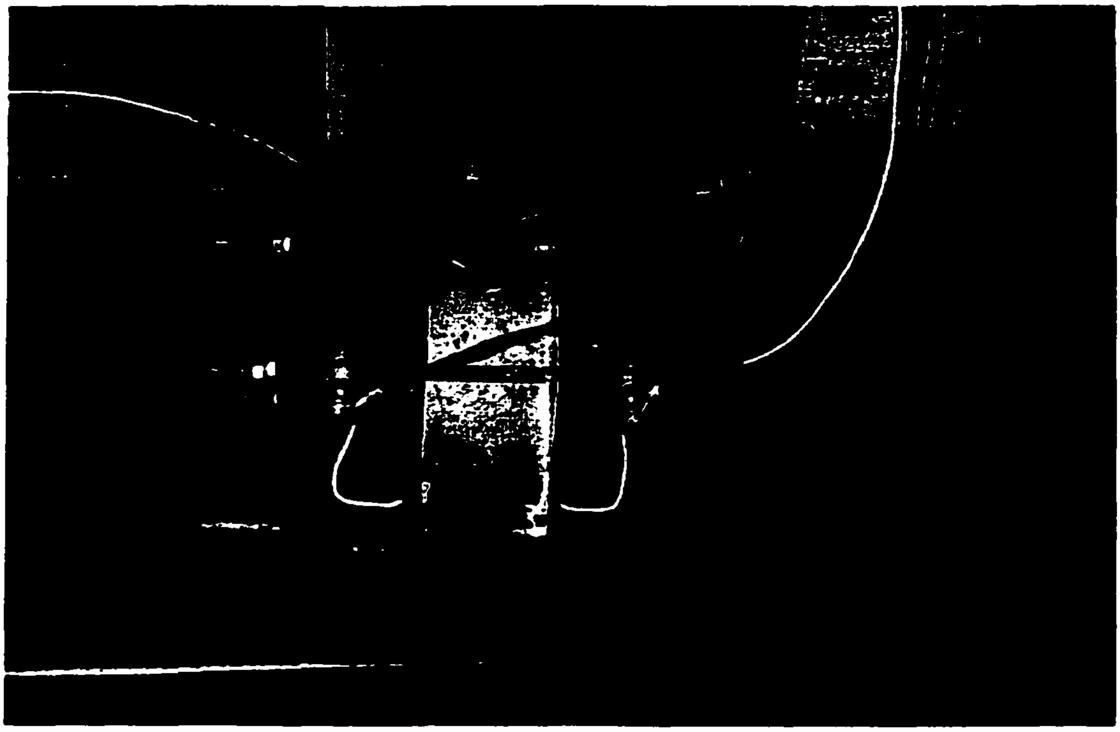

Figure 5.2. End Caps for RCIP Test.

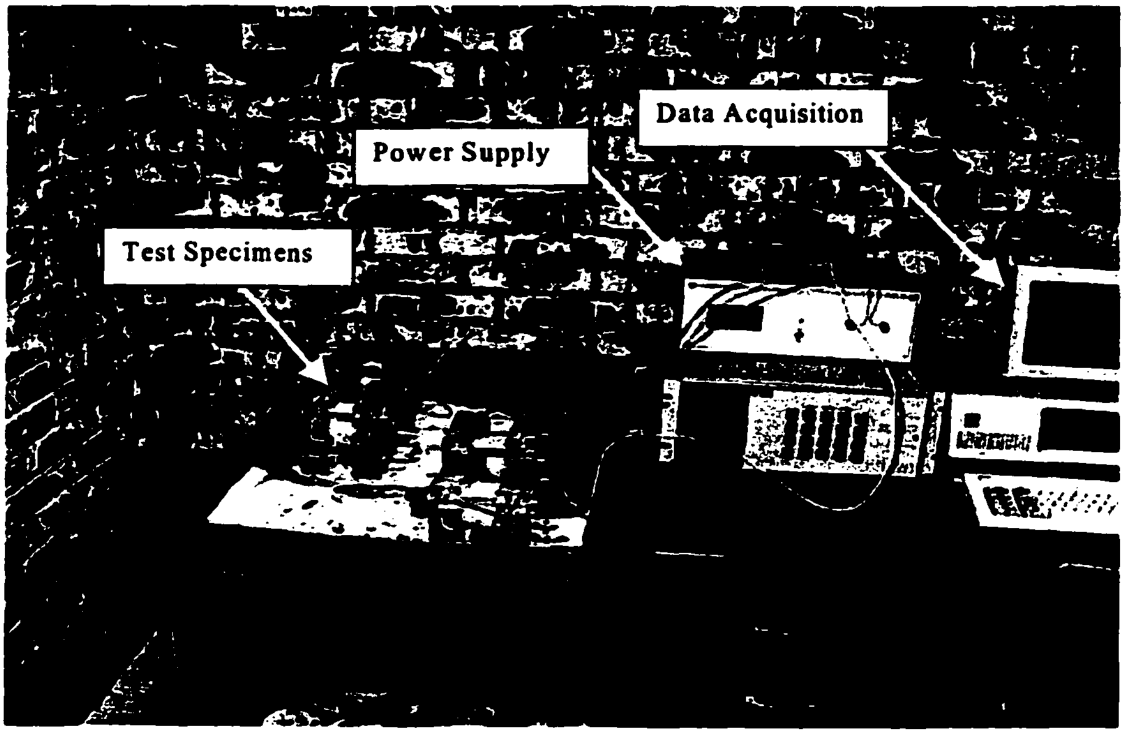

Figure 5.3. RCIP Test Set-up.

5.4 RESULTS

5.4.1 Fresh Properties

The results of the fresh property tests are shown in Table 5.2. The air contents for all the mixtures were near the targeted total air contents.

Table 5.2. Fresh Concrete Properties

Mixture	Slump (in.)	Total Air Content (%)	Unit Weight (lb/ft^3)
9-26-2.4	6.50	2.4	152.3
9-26-3.8	8.00	3.8	149.3
9-26-5.6	4.25	5.6	147.2
9-30-1.1	10.00	1.1	150.9
9-30-4.5	1.50	4.5	147.5
9-30-5.7	10.75	5.7	146.2
9-32-4.1	6.50	4.1	147.2
9-34-2.0	11.50	2.0	150.6
9-36-2.6	2.50	2.6	147.8
9-36-3.8	3.50	3.8	146.8
9-36-6.2	2.00	6.2	139.6
9-42-1.8	3.25	1.8	145.8
9-42-2.2FA	2.00	2.2	145.9
9-42-2.4SF	1.00	2.4	143.8
9-42-4.4	3.75	4.4	142.7
9-42-5.9	4.00	5.9	139.1
9-45-4.1	5.50	4.1	140.3
9-50-0.9	6.25	0.9	143.4
9-50-1.5FA	8.00	1.5	142.3
9-50-2.2SF	3.25	2.2	140.4
9-50-3.6	9.50	3.6	135.1
9-50-6.6	10.00	6.6	133.4

5.4.2 Hardened Properties

The results of the hardened property tests are shown in Tables 5.3 and 5.4. The results from the compressive strength tests are shown in Table 5.3. Shown in Table 5.4 are the number of coulombs passed at both 28 and 56 days of age. Also shown in Table 5.4 is the ASTM permeability classification based on the number of coulombs passed. The compressive strength results are the average of three compressive strength tests, and for most mixtures the RCIP results are the average of four specimens. The individual compressive strength tests and the individual RCIP tests are listed in Appendix C and Appendix D, respectively. The statistical data for the RCIP results are also located in Appendix D.

Table 5.3. Compressive Strength Results.

Mixtures	Average Compressive Strength (psi)			
	1 day[1]	7 days[1]	28 days[1]	56 days[1]
9-26-2.4	6830	11,040	12,190	12,960
9-26-3.8	4830	10,440	11,270	12,090
9-26-5.6	4560	9270	10,560	11,290
9-30-1.1	6150	10,050	12,190	13,600
9-30-4.5	5610	8650	9960	11,010
9-30-5.7	6000	9130	10,270	11,030
9-32-4.1	5730	8320	9650	10,700
9-34-2.0	4700	9080	10,250	11,200
9-36-2.6	4220	7210	9180	9620
9-36-3.8	4580	6960	8340	9040
9-36-6.2	3660	6790	7700	8450
9-42-1.8	2690	6760	8690	9050
9-42-2.2FA	2830	6840	8680	9100
9-42-2.4SF	4070	6290	7770	8240
9-42-4.4	2750	4940	6470	6860
9-42-5.9	2640	4900	6110	6710
9-45-4.1	2600	4900	6110	6980
9-50-0.9	2020	4730	6220	6990
9-50-1.5FA	1970	6020	7360	7870
9-50-2.2SF	2110	4390	6100	6380
9-50-3.6	1560	5220	6340	6760
9-50-6.6	1650	4320	5560	6150

1) Average compressive strength of three tests.
2) Individual tests and statistical data are shown in Appendix C.

Table 5.4. RCIP Results.

Mixture	Average Number of Coulombs Passed		ASTM Classification at 56 Days
	28 days[1]	56 Days[1]	
9-26-2.4	710	520	Very Low
9-26-3.8	671	660	Very Low
9-26-5.6	956	799	Very Low
9-30-1.1	1270	849	Very Low
9-30-4.5	1789	1281	Low
9-30-5.7	1222	756	Very Low
9-32-4.1	1534	811	Very Low
9-34-2.0	2471	1739	Low
9-36-2.6	2453	2319	Moderate
9-36-3.8	3416	2949	Moderate
9-36-6.2	3500	2166	Moderate
9-42-1.8	4546	4551	High
9-42-2.2FA	4639	3514	Moderate
9-42-2.4SF	1343	1134	Low
9-42-4.4	5905	5110	High
9-42-5.9	5430	4216	High
9-45-4.1	5969	5143	High
9-50-0.9	9450	7410	High
9-50-1.5FA	8624	5491	High
9-50-2.2SF	1868	1484	Low
9-50-3.6	8057	7549	High
9-50-6.6	8623	8467	High

1) Average RCIP value of four tests.
2) Individual tests and statistical data are shown in Appendix D.

5.5 DISCUSSION OF RESULTS

5.5.1 Fresh Properties

The results from the total air content test and the unit weight test directly correlated. Increases in total air content resulted in decreases in unit weight for mixtures of like w/cm, which was expected. For mixtures with the same w/cm, the unit weight decreased as the total air content increased, which was expected. Also, the unit weights of mixtures with like total air contents decreased as the w/cm increased, which was also expected. The results were discussed in more detail in Chapter 4.

5.5.2 Hardened Properties

5.5.2.1 Compressive Strength. The compressive strength tests for the mixtures followed an expected trend. As the w/cm decreased, the compressive strengths increased. This trend is verified through examination of the 90 percent confidence intervals in Appendix C. Also, for mixtures with like w/cm, the increase in total air content resulted in a decrease in compressive strength. In Chapter 3, the decrease in compressive strength per increase in air content was examined for the mixtures. The decreases in compressive strength generally followed the "2 – 5 percent decrease in compressive strength for every 1 percent increase in total air content" rule of thumb.

5.5.2.1 Permeability. From examination of Figures 5.4 and 5.5, it is apparent that increases in w/cm increased the number of coulombs passed for mixtures containing only portland cement. This trend is also evident by examination of the 90 percent confidence intervals in Figures D.1 and D.2 of Appendix D. This increase in the number of coulombs passed was expected. The effect of the w/cm on the permeability of concrete has been well documented (Neville, 1997; Mindess et al, 1981). A decrease in the w/cm reduces the porosity of the concrete, which results in a more impermeable concrete.

At 28 days of age, the total charge passed ranged from 671 coulombs for mixture 9-26-3.8 to 9450 coulombs for mixture 9-50-0.9 (mixtures containing only portland cement). At 56 days of age, the RCIP value decreased for most mixtures. By this age, mixtures with a w/cm equal to or less than 0.34 had low permeability based on ASTM C 1202 classifications. These findings are similar to the findings of Pinto et al (2001) who also found that to achieve low permeability without the use of supplementary cementitious materials, a w/cm of 0.35 or lower would be necessary. Mixtures (containing only portland cement) with a w/cm of 0.42 or greater would be classified as having high permeability.

The addition of fly ash had little affect on the chloride penetrability when compared to identical mixtures without fly ash. At a w/cm of 0.42, mixture 9-42-2.2FA passed approximately the same number of coulombs at 28 days of age as mixture 9-42-1.8 (4546 versus 4639 coulombs). Both mixtures would be classified as having high permeability. But, by 56 days of the addition of fly ash reduced the number of coulombs by more than 20 percent. At 56 days of age, mixture 9-42-1.8 would still be classified as having high permeability, but mixture 9-42-2.2FA would now be classified as having moderate permeability.

For mixtures with a w/cm of 0.50, the addition of fly ash decreased the number of coulombs passed by approximately 10 and 25 percent at 28 and 56 days of age, respectively. Even though the addition of fly ash reduced the total number of coulombs, all four mixtures would still be classified as having high permeability. These results are consistent with the report of Shi et al who stated that fly ash could either increase or decrease the Na and K concentrations of the pore fluid which could result in higher or lower chloride penetrability as classified by ASTM C 1202.

Mixtures 9-42-2.4SF and 9-50-2.2SF contained silica fume at a replacement rate of 5 percent. Figures 5.4 and 5.5 show the dramatic effect that the addition of silica fume has on the rapid chloride ion penetrability of the concrete. At 28 and 56 days of age, the total coulombs passed for the silica fume mixtures was more than 75 percent less than that of identical mixtures (9-42-1.8 and 9-50-0.9) without silica fume. The differences are statistically significant through examination of the 90 percent confidence intervals in Figures D.1 and D.2 in Appendix D. These results are consistent with Shi et al who reported that the replacement of cement with 5 percent silica fume "can lead to order of magnitude reductions in Na, K, and OH ion concentrations" which would greatly reduce the number of coulombs passed.

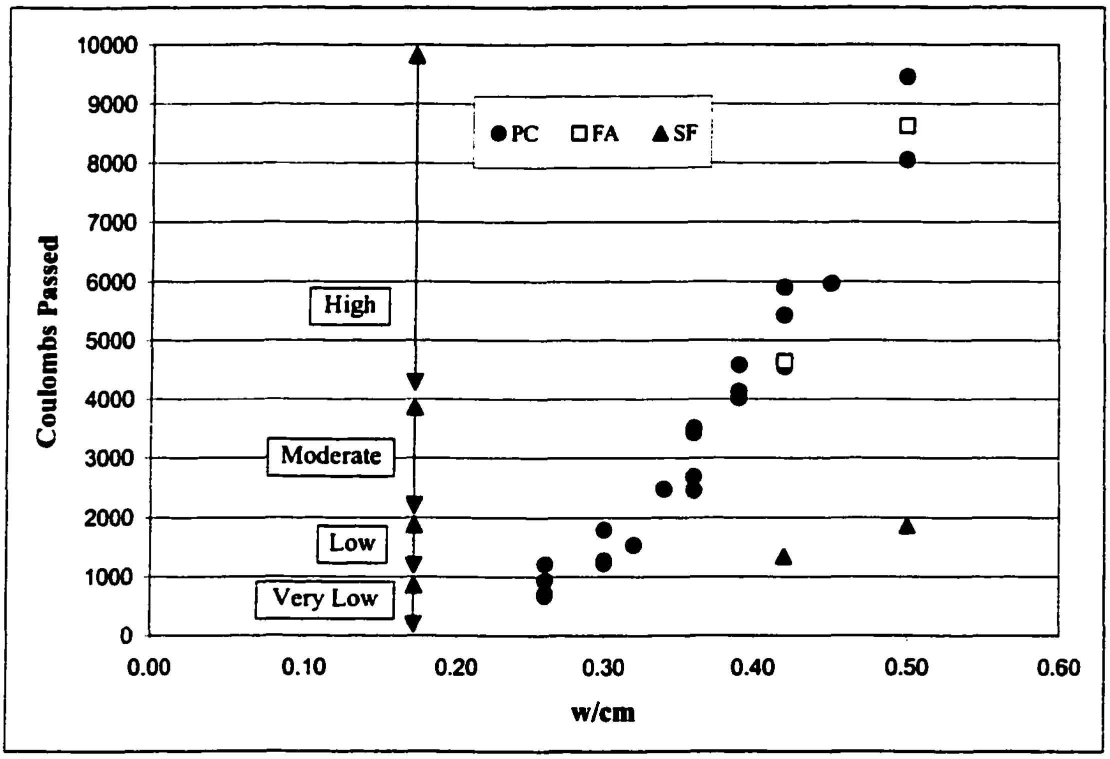

Figure 5.4. 28-Day RCIP Results and ASTM C 1202 Permeability Classifications.

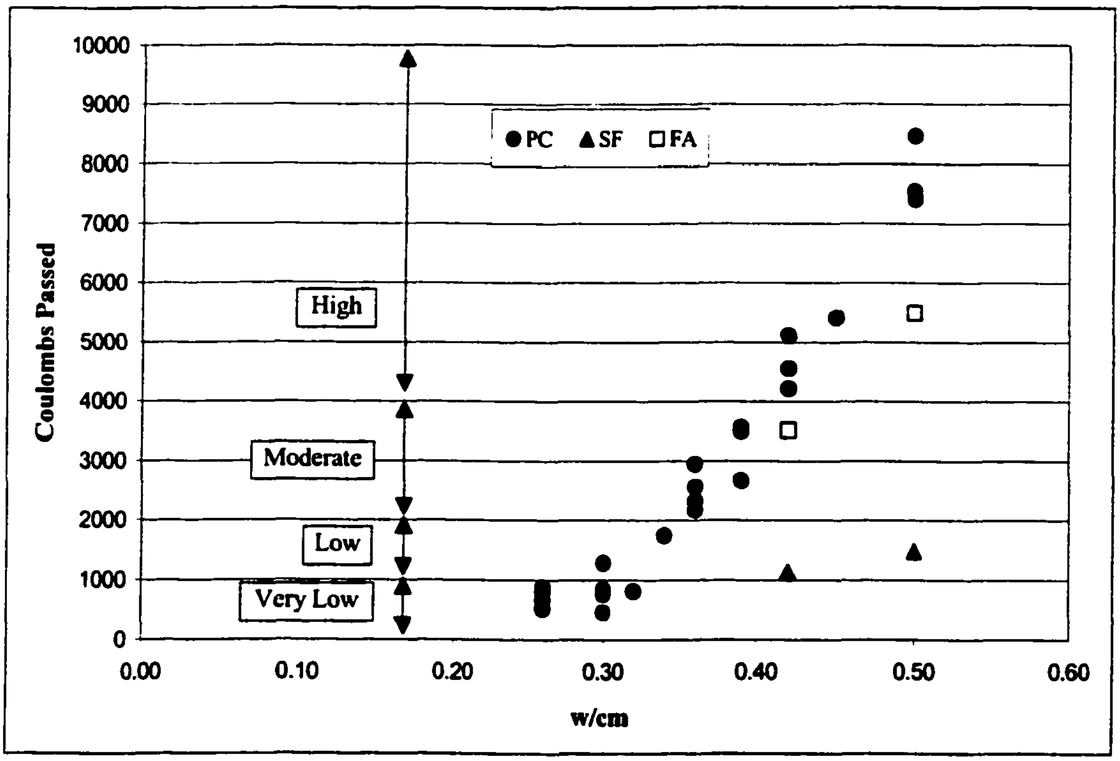

Figure 5.5. 56-Day RCIP Results and ASTM C 1202 Permeability Classifications.

The effect of entrained air on the RCIP is shown in Figures 5.6 and 5.7. As shown in the figures, the quantity of entrained air had no consistent effect on the RCIP. For example, the number of coulombs increased as the total air content increased for mixtures with a w/cm of 0.50 (at 56 days of age), while there was no consistent effect at other w/cm. Even though there were inconsistencies at most w/cm, the addition of entrained air did not affect the ASTM C 1202 permeability classifications. As an example, for mixtures with a w/cm of 0.36 (at 56 days), the addition of approximately two percent entrained air (mixture 9-36-3.8) increased the number of coulombs passed to 2949 coulombs when compared to the control mixture (mixture 9-36-2.6) without entrained air, which passed 2319 coulombs. However, the mixture with four percent entrained air (9-36-6.2) passed the least amount of coulombs (2116) of the three mixtures. Even though the total number of coulombs passed for these mixtures ranged from 2166 to 2949, they would all still be classified as having moderate permeability. These results are similar to the findings of Myers et al (1997) who found that entrained air had no "appreciable effect on the permeability of high performance concrete."

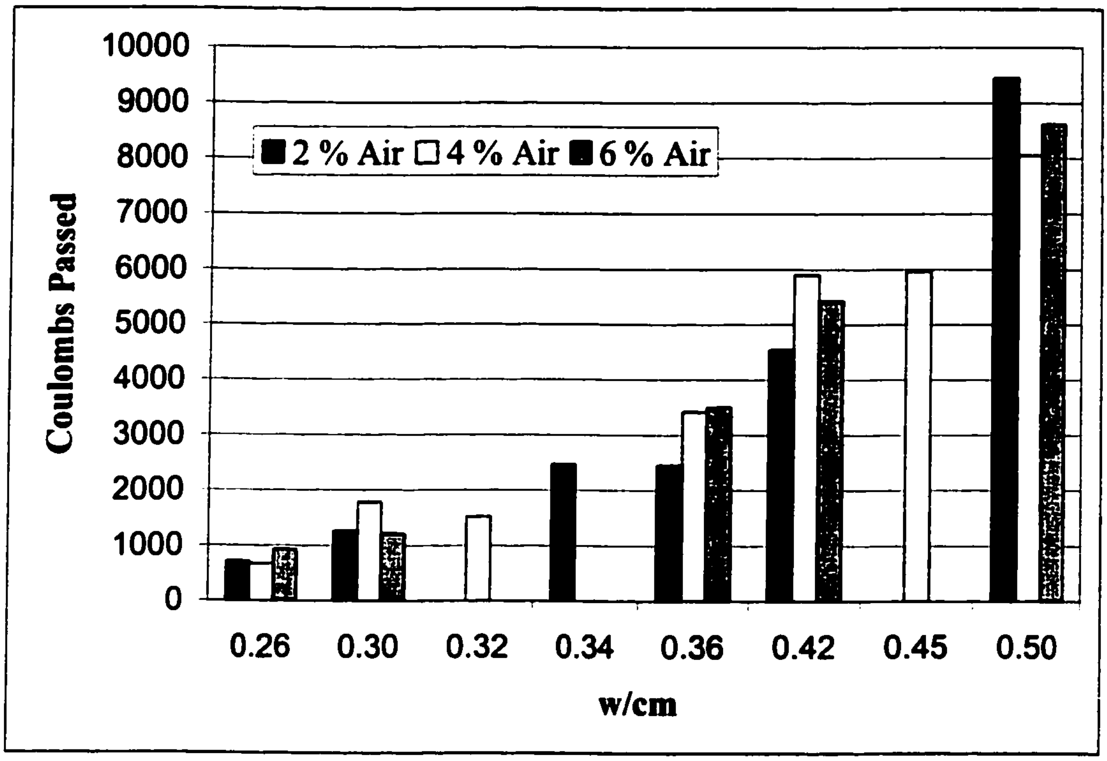

Figure 5.6. Effect of Entrained Air on 28-Day RCIP.

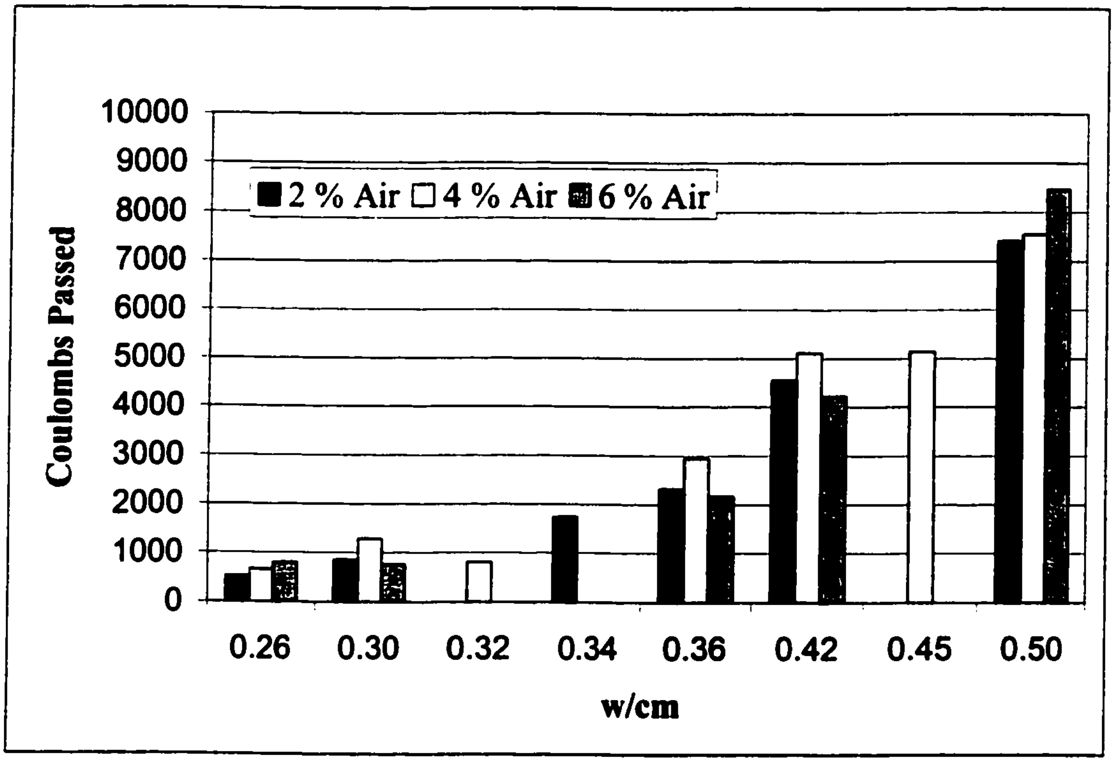

Figure 5.7. Effect of Entrained Air on 56-Day RCIP.

5.6 CONCLUSIONS

The purpose of the research program was to examine the permeability of air entrained concretes and to provide additional data on the effectiveness of the RCIP Test (ASTM C 1202) as a means to measure concrete penetrability. The findings of the research are summarized below.

1. The addition of entrained air had no noticeable effect the permeability of the mixtures. The findings are consistent with those of other researchers (Mindess et al, 1981; Neville, 1997; Kosmatka et al, 1994).
2. The permeability (measured by ASTM C 1202) increased as w/cm increased for mixtures containing only portland cement which was expected. These results are similar to those of other researchers (Mindess et al, 1981; Neville, 1997; Kosmatka et al, 1994) who state that increases in w/cm result in greater permeability.
3. To achieve low permeability concrete (based on ASTM C 1202 classifications), mixtures should be cast at a w/cm equal to or less than 0.34. The results are similar with Pinto et al (2001) who found that a w/cm of 0.35 was necessary to produce concrete with low permeability based on ASTM C 1202.
4. For mixtures not containing any mineral admixtures, the RCIP test appears to be effective in measuring concrete's chloride ion penetrability. As shown in Figure 5.4 and 5.5, the concrete's penetrability increases as the w/cm increase,

which is expected. All mixtures containing only portland cement follow this trend.

5. Once mineral admixtures are introduced, this trend is no longer valid. When compared to similar control mixtures, silica fume reduced the chloride ion penetrability by more than 75 percent. These conclusions are similar to the findings of other researchers (Shi et al, 1998; Zhao et al, 1999; Pfiefer et al, 1994; Scanlon et al, 1996) who question the use of the RCIP test when testing concrete that contains mineral admixtures.
6. Since the RCIP test does not directly measure permeability, other factors can influence the parameter the test actually does measure, which is concrete resistivity. Therefore, other factors (such as pore fluid chemistry) may influence the parameter actually measured, leading to a basic need for users to understand what the test actually measures so that results can be put into proper perspective. When testing concrete containing mineral admixtures, one should compare the results to an identical mixture without mineral admixtures. This comparison with mixtures containing only portland cement may provide a better idea as to the true permeability of the concrete.

CHAPTER 6

EXAMINING THE PRESTRESS LOSSES AND ALLOWABLE COMPRESSIVE STRESS AT RELEASE OF BRIDGE GIRDERS WITH AND WITHOUT AIR ENTRAINMENT

6.1 INTRODUCTION

This portion of the research examined the prestress losses of non-air entrained and air entrained bridge girders. The main task was focused on determining if the addition of entrained air had any noticeable effects on the prestress losses. The measured losses were then compared with the losses calculated using two prevalent methods used for estimating prestress losses. Another objective of the research was to provide additional data with regard to increasing the allowable compressive stress at release. The results of the research show that the addition of entrained air had no noticeable effect on the prestress losses and that increasing the allowable compressive stresses at release may not be beneficial.

6.2 LITERATURE REVIEW

6.2.1 Prestress Losses

Over time the initial prestressing force that is applied to a member decreases in magnitude. During a period of approximately five years, a prestressed member may lose an estimated 25 percent of the initial prestress force (Nawy, 1996, and Lin and Burns, 1981). Elastic shortening, creep, shrinkage, and relaxation are the four major contributors to prestress losses. Of the four major losses in pretensioned structures, elastic shortening is the only loss that is not time dependent and is independent of the others. Elastic shortening is the loss of prestress force that occurs when the strands are released (at transfer of the prestress force). At transfer, the strands are cut and this force, which is applied to the concrete through the strands, shortens the concrete. The strands also shorten with the concrete thereby reducing the initial prestress force (Lin and Burns, 1981). The loss due to elastic shortening occurs only once and is not affected by time.

The remaining losses (creep, shrinkage, and relaxation) are time dependent and dependent on one another. Creep is defined as the deformation of concrete due to sustained loads. As the girders deform or shorten due to the prestress force, the length of the strands shortens which results in a loss of prestress force. The loss due to creep may account for approximately 25 percent of the total losses. The majority of this loss occurs within the first year. Creep studies have shown that 60 to 83 percent of the total creep occurs during the first year (Lin and Burns, 1981). A similar loss occurs in the prestressing strands. Steel relaxation is the loss of prestress due to constant strain in the prestressing strands.

Another time dependent loss is shrinkage. Concrete decreases in volume when it loses water through evaporation. Drying, plastic, carbonation, and autogenous are all different types of shrinkage. Drying shrinkage occurs in the hardened concrete and is a result of moisture loss. Plastic shrinkage occurs in fresh concrete and is the loss of water from the paste of concrete. Carbonation shrinkage occurs when the cements paste reacts chemically with carbon dioxide. Autogenous shrinkage is a type of drying shrinkage that is caused by the internal consumption of water during hydration (Mindess and Young, 1981). Like creep and elastic shortening, as the concrete shortens, the prestress strands also shorten which results in a loss of prestress force. The loss due to shrinkage accounts for estimated 28 percent of the total losses. As with creep, approximately 80 percent of the total shrinkage will occur within the first year (Lin and Burns, 1981).

There are many methods used to estimate the effective prestress force (prestress force existing after all losses). Two widely accepted methods are the American Association of State Highway and Transportation Officials (AASHTO) and the Precast/Prestressed Concrete Institute (PCI) equations. Research is now being conducted to determine the accuracy of these equations when measuring the losses of HPC bridge girders.

Roller et al (1995) examined the prestress losses in HPC bridge girders. Along with prestress losses, creep and shrinkage of the concrete used to cast the girders were also measured. The girders tested were 70 ft. long and 54 in. deep. Four girders were cast, but the losses were reported for only Girder 3 and Girder 4. The two girders were subjected to two different curing regimens. Girder 3 was steam cured for 24 hours at 140°F and after an additional 10 hours the forms were removed. Girder 4 was not steam cured, but cured under a waterproof tarpaulin for 10 hours and the forms were removed 12 hours after casting.

Concrete mixtures with a w/cm of 0.27 and 0.28 were used to cast the girders. Girder 3 had a release strength and 28-day compressive strength of 8920 and 9930 psi, respectively. Girder 4 had release and 28 day compressive strengths of 6960 and 8830 psi, respectively. The prestress losses were measured using internal strain meters. The results of their research showed the AASHTO equations overestimated the losses by approximately 50 percent at 18 months for Girder 3. However, the researchers report that the steam curing of Girder 3 may have affected the early age prestress losses. When compared to Girder 4 (non-steam cured), the early age losses of Girder 3 were significantly lower. The results also showed that the AASHTO equations for estimating creep and shrinkage "may be overly conservative for high strength concrete". The researchers recommend further study in the creep and shrinkage behavior of HPC to determine if the AASHTO equations can be modified.

Roller et al (2000) again examined the prestress losses, but this research program examined the losses of a bridge built by the Louisiana Department of Transportation and Development. Construction of the bridge was completed in October 1999. The bridge consisted of five 73 ft. spans. The compressive strength of the girders was 7020 psi at release (39 hours) and 10,100 psi at 56 days of age. The prestress losses were reported for 12 months. The measured losses were approximately 35 percent less than the calculated losses. These results are consistent with Roller et al's earlier research.

Pessiki et al (1996) examined the effective prestress force in bridge girders that were 28 years old. The girders were I-beams with dimensions of 24 x 60 in. and a length of 90 ft. 5 in. The girders were removed from a seven span bridge on I-80 in Pennsylvania. Load tests were performed on the girders to determine the decompression load. Visual observations, strain gauges, and displacement transducers were used in obtaining the decompression loads. The average prestress loss for both girders were 18 percent compared a loss of 33 percent predicted by the AASHTO equations. Azizinamini et al (1996) also examined the available prestress in an existing bridge girder. These researchers investigated the effective prestress force in a 25-year-old girder. The girder was removed from a bridge on I-80 in Nebraska. The length of the girder was 54 ft. 10 in. The testing of the girder showed that the prestress loss after 25 years of service was 20.7 percent, which is less than the 25.7 percent, predicted by the AASHTO equations.

Idriss (2001) measured the prestress losses in bridge girders during construction and during service. The bridge has spans of 96 and 101 ft. The beams were cast with HPC with compressive strengths of 7330 psi at three days of age and 10,150 psi at 56 days of age. Deformation sensors were placed in the girders to measure the prestress losses. After five months of service, the measured losses were less than the losses predicted by AASHTO and PCI. The AASHTO equations predicted losses of 28 percent and the PCI equations predicted losses of 22.6, which are both greater than the measured losses of 11 percent. However, it must be noted that the losses reported are after five months service. The girders will continue to lose additional prestress force over the next five years. Many researchers have shown that the AASHTO equations overestimate the total prestress losses, but most researchers agree that more testing needs to be done before the prediction equations are modified.

6.2.2 Allowable Compressive Stress at Release

The AASHTO Bridge Specifications, the American Concrete Institute (ACI) Building Code, and PCI Design Handbook limit the allowable compressive stress imposed on the concrete by the prestress force to 60 percent of the compressive strength at release ($0.6f_{ci}$). The purpose of the limit was to control creep deformation and prevent crushing due to prestress force (Huo et al, 1995). However, researchers are examining if this limit can be raised. Raising the limit would reduce the number of unbonded strands in the end regions of girders, increase production time, increase capacity of a member, and eliminate the need for steam curing (Huo and Tadros, 1997).

There is little data available regarding the effects of increasing the allowable compressive stress at release. Even though there is minimal published data, it appears that many prestressed/precast concrete manufacturers regularly release at stresses of 75 percent of the concrete strength at release. The PCI Standard Design Practice states, "no problems have been reported by allowing compression as high as $0.75f'_{ci}$ (PCI, 1996)." There has been some work on allowable stresses done at The University of Oklahoma. Pang (1997) investigated the effects of large compressive stresses on the hardened properties of concrete. Pang conducted creep tests on concrete cylinders loaded to $0.6f_{ci}$, $0.7f_{ci}$, and $0.8f_{ci}$. He concluded that the creep at higher stress levels was "not excessive and was similar to creep experienced by concrete stressed at lower levels." Pang also concluded that allowable compressive limit could be raised to at least $0.7f_{ci}$.

Noppakunwijai (2001) conducted one of the few experimental programs that examined the effects of high release stresses on prestressed/precast girders. Two 33 ft. girders with a span to depth ratio of 32 were cast. The concrete stress at release was $0.79f_{ci}$ and $0.84f_{ci}$. The prestress losses of the girders were measured. Noppakunwijai et al (2001) concluded that higher release stresses had no negative impact on the test specimen. The amount of literature on the release stresses of prestressed concrete is very limited, particularly experimental data on bridge girders. The justification for increasing the allowable compressive stress limit at release appears to be based on common practices in the prestressed/precast concrete industry (PCI, 1996). Even though there appears to be no negative impact caused by this practice, research should be conducted to examine the effects of release stresses greater than $0.6f_{ci}$.

6.3 EXPERIMENTAL PROGRAM

6.3.1 Scope

The research program examined the prestress losses in bridge girders with and without air entrainment. The purpose of the study was to determine: (1) What the prestress losses are and (2) if there are any differences in the prestress losses of air entrained concrete when compared to non-air entrained concrete. Four prestressed girders were cast. Two girders had a targeted total air content of two percent, and two had a targeted total air content of six percent. The girders were cast using concrete mixtures developed during the second (Mixture 9-26-15/3, Table 2.11) and third (Mixture 9-26-5.9, Table 3.3) phases of the research. The measured prestress losses were compared with estimated losses calculated from the AASHTO Guidelines and the PCI equations to determine the accuracy of the prediction equations.

The research program also examined the allowable compressive stresses at release for prestressed beams. Currently the AASHTO Bridge Design Specifications permits the concrete compressive stress at release to be 60 percent or less of the concrete compressive strength at release ($0.60f_{ci}$). Researchers are investigating whether this limit can be increased beyond $0.60f_{ci}$. The research results will provide additional data to examine the efficacy of various proposals. Two of the beams had a targeted release stress of 60 percent, and two of the beams had a targeted release stress of 75 percent. The research program is shown in Table 6.1.

Table 6.1. Research Program.

Beam	Targeted Allowable Compressive Stresses (f_{bot} / f_{ci})		Targeted Total Air Content (%)	
	0.60	0.75	2	6
1	-	X	X	-
2	-	X	-	X
3	X	-	-	X
4	X	-	X	-

6.3.2 Materials

Ash Grove Type III Cement was used in all mixtures. The coarse aggregate was 3/8 inch, crushed limestone from Davis, Oklahoma. The fine aggregate was washed river sand from Dover, Oklahoma. The material properties of the cement and aggregates are listed in Chapter 2. To provide adequate workability, a water reducer (WR) and/or a high range water reducer (HRWR) was used. The WR used was WRDA with Hycol, and the HRWR used was ADVA Flow. An air entraining agent (AEA), DARAVAIR 1000, was used to attain the required target total air contents. The girders contained 0.60 in. diameter, low relaxation prestressing strand and Grade 60 mild reinforcement.

6.3.3 Mixtures

The mixtures used to cast the girders were developed in the first and second phases of the research program (Chapters 2 and 3). The mixtures were chosen based on workability and strength. The chosen mixtures are shown in Table 6.2. Girders 1 and 4 were cast with the same mixture, and girders 2 and 3 contained the same mixture. The only difference in the two mixtures was the targeted total air content and quantity of sand. Girders 1 and 4 had a targeted total air content of two percent, and girders 2 and 3 had a targeted total air content of six percent.

Table 6.2. Mixture Designs.

	Girder 1	Girder 2	Girder 3	Girder 4
Cement (lb/yd^3)	900	900	900	900
Coarse Aggregate	1790	1790	1790	1790
Fine Aggregate	1217	1040	1040	1217
Water	254	254	254	254
w/cm	0.26	0.26	0.26	0.26
Targeted Total Air Content (%)	2.0	6.0	6.0	2.0
Calculated Unit Weight (lb/ft^3)	153.4	146.8	146.8	153.4

6.3.4 Girder Design

Four prestressed girders were cast as part of the research program. The girders were cast at Coreslab Structures, Inc, of Oklahoma City. All four girders had the same cross-sectional properties. The girders had a depth of 24 inches and a length of 24 feet.

The moment of inertia and area of the girders were 12,370 in^4 and 163.25 in^2, respectively. The dimensions and cross-sections of the girders are shown in Figures 6.1 through 6.4.

Girders 1 and 2 were designed to have allowable compressive stresses at release of $0.75f_{ci}$. Girders 3 and 4 were designed to have allowable compressive stresses at release of $0.60f_{ci}$. All four girders contained 10 prestressing strands, however some of girders contained debonded strands. Since Girders 1 and 4 required the same concrete but different compressive stresses at release, two strands in Girder 4 were debonded to lower the compressive stresses at release. Girder 1 contained 10 bonded prestressing strands, and Girder 4 contained eight bonded strands. Girders 2 and 3 also required some debonding to achieve the desired compressive stresses at release. Girder 2 contained nine bonded prestressing strands and one unbonded strand, whereas Girder 3 contained eight bonded strands and two unbonded strands. Girders 1 and 4 contained no air entrainment, while Girders 2 and 3 were targeted to have a total air content of six percent.

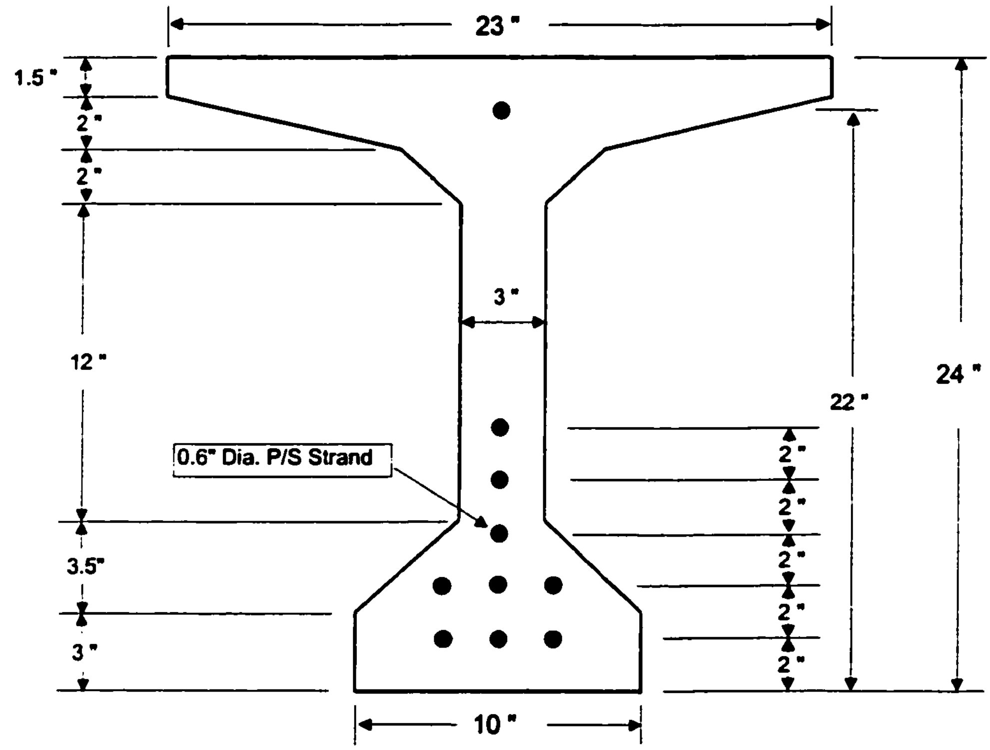

Figure 6.1. Cross-section of Girder 1.

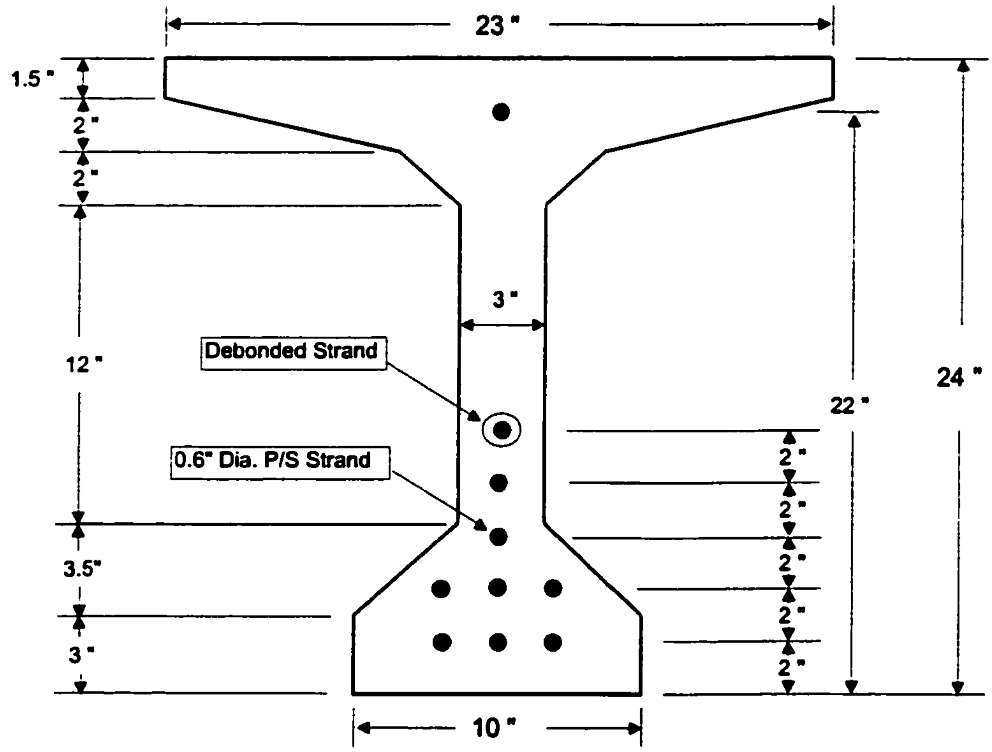

Figure 6.2. Cross-section of Girder 2.

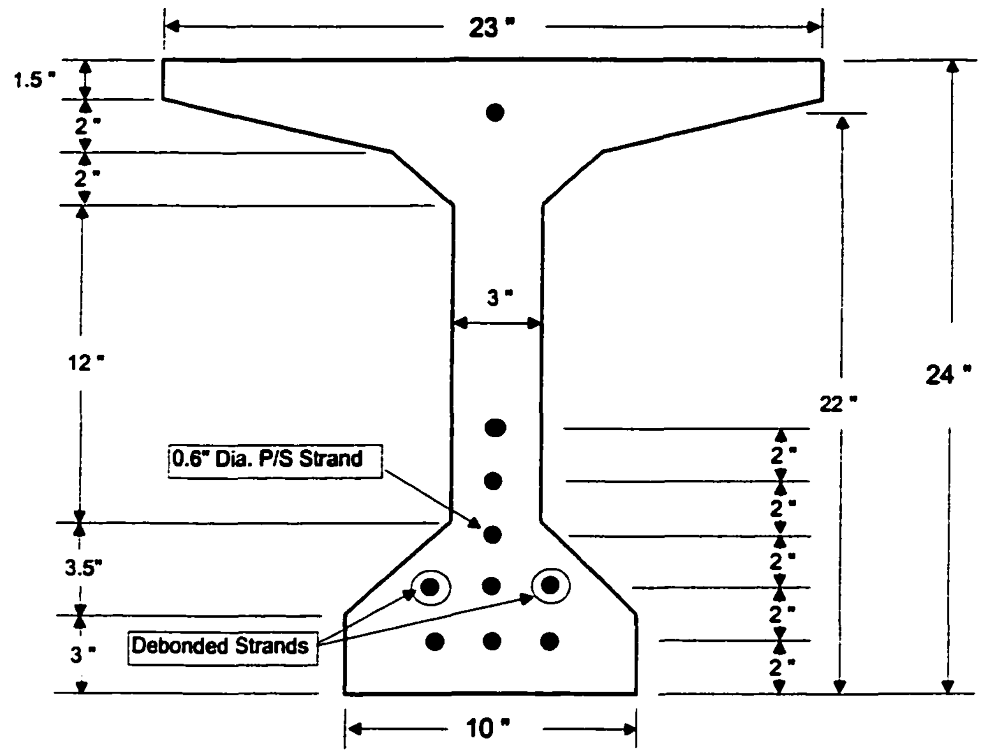

Figure 6.3. Cross-section of Girder 3.

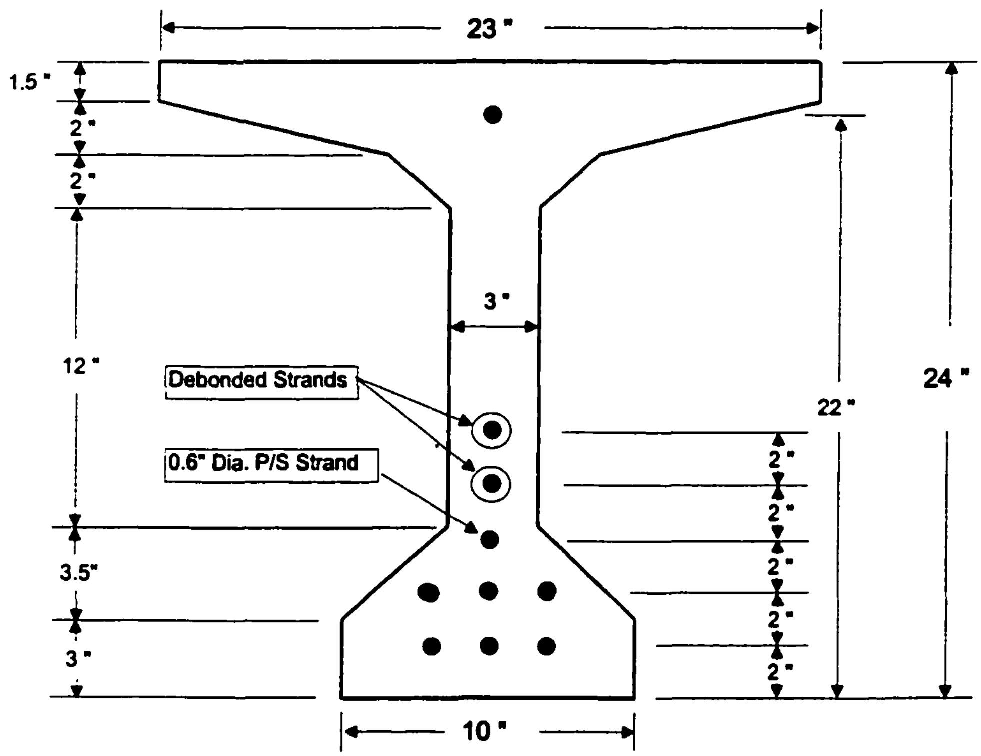

Figure 6.4. Cross-section of Girder 4.

6.3.5 Batching

As mentioned earlier, the mixture designs for the girders were chosen from earlier phases of the research program. The mixtures were chosen based on workability and strength. Prior to the casting of the girders, several trial batches were cast at Coreslab Structures in Oklahoma City. The selected mixtures were developed at Fears Structural Engineering Laboratory (FSEL) using a 6-ft^3 rotary concrete mixer, but at Coreslab a much larger mixer (4 yd^3) was used to batch the concrete. The trial batches were necessary to determine appropriate dosages of water reducers and air entraining agents. Shown in Table 6.3 is a comparison of mixture proportions (particularly admixture dosages) between the mixtures cast at FSEL and the mixtures cast at Coreslab. The main difference between the mixtures cast at FSEL and at Coreslab was the dosage of AEA. At FSEL, a dosage of 15 fl oz/cwt was required to attain approximately six percent air, but at Coreslab only 0.65 fl oz/cwt was necessary. Higher dosages of AEA were required at FSEL because the smaller mixer was not as efficient in agitating the fresh concrete as well as the larger mixer used by Coreslab.

Table 6.3. Comparison of FSEL and Coreslab Mixtures.

	FSEL	Coreslab	FSEL	Coreslab
Cement (lb/yd^3)	900	900	900	900
Coarse Aggregate	1790	1790	1790	1790
Fine Aggregate	1217	1217	1040	1040
Water	254	254	254	254
w/cm	0.26	0.26	0.26	0.26
Targeted Total Air Content (%)	2.0	2.0	6.0	6.0
Calculated Unit Weight (lb/ft^3)	153.4	153.4	146.8	146.8
WR (fl oz/cwt)	3	3	3	3
HRWR (fl oz/cwt)	15	18	15	18
AEA (fl oz/cwt)	0	0	15	0.65

6.3.6 Instrumentation

After the girders were cast and the forms removed, detachable mechanical strain gauge (DEMEC) targets were glued onto the beams. The DEMEC targets were placed along the side of the bulbs and along the top of the girders. The targets were placed 60, 68, and 76 inches from each end and also at 144, 152, and 160 inches. The placement of the targets along the bulb of the girders is shown in Figure 6.5. The distances between the DEMEC targets were measured using a DEMEC dial gauge. A close-up view of the targets along the bulb and on the top of the girders is shown in Figures 6.6 and 6.7, respectively. Readings were taken before the cutting of the prestressing strands, and immediately (within one to two hours) after cutting the strands. By evaluating the changes in length between DEMEC targets, the concrete strain was calculated.

Measurements were taken periodically until December 2001 and at that time the beams were one year old.

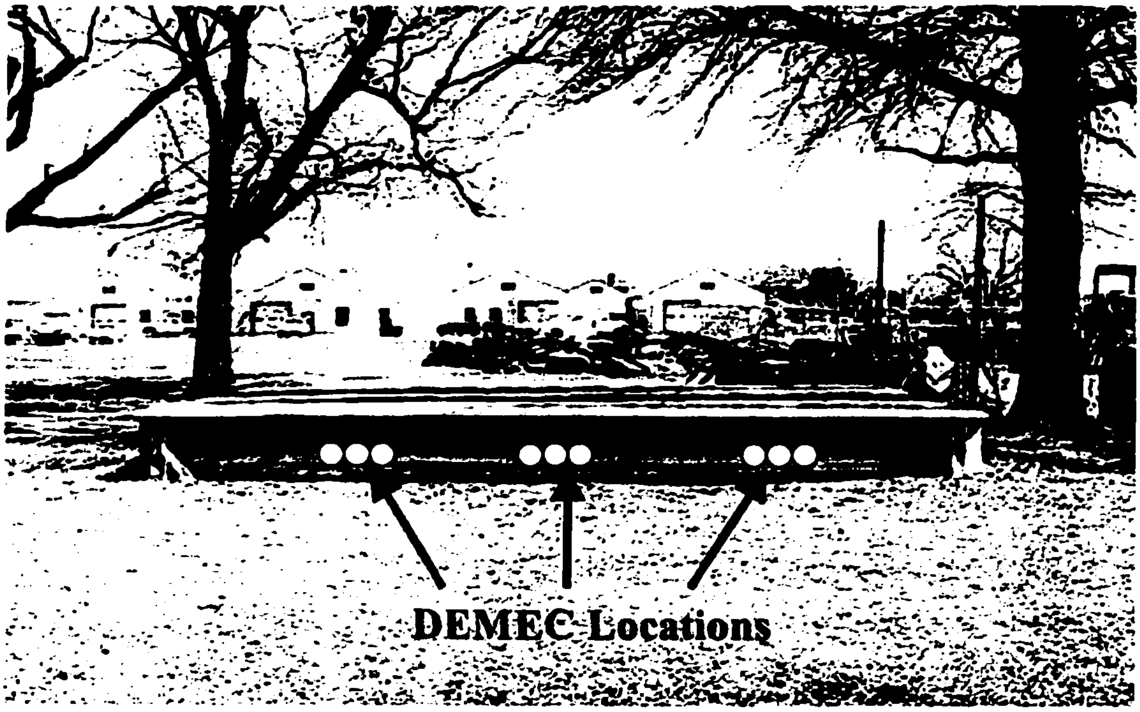

Figure 6.5. DEMEC Locations Along the Side of the Girders.

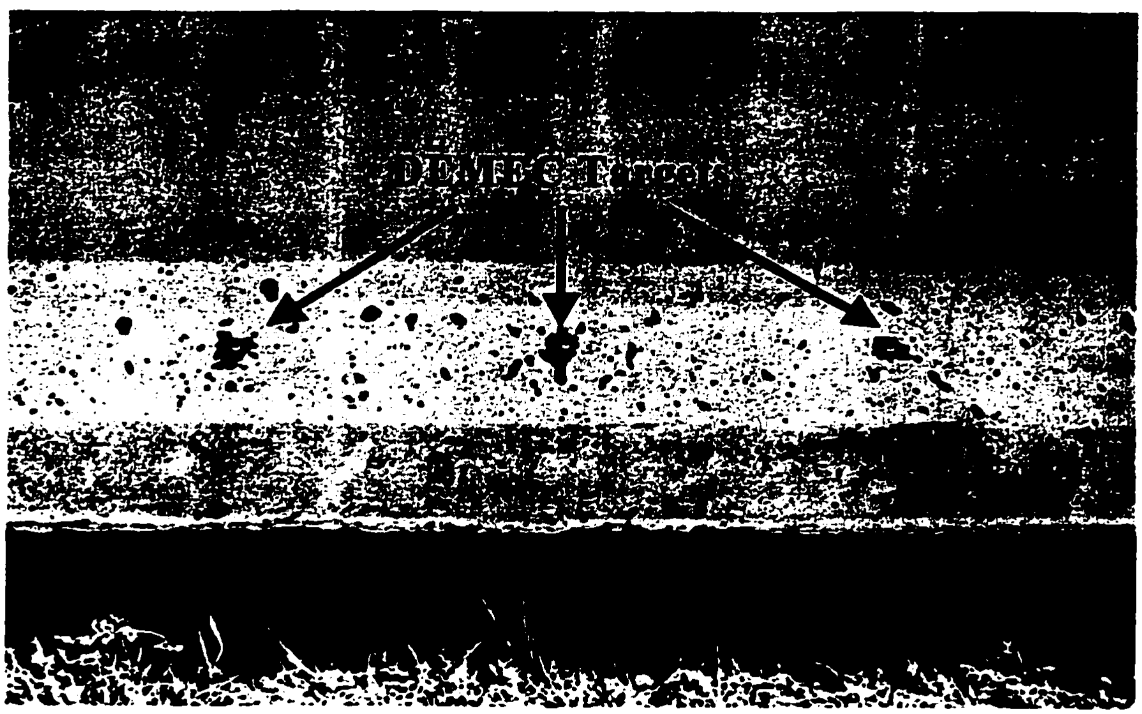

Figure 6.6. DEMEC Targets on the Bulb of the Girder.

Figure 6.7. DEMEC Targets on the Top of the Girder.

6.3.7 Tests

The two mixtures were subjected to several tests. The fresh concrete properties tested were slump (ASTM C 143), unit weight (ASTM C 138), and air content (ASTM C 231). The hardened properties tested were compressive strength (ASTM C 39), length change (ASTM C 157), and modulus of elasticity (ASTM C 469). The prestress losses were also measured for a period of one year.

Earlier research (Johnson 2001) had examined the creep (ASTM C 512) and shrinkage of the mixtures. To examine the creep of the mixtures, five 4 x 10 in. concrete cylinders were loaded in a test frame. The 4 x 10 in. cylinders were chosen for the creep test (instead of the standard 4 x 8 in. cylinders) because the larger length allowed for the use of an eight-inch gauge length versus a four-inch gauge length. Prior research at

FSEL had shown that an eight-inch gauge length produced more consistent data than the four-inch gauge lengths. Prior to loading, DEMEC targets were glued onto the front and rear of each cylinder. The targets were placed one inch from the top and bottom of the cylinders. Using a hydraulic pump, the cylinders were loaded to 45 percent of their one-day compressive strength. DEMEC readings were taken before and after loading. Readings were also taken periodically over a period of 56 days as described by ASTM C 512. A diagram of the creep test set up is shown in Figure 6.8.

The shrinkage of the mixtures was also measured using the 4 x 10 inch cylinders. Companion, unloaded cylinders were stored with the creep specimens. These specimens also had DEMEC targets attached in the same positions as the creep specimens. Readings were taken on the shrinkage specimens at the same time as the creep specimens. The shrinkage specimens were used to determine the creep of the concrete. The amount of shrinkage was subtracted from the change in length of the loaded cylinders. The difference between the two readings is the change in length due only to creep.

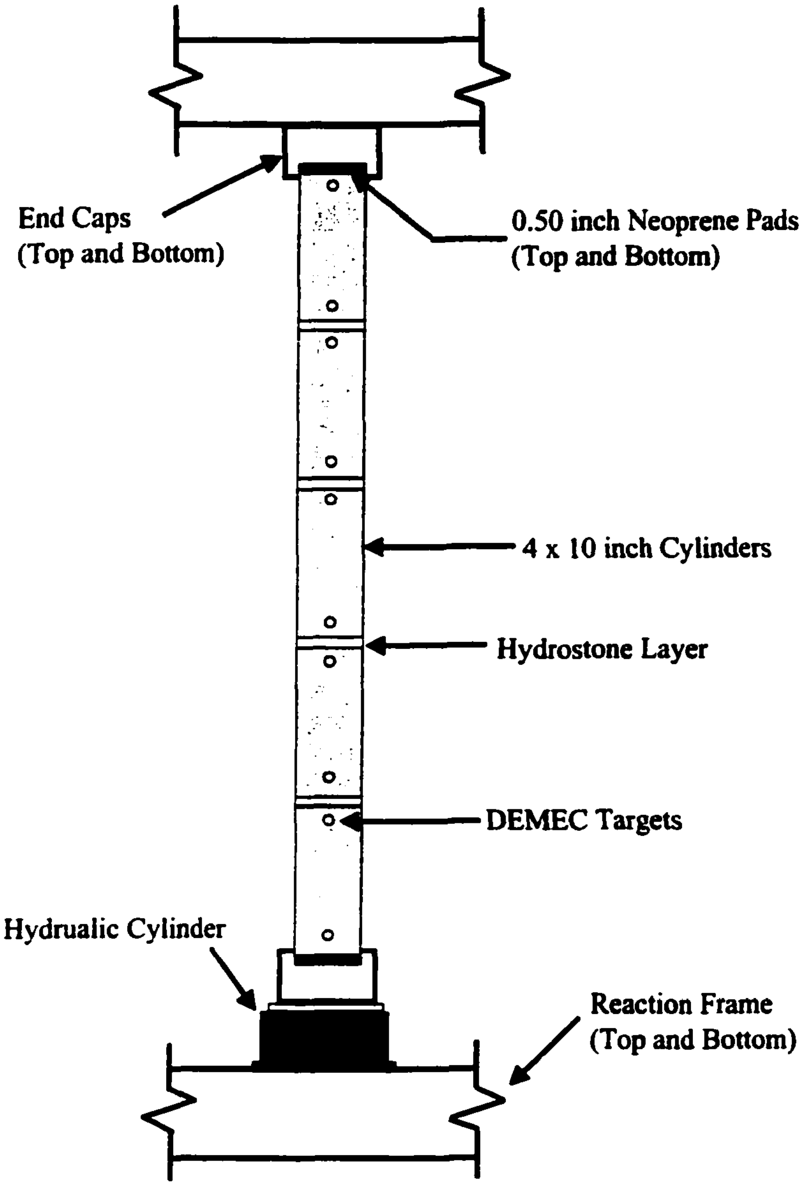

Figure 6.8. Creep Test Set-up.

6.4 PRESENTATION OF RESULTS

6.4.1 Concrete and Girder Properties

The results from the fresh concrete tests are shown in Table 6.4. Also shown in the table are the fresh concrete properties of identical mixtures cast at FSEL. The compressive strength and modulus of elasticity of the girders are shown in Table 6.5. The creep results from Johnson's research are shown in Table 6.6. A comparison of the compressive strength between the FSEL mixtures and the Coreslab mixtures is shown in Table 6.7. The compressive strength, modulus of elasticity, and shrinkage reported are the averages of at least three individual tests. The individual tests and statistical data are reported in Appendix E.

The measured prestress losses at the north and south ends of the girders are shown in Tables 6.8 and 6.9, respectively. The average losses of both ends of the girders are reported in Table 6.10. The measured prestress losses at the center of the girders are listed in Table 6.11. All prestress losses reported are the losses at the center of gravity for the prestressing strand. The losses are the average of four readings (two reading from each side of the girder). Also in the tables are the locations where the girders were stored. The girders were stored outside at Coreslab (superscript 1), inside at FSEL (superscript 2), and outside at FSEL (superscript 3).

Table 6.4. Fresh Concrete Properties.

Fresh Concrete Properties	Girders 1 & 4	9-26-2.4	Girders 2 & 3	9-26-5.6
Batch Location	Coreslab	FSEL	Coreslab	FSEL
Fresh Concrete Temp. (F)	68	74	60	75
Slump (in.)	10.0	6.50	9.75	4.25
Air Content (%)	2.3	2.4	6.2	5.6
Unit Weight (lb/ft^3)	151.1	152.3	146.9	147.2
Air Temperature (F)	36	91	35	91
Date Batched	12/19/00	8/2/99	12/21/00	7/29/00

Table 6.5. Hardened Concrete Properties of the Girders.

	Girder 1	Girder 2	Girder 3	Girder 4
Average Compressive Strength (psi)[1]				
1 day	8700	6130	6130	8700
14 days	10,190	7110	7110	10,190
28 days	11,060	8390	8390	11,060
56 days	12,440	9200	9200	12,440
180 days	14,460	10,850	10,850	14,460
360 days	15,610	11,460	11,460	15,610
Average Modulus of Elasticity (ksi)[1]				
1 day	5600	4700	4700	5600
14 days	5800	4900	4900	5800
28 days	6000	5500	5500	6000
56 days	6300	5400	5400	6300
180 days	6800	5500	5500	6800
360 days	6900	6000	6000	6900

1. Reported values are the average of three tests.
2. Individual tests and statistical data are located in Appendix E.

Table 6.6. Creep Results from Johnson.

Age at Testing (days)	Creep Strain (10^{-6} in./in.)	
	9-26-2.4	9-26-5.6
2	206.69	241.11
3	254.75	291.06
4	316.17	303.75
5	333.32	311.58
6	341.01	326.27
7	357.21	336.15
14	402.17	410.13
21	433.62	454.14
28	446.04	463.46
56	534.06	506.52

Table 6.7. Compressive Strength of FSEL and Coreslab Mixtures.

	Beams 1 & 4	9-26-2.4	Beams 2 & 3	9-26-5.6
Batch Location	Coreslab	FSEL	Coreslab	FSEL
1 day (psi)	8700	6830	6130	4560
7 day	-	11,040	-	9270
14 day	10,190	-	7110	-
28 day	11,060	12,190	8390	10,560
56 day	12,440	12,960	9200	11,290

Table 6.8. Measured Prestress Losses at the North End.

Beam Age (days)	Temperature (F)	Prestress Losses at the North End (ksi)[4]			
		Girder 1	Girder 2	Girder 3	Girder 4
		Air = 2.3%	Air = 6.2%	Air = 6.2%	Air = 2.3%
		f_{bot}/f_{ci} = 66.0	f_{bot}/f_{ci} = 86.9	f_{bot}/f_{ci} = 71.4	f_{bot}/f_{ci} = 55.4
1 (at release)	36 [1]	28.98	33.88	23.70	25.29
16	40 [1]	40.65	52.05	40.84	36.33
29	65 [2]	41.32	55.00	42.38	37.22
43	65 [3]	47.34	62.90	49.18	42.43
60	51 [3]	45.93	59.31	45.66	39.75
84	63 [3]	47.99	64.22	49.31	42.47
120	71 [3]	49.70	65.62	50.25	44.28
180	90 [3]	53.91	70.35	56.93	48.22
360	63 [3]	58.22	72.97	59.46	50.81

1 = Cured outside at Coreslab Structures.
2 = Cured inside at FSEL.
3 = Cured outside at FSEL.
4 = Strain measurements are located in Appendix E.

Table 6.9. Measured Prestress Losses at the South End.

Beam Age (days)	Temperature (F)	Prestress Losses at the South End (ksi)[4]			
		Girder 1	Girder 2	Girder 3	Girder 4
		Air = 2.3%	Air = 6.2%	Air = 6.2%	Air = 2.3%
		f_{bot}/f_{ci} = 66.0	f_{bot}/f_{ci} = 86.9	f_{bot}/f_{ci} = 71.4	f_{bot}/f_{ci} = 55.4
1 (at release)	36 [1]	26.45	31.67	25.16	25.77
16	40 [1]	38.6	50.36	41.65	36.86
29	65 [2]	39.09	52.66	42.17	37.28
43	65 [3]	44.61	60.51	49.06	43.1
60	51 [3]	43.35	58.95	47.11	41.35
84	63 [3]	44.59	61.73	49.74	43.76
120	71 [3]	46.22	63.16	50.76	44.77
180	90 [3]	50.78	67.76	55.69	48.51
360	63 [3]	54.93	70.93	58.98	51.5

1 = Cured outside at Coreslab Structures.
2 = Cured inside at FSEL.
3 = Cured outside at FSEL.
4 = Strain measurements are located in Appendix E.

Table 6.10. Average Prestress Losses at Both Ends.

Beam Age (days)	Temperature (F)	Average Prestress Losses at Ends (ksi)[4]			
		Girder 1	Girder 2	Girder 3	Girder 4
		Air = 2.3%	Air = 6.2%	Air = 6.2%	Air = 2.3%
		f_{bot}/f_{ci} = 66.0	f_{bot}/f_{ci} = 86.9	f_{bot}/f_{ci} = 71.4	f_{bot}/f_{ci} = 55.4
1 (at release)	36 [1]	27.72	32.78	24.43	25.53
16	40 [1]	39.63	51.21	41.25	36.60
29	65 [2]	40.21	53.83	42.28	37.25
43	65 [3]	45.98	61.71	49.12	42.77
60	51 [3]	44.64	59.13	46.39	40.55
84	63 [3]	46.29	62.98	49.53	43.12
120	71 [3]	47.96	64.39	50.51	44.53
180	90 [3]	52.35	69.06	56.31	48.34
360	63 [3]	56.78	71.95	59.22	51.56

1 = Cured outside at Coreslab Structures.
2 = Cured inside at FSEL.
3 = Cured outside at FSEL.
4 = Strain measurements are located in Appendix E.

Table 6.11. Prestress Losses at Center (ksi).

Beam Age (days)	Temperature (F)	Prestress Losses at Center (ksi)[4]			
		Girder 1	Girder 2	Girder 3	Girder 4
		Air = 2.3%	Air = 6.2%	Air = 6.2%	Air = 2.3%
		f_{bot}/f_{ci} = 66.0	f_{bot}/f_{ci} = 86.9	f_{bot}/f_{ci} = 71.4	f_{bot}/f_{ci} = 55.4
1 (at release)	36 [1]	27.83	33.58	23.44	24.86
16	40 [1]	39.92	51.93	40.19	35.67
29	65 [2]	40.17	55.32	41.35	36.87
43	65 [3]	46.08	62.45	48.58	42.60
60	51 [3]	44.14	60.25	45.99	40.71
84	63 [3]	45.96	64.27	48.56	43.73
120	71 [3]	47.57	65.94	50.55	45.21
180	90 [3]	49.71	71.53	55.09	49.30
360	63 [3]	53.30	74.03	58.46	51.80

1 = Cured outside at Coreslab Structures.
2 = Cured inside at FSEL.
3 = Cured outside at FSEL.
4 = Strain measurements are located in Appendix E.

6.5 DISCUSSION OF RESULTS

6.5.1 Fresh Concrete Properties

As mentioned in earlier chapters, one goal of the research was to develop a HPC mixture that could be easily used in the local prestressed industry. The slumps of the mixtures used in casting the girders were 9.75 and 10 inches. Both mixtures had sufficient workability to produce the girders. As can be seen in Figures 6.9 and 6.10, the girders had a smooth finish and there were no "honeycombed" areas.

Girders 1 and 4 had a targeted air content of two percent, and Girders 2 and 3 had a targeted air content of six percent. Both mixtures were within ± 0.50 percent of the targeted air contents. The first batch of concrete for Girders 2 and 3 had to be discarded due to an air content of 9.6 percent. The unit weights of the mixtures inversely correlated to the air contents of the mixtures. The mixture for Girders 1 and 4 had a higher unit weight than the concrete mixture for Girders 2 and 3, which was expected.

The unit weights of the mixtures cast at FSEL and at Coreslab are shown in Table 6.12. The calculated unit weights and the percent difference between the calculated and measured unit weights are also shown in Table 6.12. All mixtures were within 1.5 percent of the calculated unit weight. The difference between the measured and calculated unit weights was considered acceptable. This is a testament to the quality control procedures employed by Coreslab. The mixtures cast at FSEL were smaller mixtures (approximately 2.2 ft^3) and the materials were weighed using a digital scale. The materials were weighed to the nearest hundredth of a pound. At Coreslab, the mixtures were much larger (approximately 2.5 yd^3 or 67.5 ft^3) and the materials were weighed using a weight and balance system and mixed in a concrete mixer that had been in use since the 1926. The minimal differences between the measured and calculated unit weights (1.5 percent and less) and the use of a mixer that was built in 1926 confirms that HPC can be produced at most facilities that are currently casting concrete.

Table 6.12. Comparison of Unit Weights.

Location	9-26-2.4	$\frac{\text{Measured}}{\text{Calculated}}$	9-26-5.6	$\frac{\text{Measured}}{\text{Calculated}}$
Unit Weight - Coreslab (lb/ft^3)	151.1	98.5 %	146.9	100.1 %
Unit Weight – FSEL (lb/ft^3)	152.3	99.3 %	147.2	100.3 %
Unit Weight – Calculated (lb/ft^3)	153.4	100.0 %	146.8	100.0 %

Figure 6.9. Photograph of Girders.

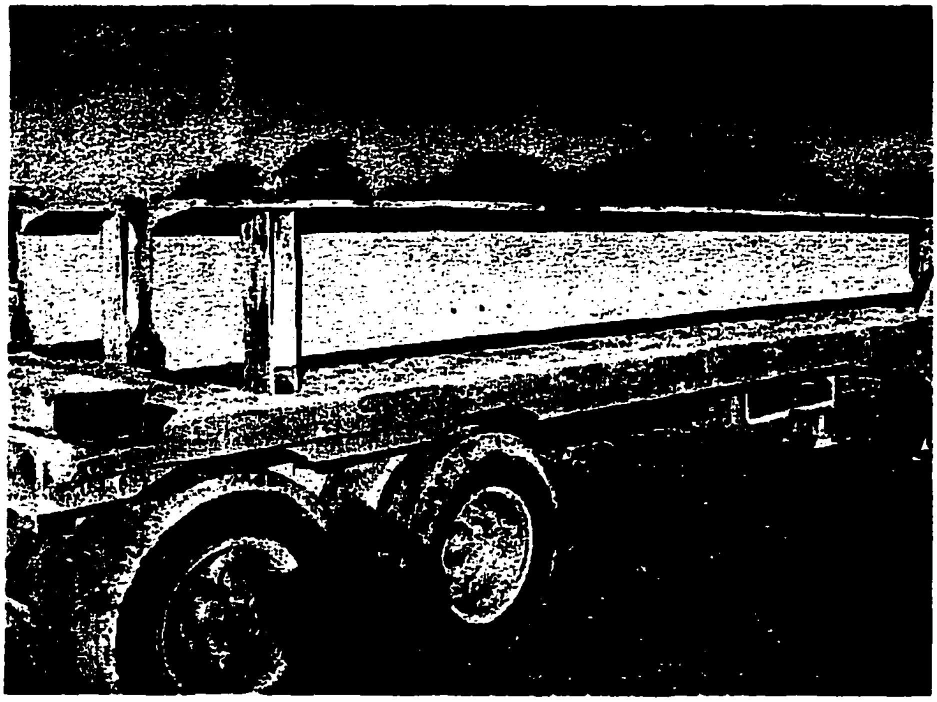

Figure 6.10. Photograph of Girders.

6.5.2 Hardened Concrete Properties

6.5.2.1 Compressive Strength. A comparison of the FSEL and Coreslab mixtures are shown in Figures 6.11 and 6.12. The difference between the mixtures was the curing regimen. The cylinders cast at Coreslab were steam cured for approximately 12 hours (overnight) with the girders and then cured with the girders outside in ambient temperatures. The FSEL cylinders were cured at 73°F and 50 percent relative humidity for 24 hours and then cured at 73°F and 100 percent relative humidity until tested. The use of heat curing increased the one-day compressive strengths of the Coreslab mixtures, but decreased their long term compressive strength, which is shown in Figures 6.11 and 6.12. Steam curing increases early age strength by increasing the initial hydration. This results in a "less uniform distribution of hydration products within the paste" which decreases the ultimate strength of concrete (Mindess et al, 1981). This is evident in

Figure 6.11 and 6.12 where the compressive strength of the Coreslab mixtures was approximately 30 percent greater than the FSEL mixtures at one day of age, but by 28 days of age, the FSEL achieved compressive strengths that were greater than the steam cured Coreslab mixtures.

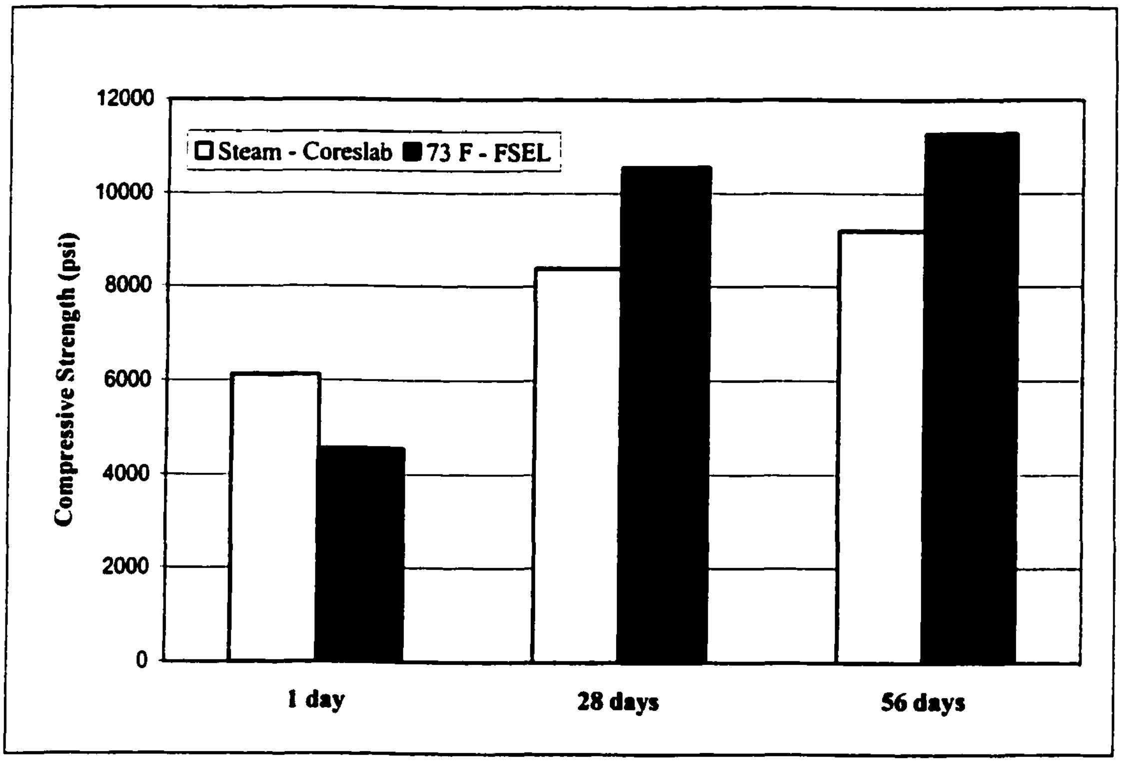

Figure 6.11. Comparison of One-day Compressive Strengths (psi).

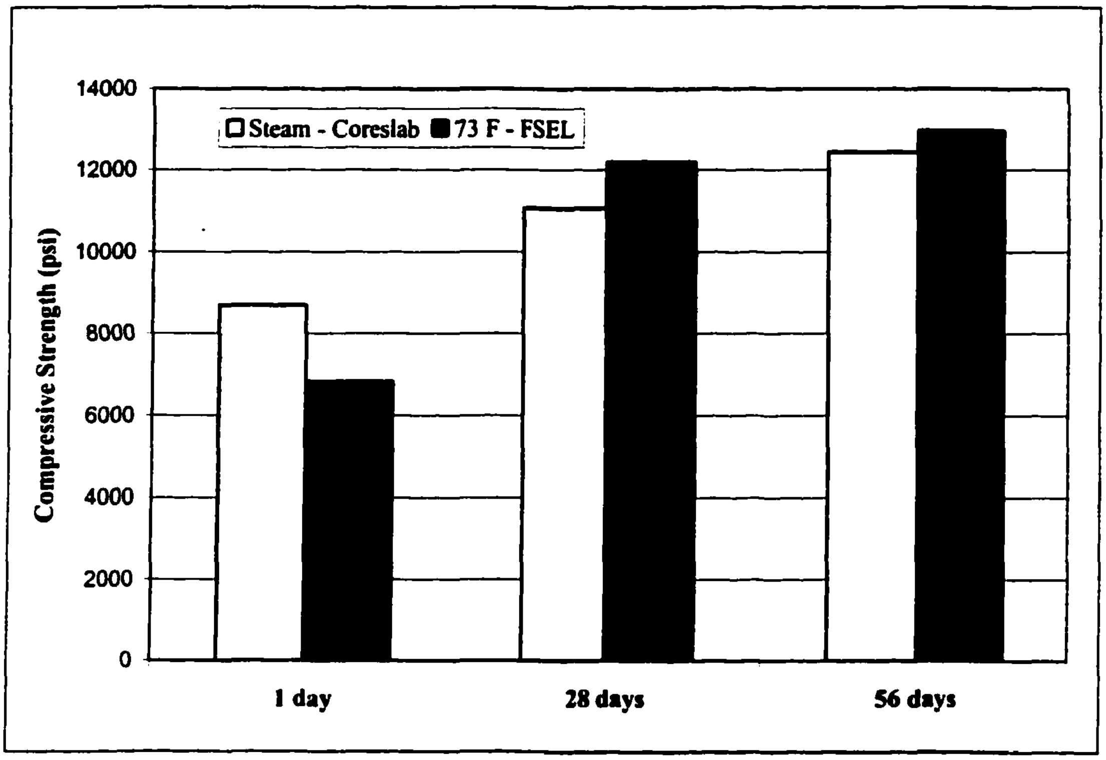

Figure 6.12. Comparison of 28-day Compressive Strengths (ksi).

The strength gain of the two mixtures cast at Coreslab is shown in Figure 6.13. The difference between the two mixtures is the total air content. One mixture contained no entrained air (air content of 2.3 percent) and the other contained 6.2 percent air (approximately 4 percent entrained air). The effects of the entrained air are shown in Figure 6.13. The rate of strength gain for the two mixtures is almost identical, but for each age tested the addition of entrained air reduced the compressive strength by roughly 30 percent.

The decrease in compressive strength due to the addition of air entrainment was discussed Chapter 3. The addition of one percent of entrained air normally decreases the compressive strength by two to five percent (Mindess et al, 1981). The decrease in compressive strength per increase in entrained air is shown in Table 6.13 for each age tested. The decrease in compressive strength per one percent increase in entrained air ranged from 6.2 to 7.8 percent. This is slightly more than the "2 to 5 percent decrease" suggested by many books, but still within an acceptable and expected range.

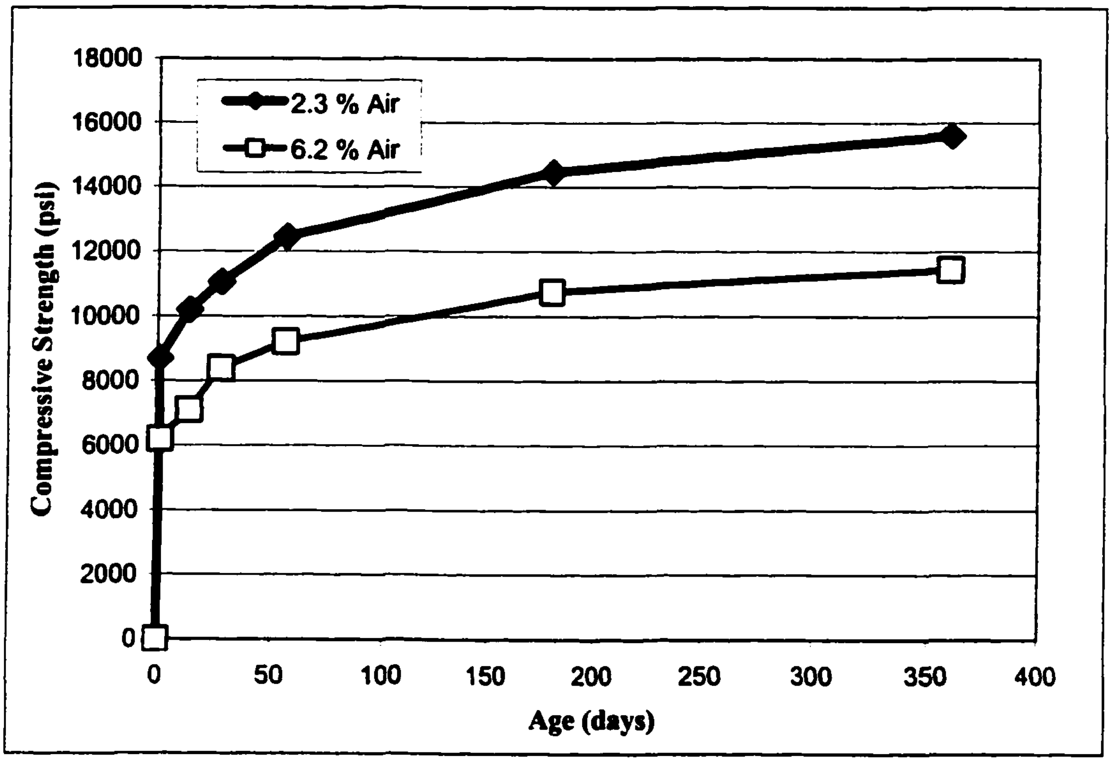

Figure 6.13. Strength Gain for the Coreslab Mixtures.

Table 6.13. Decrease in Compressive Strength due to Entrained Air.

Age (days)	Girder 1	Girder 2	% Decrease in Compressive Strength per One % Increase in Entrained Air
1	8700	6130	7.60
14	10,190	7110	7.80
28	11,060	8390	6.20
56	12,440	9200	6.70
180	14,460	10,850	6.40
360	15,610	11,460	6.80

6.5.2.2 Modulus of Elasticity. The modulus of elasticity (MOE) is a measure of the stiffness of the concrete. The MOE of concrete is seldom measured for design; instead ACI318-99 provides two prediction equations (Eqns. 6.1 and 6.2) that can be used to determine the MOE. As can be seen in the equations, the compressive strength and unit weight are the major factors that influence the MOE of concrete. Increases in both compressive strength and unit weight result in higher MOE. These trends exist in the results of the mixtures examined in this research. Increases in compressive strength resulted in increases in MOE.

The MOE of the mixtures are plotted in Figure 6.14 versus the square root of the compressive strength. Also plotted on the graph are equations 6.1 and 6.2. As shown in Figure 6.14, the MOE of the mixtures plot near the prediction equations, meaning the equations can be used when estimating the MOE based on their compressive strength.

$E_c = 57{,}000*\sqrt{f'_c}$ Eqn. 6.1

Where
f'_c = compressive strength (lb/in^2)

$E_c = 33w_c^{1.5}*\sqrt{f'_c}$ Eqn. 6.2

Where
f'_c = compressive strength (lb/in^2)
w_c = unit weight of the concrete (lb/ft^3)

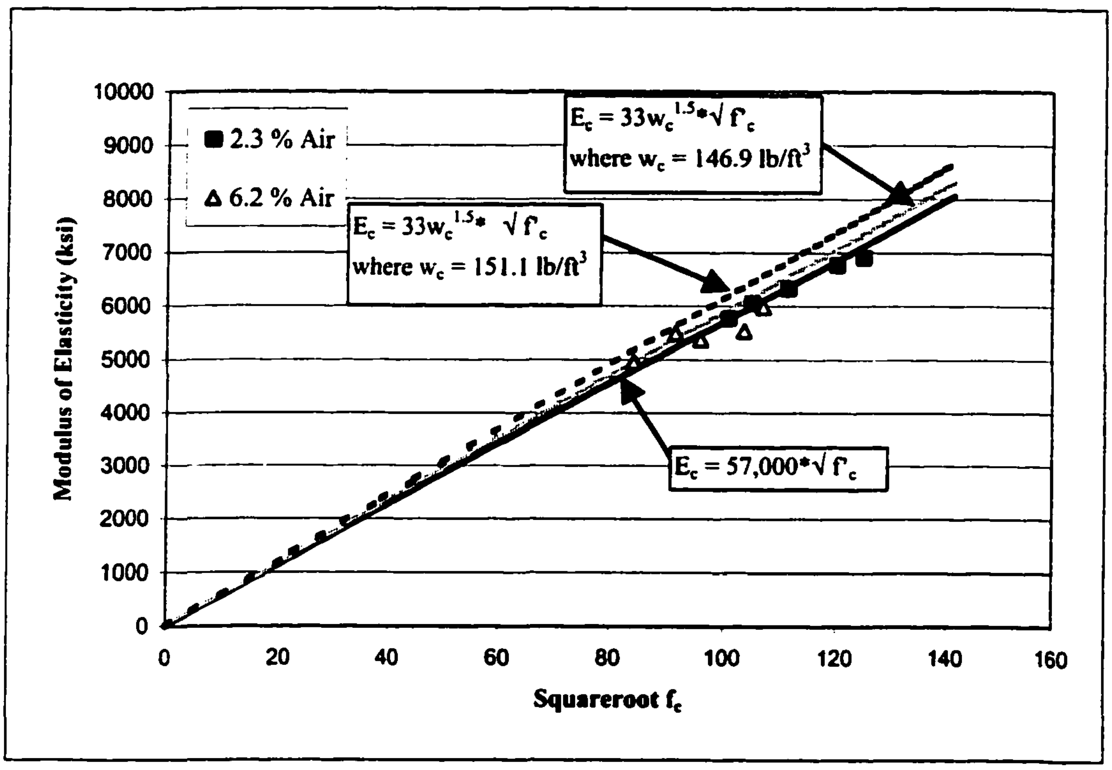

Figure 6.14. MOE and Prediction Equations.

6.5.2.3 Creep. Earlier research by Johnson (2001) examined the differences between the creep of air entrained and non-air entrained mixtures. Creep is the deformation of concrete due to sustained loads. The creep of concrete is affected by the strength, age at loading, curing conditions, aggregate content, and modulus of elasticity of the concrete mixture (Mindess et al, 1981). The mixtures examined by Johnson were the identical to the mixtures (mixtures 9-26-2.3 and 9-26-6.2) used to cast the girders for this research.

The results of Johnson's research are shown in Figure 6.15. As shown in the graph, the addition of entrained air has little or no effect on the creep of concrete, which was expected. Both mixtures were cured at the same temperature and relative humidity, and the quantity of coarse aggregate was also equal in both mixtures. Even though the compressive strengths were different, both mixtures were loaded to 40 percent of their one-day compressive strength. Johnson's results show that the creep of concrete is not directly affected by the addition of entrained air. The creep is indirectly affected by entrained air due to the reduction of strength caused by the addition of entrained air. The results of Johnson's research show that air entrained and non-air entrained prestressed girders will exhibit similar creep behavior.

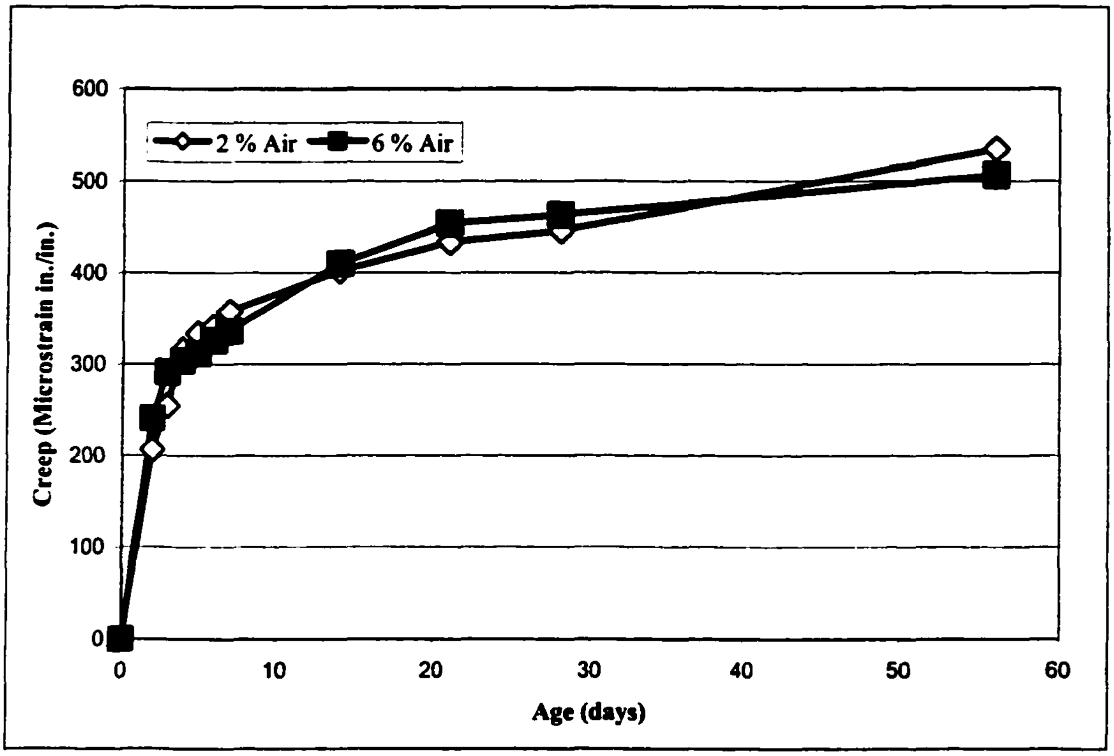

Figure 6.15. Creep Results from Johnson's Research.

6.5.2.4 Shrinkage. The shrinkage or length change of the mixtures used to cast the girders was measured for one year. Shrinkage specimens were cast from the same concrete mixtures that were used in casting the girders (mixtures 9-26-2.3 and 9-26-6.2). The shrinkage curves for the mixtures are shown in Figure 6.16.

Shrinkage is caused by the loss of moisture in the concrete. The major contributors to shrinkage are the w/cm, the volume of paste, and the volume and type of coarse aggregate. Since the paste content was approximately the same for both mixtures and the coarse aggregate was the same for both mixtures, the shrinkage would also have been expected to be approximately the same. As shown in Figure 6.16, the shrinkage curves for the two mixtures were similar. The shrinkage at one year of age was 680 and 740 microstrains for mixtures 9-26-2.3 and 9-26-6.2, respectively. The results are typical of concrete mixtures containing high quantities of cement (Ramseyer, 1999). Like the creep results, the results from the shrinkage tests were also used in the analysis of the prestress losses.

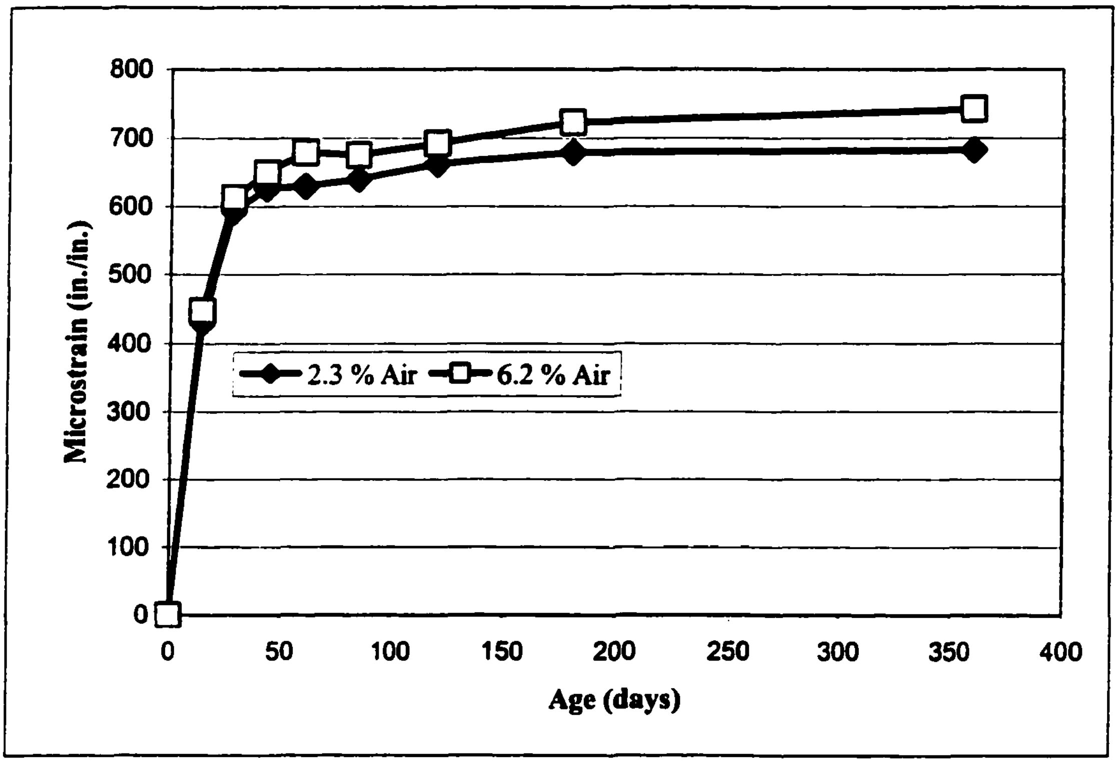

Figure 6.16. Shrinkage Curves.

6.5.3 Prestress Losses

6.5.3.1 Elastic Shortening. Elastic shortening is the loss of prestress force that occurs when the strands are released (at transfer of the prestress force). At transfer, the strands are cut and this force, which is applied to the concrete through the strands, shortens the concrete. The strand length also shortens as the length of concrete shortens thereby reducing the initial prestress force.

The loss due to elastic shortening was determined by measuring the distance between DEMEC points before transfer and after transfer of the prestress force. The elastic shortening was also calculated using equations developed by AASHTO and PCI (equations 6.3 and 6.4). The AASHTO and PCI equations are almost identical. The difference being that the concrete stress due to the prestressing strands is multiplied by the Kcir factor in the PCI equation. The Kcir factor is equal 0.90 for pretensioned girders. This term represents a 10 percent loss of prestress force due to elastic shortening. Researchers suggest that the 10 percent loss assumption will provide sufficient accuracy for most prestensioned members (Lin and Burns, 1981). For the calculations in this research, a "modified" Kcir factor was used to reflect the actual losses due to elastic shortening. The "modified" Kcir factor was determined through an iterative process. The "modified" factors ranged from 0.89 to 0.91. The calculations for the Kcir factors and the elastic shortening losses are shown in Appendix E.

The measured and predicted values are shown in Table 6.14. The two prediction equations produced results that were very similar which was expected. The calculations for the elastic shortening losses are shown in Appendix E. For each beam, the AASHTO equations predicted elastic shortenings losses that were approximately two ksi (or 11 percent) greater than the PCI equations. Once again, this is due to the addition of the Kcir factor in the PCI equations, which reduces the concrete stress by approximately 10 percent.

Table 6.14. Prestress Losses Due to Elastic Shortening.

Girders	Location	Prestress Losses Due to Elastic Shortening (KSI)		
		Measured	AASHTO Predicted	PCI Predicted
1	Ends	27.72	23.67	20.35
	Center	27.83	23.22	19.91
2	Ends	32.78	26.30	22.09
	Center	33.58	25.75	21.54
3	Ends	24.43	20.73	18.24
	Center	23.44	20.25	17.77
4	Ends	25.53	20.41	17.96
	Center	24.86	19.93	17.48

However, both equations underestimated the elastic shortening losses (Figure 6.17). The AASHTO equations predicted losses that were four to eight ksi (15 to 30 percent) less than the measured losses whereas as the PCI equations predicted losses which were six to ten ksi (or 30 to 45 percent) less. The losses for Girders 1, 3, and 4 were approximately four ksi greater than the losses predicted by the AASHTO equations and seven ksi greater than the losses predicted by the PCI equations. The elastic shortening losses for Girder 2 were seven and 10 ksi greater than those predicted by the AASHTO and PCI equations, respectively. The higher than expected measured elastic shortening losses in Girder 2 can be attributed to high compressive stress at release of $0.87f_{ci}$. At approximately 70 percent of the ultimate strength, the stress-strain curve of concrete begins to "bend over" (Neville, 1997). At stresses greater than 70 percent of ultimate, the slope of the stress-strain curves begins to decreases, which reduces the modulus of elasticity. Girders 2 had a release stress of 0.87fci. At this level of stress the modulus of elasticity is most likely less than the measured values (4700 ksi at one day) used in the calculations. The lower modulus of elasticity would increase the elastic shortening losses predicted by both PCI and AASHTO equations which would bring the predicted and measured values closer.

The differences between the measured and predicted elastic shortening losses might also be due to the changes in concrete temperature between readings (before and after transfer). The thermal expansion for the concrete and strand was taken as 6×10^{-6} in./in./degree F (MacGregor, 1999). When the strands were released, the morning temperatures were near 15°F with daily high temperatures of approximately 45°F. The temperature of the girders was approximately 100°F when the forms were removed and the DEMEC points were being glued onto the girders. A 20 °F change in concrete temperature would result in a loss of 3.40 ksi and a 30 °F difference in concrete temperature would result in a loss of 5.10 ksi. The potential losses due to changes in concrete temperature lower the measured elastic shortening losses into the range of losses predicted by the AASHTO equations. The calculations for the losses due to temperature changes are also shown in Appendix E.

One can not be certain as to why the measured losses were greater than those predicted by AASHTO and PCI. However, the changes in concrete temperature and the allowable compressive stress at release greater than $0.60f_{ci}$ contributed to the greater than expected elastic shortening losses.

$\Delta f_{pES} = (E_p/E_{ci})f_{cgp}$ **(AASHTO)** Eqn. 6.3

where;

E_p = modulus of elasticity of prestressing steel (ksi)

E_{ci} = modulus of elasticity of concrete at transfer (ksi)

f_{cgp} = sum of concrete stresses at the center of gravity of prestressing tendons due to the prestressing force at transfer and the self-weight of the at the sections of maximum moment (ksi)

$ES = K_{es}E_s f_{cir}/E_{ci}$ **(PCI)** Eqn. 6.4

where;

K_{es} = 1.0 for pretensioned members

E_s = modulus of elasticity of prestressing steel (ksi)

E_{ci} = modulus of elasticity of concrete at transfer (ksi)

f_{cir} = net compressive stress in concrete at the center of gravity of tendons immediately after the prestress has been applied to the concrete (ksi).

$f_{cir} = K_{cir}\,(P_i/A_g + P_i e^2/I_g) - M_g e/I_g$

where:

K_{cir} = 0.9 for pretensioned members

P_i = initial prestress force

e = eccentricity of center of gravity of tendons with respect to center of gravity of concrete at the cross section considered.

A_g = area of gross concrete section at the cross section considered

I_g = moment of inertia of gross concrete section at the cross section considered

M_g = bending moment due to dead weight of prestressed member and any other permanent loads in place at time of prestressing

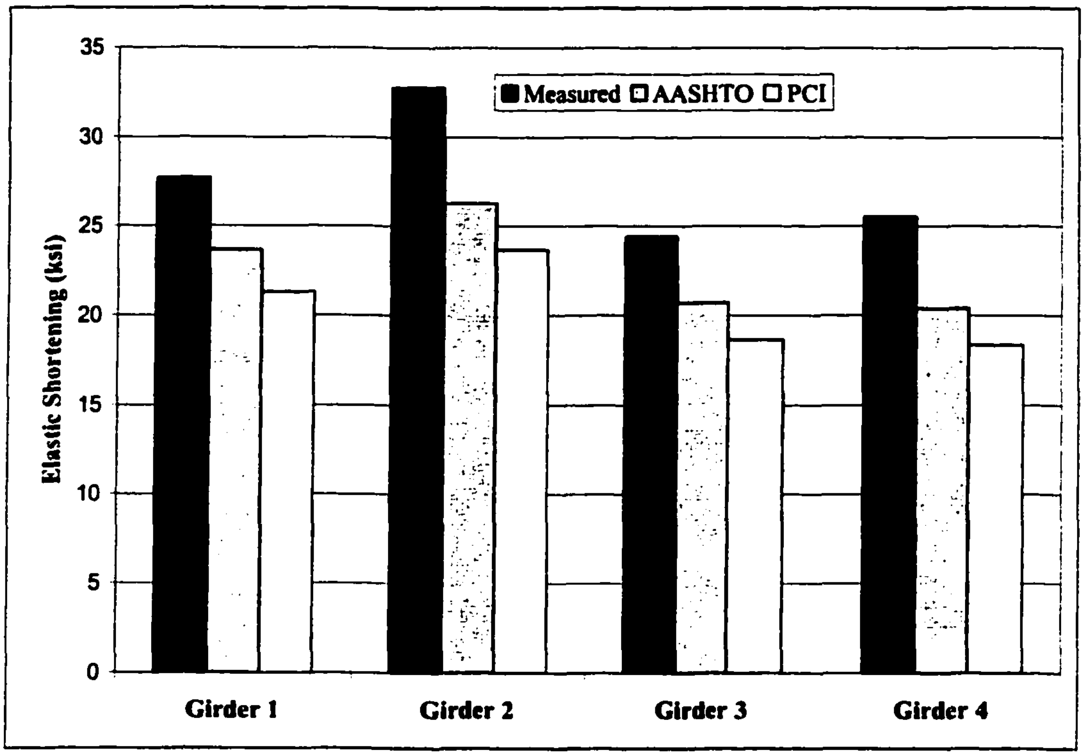

Figure 6.17. Comparison of Predicted and Measured E.S. Losses.

6.5.3.2 Creep. Creep is defined as the deformation of concrete due to sustained loads. As the girders deform or shorten due to the prestress force, the length of the strands shortens which results in a loss of prestress force. Creep is a major contributor to the loss of prestress force. The loss due to creep normally accounts for approximately 24 percent of the total losses (Lin and Burns, 1981).

The AASHTO and PCI predictions equations are shown in Equations 6.5 and 6.6, respectively. The results from the prediction equations are shown in Table 6.15. The calculations for the creep losses are shown in Appendix E. Unlike the results from the elastic shortening equations, there are major differences between the predicted values of the two equations. The AASHTO equations predicted creep losses that were between 22 to 29 percent greater than the PCI equations. However, there are some similarities between the equations. Both equations include the difference between the concrete stress at transfer and the concrete stress due to permanent dead loads. Also, by rearranging the PCI equation (Eqn. 6.7) and assuming a modular ratio of six, the Kcr(Es/Ec) terms becomes 12 which is similar to the AASHTO equation. The use of HPC in these girders resulted in a modulus of elasticity of 6000 ksi at 28 days of age. The greater modulus of elasticity reduced the modular ratio to 4.75 making the $K_{cr}(E_s/E_c)$ equal to 9.50 instead of 12. This decreased the creep losses predicted by the PCI equations when compared to the AASHTO equations. This is the most likely reason as to why the AASHTO equation predicted creep losses that were greater than the PCI equation.

$\Delta f_{pCR} = 12.0 f_{cgp} - 7.0 \Delta f_{cdp}$ **(AASHTO)** Eqn. 6.5

where:

f_{cgp} = concrete stress at the center of gravity of prestressing steel at transfer (KSI)

Δf_{cdp} = change in concrete stress at the center of gravity of prestressing steel due to permanent loads, except the load acting at the time the prestressing force is applied. Values of Δf_{cdp} should be calculated at the same section or sections for which f_{cgp} is calculated (KSI)

$CR = K_{cr}(E_s/E_c)(f_{cir}-f_{cds})$ **(PCI)** Eqn. 6.6

where:

K_{cr} = 2.0 for normal weight concrete

f_{cir} = net compressive stress in concrete at the center of gravity of tendons immediately after the prestress has been applied to the concrete (ksi).

f_{cds} = stress in concrete at center of gravity of tendons due to all superimposed permanent dead loads that are applied to the member after it had been prestressed

E_c = modulus of elasticity of concrete at 28 days

$f_{cds} = M_{sd}(e)/I_g$

M_{sd} = moment due to all superimposed permanent dead loads applied after prestressing

$CR = 12.0(f_{cir}-f_{cds})$ **(PCI-Rearranged)** Eqn. 6.7

Table 6.15. Estimated Losses Due to Creep.

Girders	Location	Prestress Losses Due to Creep (KSI)	
		AASHTO Predicted	PCI Predicted
1	Ends	55.81	39.54
	Center	54.75	38.71
2	Ends	52.04	39.82
	Center	50.96	38.88
3	Ends	41.02	32.24
	Center	40.08	31.43
4	Ends	48.13	34.67
	Center	46.98	33.76

6.5.3.3 Shrinkage. Concrete decreases in volume when it loses water through evaporation. Like creep and elastic shortening, as the concrete shortens, the prestress strands also shorten which results in a loss of prestress force. The shrinkage of the girders was determined by measuring the length change (ASTM C 157) of concrete prisms cast with the same concrete as the girders. The measured and predicted shrinkage losses are shown in Table 6.16. The shrinkage losses were 19.40 ksi for Girders1 and 4 and 21.10 ksi for Girders 2 and 3. The measured losses were calculated by using Hooke's Law. The measured losses were then compared to the losses predicted by the AASHTO (Eqn. 6.8) and PCI (Eqn. 6.9) equations. The calculations for the shrinkage losses are shown in Appendix E. Both equations estimated shrinkage losses that were over 50 percent less than those measured from the shrinkage prisms.

It must be noted that the measured shrinkage losses were determined from 2 x 2 x 11 in. concrete prisms cast with the same concrete as the girders. This may account for the differences between the measured and predicted losses due to shrinkage. Unlike the girders, the shrinkage prisms did not contain any reinforcement, which would restrain the concrete and reduce shrinkage. The addition of reinforcement would assist in preventing the girders from shrinking as much as the non-reinforced shrinkage prisms. Another possible explanation for the differences between the predicted and measured shrinkage is the quantity of cement used in the production of the girders. Neither equation (AASHTO or PCI) accounts for the quantity of cement used in the girders. As the quantity of cement in the mixture increases, the amount of shrinkage also increases (Mindess and Young, 1981). The AASHTO and PCI equations were developed based on mixtures with lower quantities of cement, but with the introduction of HPC more girders are being cast with greater cement quantities (at or near 1000 lb/yd^3).

$\Delta f_{pSR} = (17.0 - 0.150\ H)$ **(AASHTO)** Eqn. 6.8

where:

H = average ambient relative humidity

$SH = (8.2 \times 10\text{-}6) K_{sh} E_s \times (1 - 0.06V/S)(100 - R.H.)$ **(PCI)** Eqn 6.9

where:

K_{sh} = 1.0 for pretensioned members

V/S = volume to surface ratio

R.H. = average ambient relative humidity

Table 6.16. Prestress Losses Due to Shrinkage.

Girder	Prestress Losses Due to Shrinkage (KSI)		
	Measured (1 year)	AASHTO Predicted	PCI Predicted
1	19.40	7.25	7.36
2	21.10	7.25	7.36
3	21.10	7.25	7.36
4	19.40	7.25	7.36

6.5.3.4 Relaxation. Relaxation of the steel strand is similar to creep in concrete. Steel relaxation is the loss of prestress due to the constant strain AASHTO provides two different equations (Eqns. 6.10 and 6.11) for measuring the relaxation of the strands. The losses due to relaxation are calculated at transfer and after transfer. PCI developed one equation to predict relaxation losses (Eqn. 6.12).

The predicted losses due to relaxation are shown in Table 6.17. For all girders, the loss due to relaxation ranged 0.68 to 2.79 ksi. The calculations for relaxation losses are shown in Appendix E. As shown in Table 6.17, there are some differences between the predicted values between the two equations. For all girders, the AASHTO equations predicted losses that were less than those predicted by the PCI equations. The differences between the losses can be attributed to the greater creep losses predicted by the AASHTO equations.

$\Delta f_{pR1} = (\log(24.0t)/40.0)[f_{pj}/f_{py} - 0.55]\, f_{pj}$ at transfer (**AASHTO**) Eqn. 6.10

where:

$t =$ time estimated in days from stressing to transfer (days)

$f_{pj} =$ initial stress in the tendon at the end of stressing (ksi)

$f_{py} =$ specified yield strength of prestressing steel (ksi)

$\Delta f_{pR2} = 0.30[20.0 - 0.4\Delta f_{pES} - 0.2(\Delta f_{pSR} + \Delta f_{pCR})]$ (**AASHTO**) Eqn. 6.11

where:

$\Delta f_{pES} =$ loss due to elastic shortening (ksi)

Δf_{pSR} = loss due to shrinkage (ksi)

Δf_{pCR} = loss due creep of the concrete (ksi)

$RE = [K_{re} - J(SH + CR + ES)]C$ **(PCI)** Eqn. 6.12

where Kre, J, and C are taken from tables

ES = loss due to elastic shortening (ksi)

SH = loss due to shrinkage (ksi)

CR = loss due creep of the concrete (ksi)

Table 6.17. Predicted Losses Due to Relaxation.

Girders	Location	Prestress Losses Due to Relaxation (KSI)	
		AASHTO Predicted	PCI Predicted
1	Ends	0.84	2.39
	Center	0.95	2.44
2	Ends	0.68	2.18
	Center	0.81	2.24
3	Ends	1.96	2.53
	Center	2.07	2.58
4	Ends	1.69	2.70
	Center	1.82	2.76

6.5.3.5 Total Losses. The measured total losses and the predicted total losses are shown below in Table 6.18. For all beams, the AASHTO equations predicted losses that were greater than the measured losses. Once again, the differences between the predicted losses of the two equations can be attributed to the higher creep losses estimated by the AASHTO equations.

The measured losses for all girders are also shown in Figure 6.18. As shown in Figure 6.18, it appears that the majority of the losses occur by 180 days of age and afterward the rate of prestress loss decreases. Also, in the graph each curve has a noticeable "jump" in losses at 43 days of age. Moving the girders from the inside of FSEL to the outside of FSEL probably caused the increase in losses. This seems to be the most logical explanation. Prior to the readings taken at day 43, the girders had been stored inside at FSEL and then were moved outside of FSEL. The girders were loaded onto a trailer by the use of a spreader beam and then hauled outside where they were removed from the trailer by a forklift. The noticeable increases in losses were most likely caused by stresses imposed on the girder while they were being lifted by the spreader beam and forklift. Even though there was a noticeable jump in losses at 43 days of age, the next readings were more inline with the remaining data (Figure 6.18).

Table 6.18. Total Prestress Losses.

Girders	Location	Total Prestress Losses (KSI)		
		Measured (1 year)	AASHTO Predicted	PCI Predicted
1	Ends	56.78	87.56	70.48
	Center	53.30	86.17	69.25
2	Ends	71.95	86.26	72.66
	Center	74.03	84.76	71.24
3	Ends	59.22	70.96	61.00
	Center	58.46	69.66	59.75
4	Ends	51.86	77.48	63.32
	Center	51.80	75.97	61.98

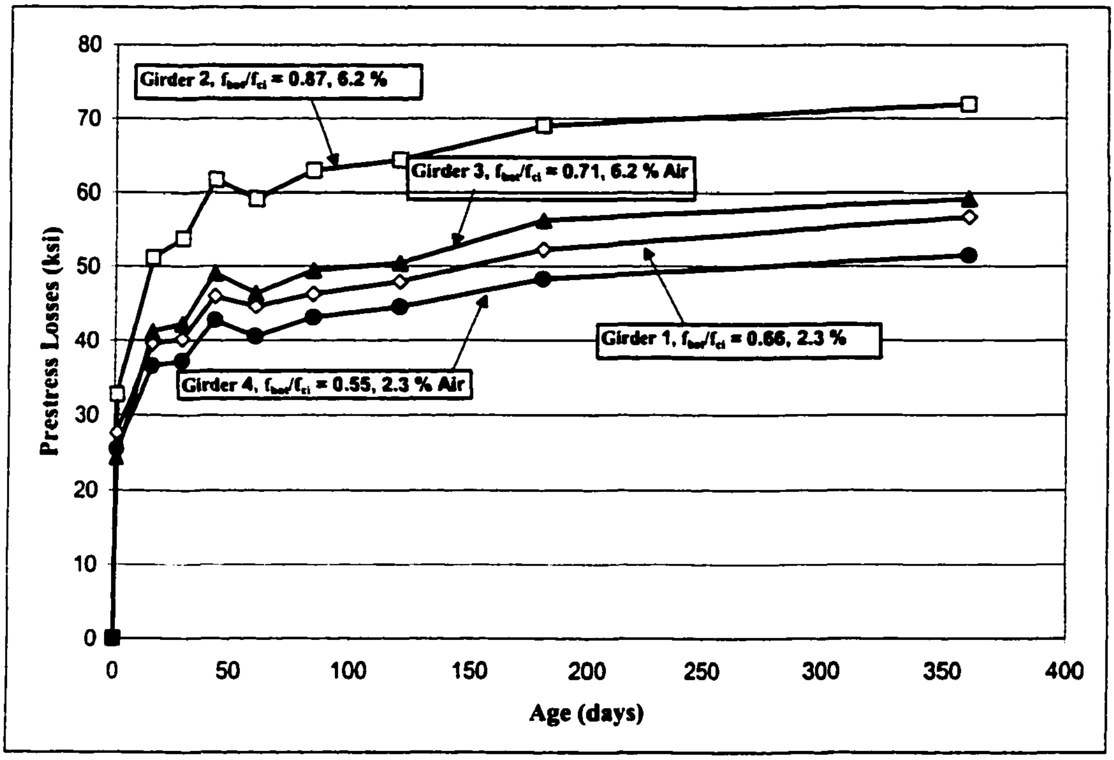

Figure 6.18. Measured Total Losses for the Girders.

6.5.4 Allowable Compressive Stress

The research program also investigated the allowable compressive stress at release. Currently the AASHTO Bridge Design Specifications and the Building Requirements for Structural Concrete (ACI 318-99) limits the compressive stress at release to be 60 percent or less of the concrete compressive strength at release ($0.60f_{ci}$). Researchers are investigating whether this limit can be increased beyond $0.60f_{ci}$. The lower release limit of $0.60f_{ci}$ causes the need for steam curing in many instances to achieve strengths. Steam curing increases the rate of strength gain in the concrete, allowing the concrete to achieve greater compressive strengths at earlier ages. However, steam curing has negative side effects. Steam curing reduces the ultimate strength and increases the permeability of steam-cured concrete when compared to concrete that is not steam cured (Neville, 1997 and Mindess and Young, 1981). By raising the allowable compressive stress to 75 percent, the need for steam curing might be eliminated which could result in concrete of higher quality. The research results will provide additional data to examine the efficacy of various proposals. The testing matrix is shown in Table 6.19.

Table 6.19. Testing Matrix.

Beam	Targeted Allowable Compressive Stresses (f_{bot} / f_{ci})		Targeted Total Air Content (%)	
	0.60	0.75	2	6
1	-	X	X	-
2	-	X	-	X
3	X	-	-	X
4	X	-	X	-

The targeted allowable stresses of 0.60 and 0.75 were not attained due to higher and lower than expected one-day compressive strengths. Girders 3 and 4 had a targeted release stress of 60 percent, and girders 1 and 2 had a targeted release stress of 75 percent. The one-day compressive strength of Girders 1 and 4 was expected to be near 8000 psi based on trial batching at FSEL and Coreslab. However, the compressive strength of Girders 1 and 4 was 8700 psi at one day. The 700 psi increase was due to the steam curing used by Coreslab. The increase in the one-day compressive strength for Girders 1 and 4 resulted in the targeted allowable compressive stress not being attained. Girder 1 was designed to have an allowable compressive stress at release of 0.75, but due to the unexpectedly high one-day compressive strength, this ratio was only 0.66. The same can be said for Girder 4. Girder 4 was designed to have allowable compressive stress of 0.60, but this ratio was only 0.55.

The lower than expected one-day compressive strengths of Girders 2 and 3 resulted in higher compressive stresses at release. Girder 2 was designed to have an allowable compressive stress at release of 0.75, but due to the low compressive strength, this ratio was 0.87. Girder 3 was designed to have an allowable compressive stress at release of 0.60, but the stress was 0.71. The low one-day compressive strength was most likely the result of the extremely low temperature the previous night, which prevented the steam in the prestressing beds from attaining its normal temperature. Girders 2 and 3 were cast on Thursday, December 21, and the prestressing strands were released on Friday, December 23. Due to the holiday weekend (Christmas), Coreslab was closing early so waiting for the concrete to reach higher compressive strengths was not an option.

Even though the targeted allowable stresses were not attained, three of the four girders had allowable compressive stresses at release greater than 0.60. The high release stresses did not appear to have any noticeable detrimental effects on the girders. The only possible effect of the high compressive stress at release occurred on the south of Girder 2. As shown in Figure 6.19 and 6.20, a portion of concrete fractured along the web of the girder. It is uncertain to whether this fracture is due to the high release stress or due to other factors such as low minimum cover throughout the web of the girder (less than 1.5 inches). Girder 2 did have the highest stress at release of 87.0 percent, which is 45 percent greater than that allowed by AASHTO and ACI. The remaining girders did not have any major cracks or other signs of failure.

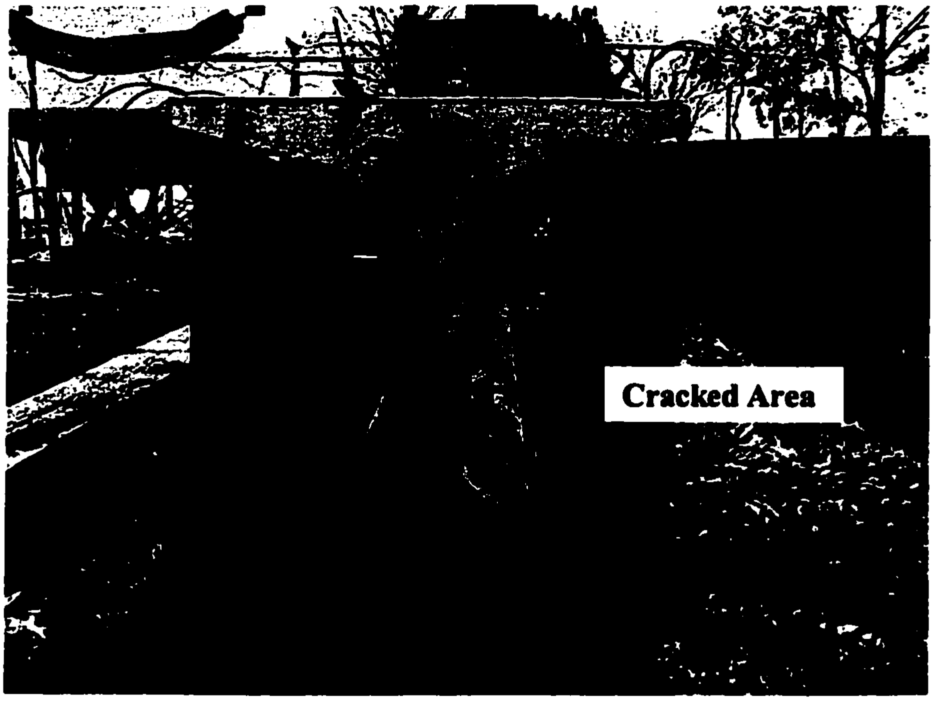

Figure 6.19. Photograph of Girder 2.

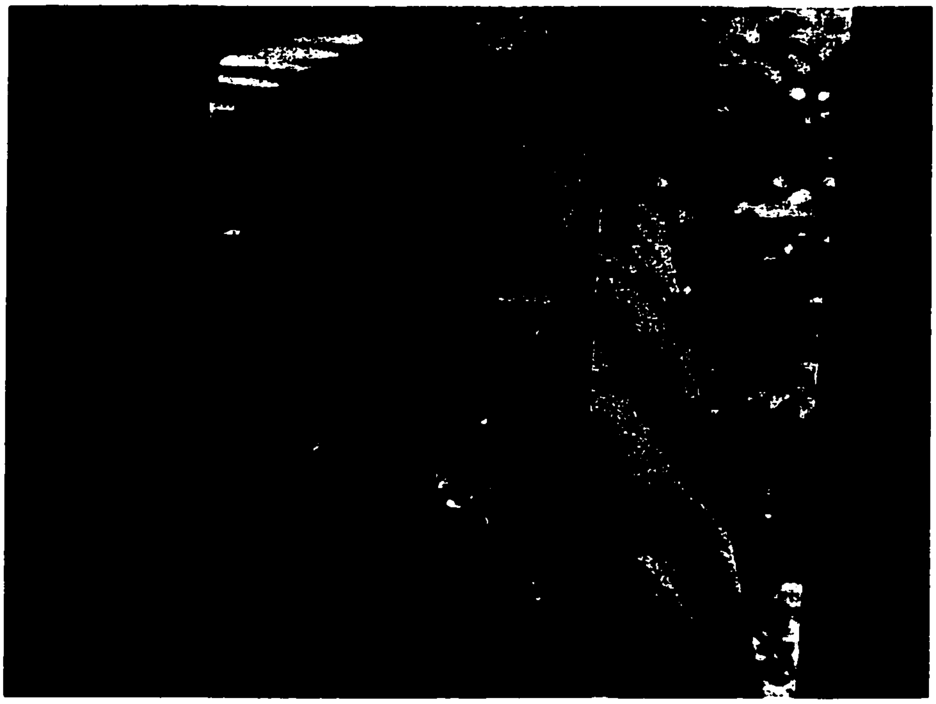

Figure 6.20. Photograph of Fractured Area of Girder 2.

Although the high release stresses did not appear to have any effect on the appearance of the girders, they did affect the amount of prestress losses. The total measured losses for the four girders and the allowable compressive stress at release are shown in Table 6.20. As shown in Figure 6.21, the girders with the greatest release stress had the highest amount of prestress loss. Girder 2 which had the highest stress of 0.87 had the most losses followed in descending order of release stress.

Even though the two girders with the greatest losses (Girders 2 and 3) contained entrained air, the increase in losses is most likely due to the high release stress. Earlier in the research program the shrinkage of the girders was examined. Shrinkage prisms cast from the concrete used in both girders had similar shrinkage losses (19.40 and 21.10 ksi at one year). Johnson (2000) examined the creep of the mixtures. His results showed that addition of entrained air should not have affected the creep behavior of the girders (Figure 6.15). The elastic shortening losses were higher for Girder 2, which was expected because of the high release stress of 0.87, but the losses for the remaining girders fell within a range of 24 to 28 ksi. The results from this research and Johnson's (2001) research show that entrained air does not have any affect on the prestress losses other than those losses affected by the reduction in compressive strength caused by entrained air. Therefore, the increase in losses shown in Figure 6.21 is most likely due to the increase in the allowable compressive stress at release.

Further examination of Figure 6.21 shows that there is a point where the increase in release stress has a greater effect on the prestress losses. It appears that Girders 1, 3, and 4 show increases in prestress losses that are consistent with their increase in release stress. However, the losses for Girder 2 do not follow the same trend. Shown in Table

6.21 is the change in prestress losses due to the percent change in release stress. Each girder is used as a reference to show the change in losses due to the change in release stress. Excluding Girder 2, a one percent increase in release stress resulted in a 0.45 or 0.46 ksi increase in prestress losses. Girder 2 which had a release stress of 0.87 did not follow this trend. Depending on which girder is being referenced, the change in the prestress losses per increase in release stress ranged from 0.64 to 0.82 ksi for Girder 2 which is approximately 50 to 90 percent greater than the rate of Girders 1, 3, and 4. These results show that above a release stress of 0.71 there is a point where the increase in release stress versus the increase in prestress losses is no longer linear. Even though there was no significant damage to Girder 2 caused by the high release stress of 0.87, the increase in total losses of at least 20 percent when compared to the remaining girders would deter most engineers and researchers from raising allowable stress limit as high as 0.87.

Table 6.20. Compressive Stress at Release and Measured Losses.

Girders	Location	Total Prestress Losses (KSI)			
		f_{bot}/f_{ci}	Measured (1 year)	AASHTO Predicted	PCI Predicted
1	Ends	66.0	56.78	87.56	70.48
	Center		53.30	86.17	69.25
2	Ends	86.9	71.95	86.26	72.66
	Center		74.03	84.76	71.24
3	Ends	71.4	59.22	70.96	61.00
	Center		58.46	69.66	59.75
4	Ends	55.4	51.86	77.48	63.32
	Center		51.80	75.97	61.98

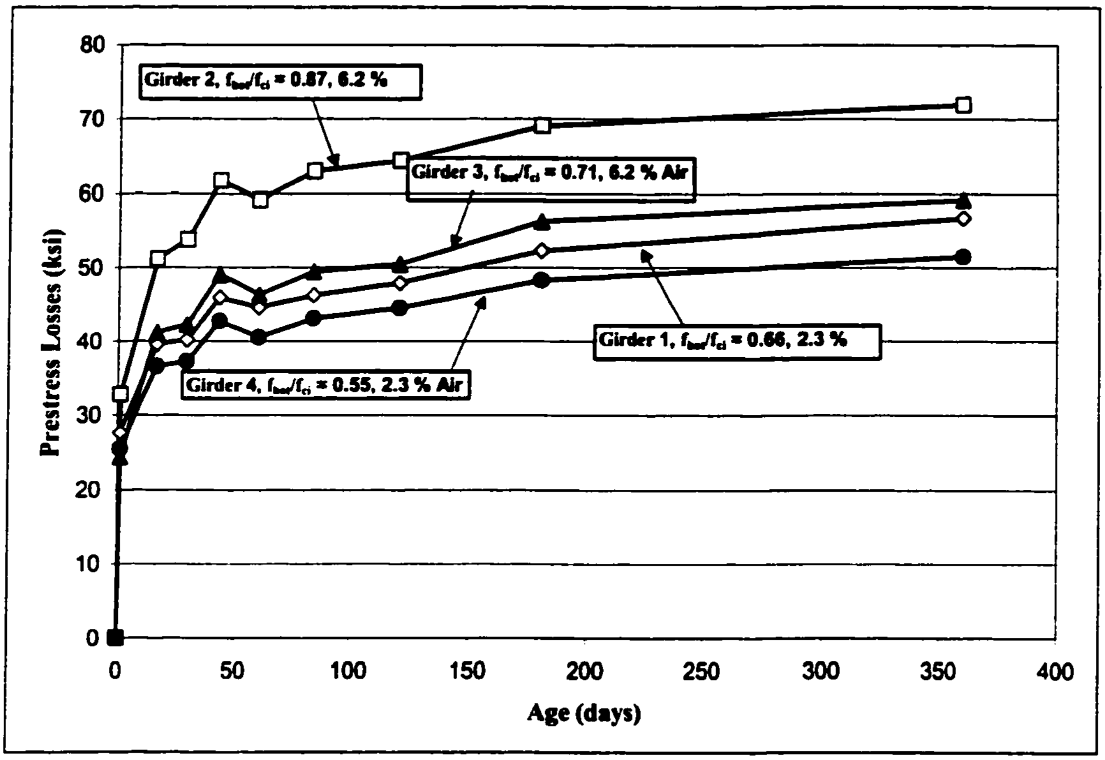

Figure 6.21. Measured Total Losses.

Table 6.21. Change in Prestress Losses (ksi) per Increase in Release Stress.

Girders	f_{bot}/f_{ci}	Measured Losses (1 year)	Change in Prestress Losses per Percent Change in Release Stress			
			Reference to Girder 4	Reference to Girder 3	Reference to Girder 2	Reference to Girder 1
1	66.0	56.78	0.46	0.45	0.73	0.00
2	86.9	71.95	0.64	0.82	0.00	0.72
3	71.4	59.22	0.46	0.00	0.82	0.45
4	55.4	51.86	0.00	0.46	0.64	0.46

6.0 CONCLUSIONS

One of main objects of the research program was to demonstrate that the use HPC is possible in the local prestressed concrete industry. Girders were cast with mixtures that had a w/cm of 0.26. The mixtures attained an early strength of 8700 psi (for Girders 1 and 4) and reached over 15,000 psi at one year (for Girders 1 and 4). It must be noted that these mixtures did not contain any special chemical or mineral admixtures. The girders were cast with materials that Coreslab uses in their normal day-to-day operations. The casting of the girders also reemphasized the detrimental effects that air entrainment has on the compressive strength of concrete. At one day of age, the difference between the two mixtures was over 2500 psi, and at later ages (14, 28, and 56 days of age) this difference was approximately 3000 psi.

Another task of the research program was to examine the prestress losses of girders with and without air entrainment. Other than the increase in losses due to the decrease in compressive strength, entrained air did not have any noticeable effect on the prestress losses. Increasing the allowable compressive at release was also investigated. Current codes limit the release stress to $0.60f_{ci}$. Another goal of the research was to provide additional data to determine of this limit could be raised. Based on the results, the allowable compressive stress limit can be raised, but the prestress losses would increase. The results from the research support increasing the limit to $0.71f_{ci}$, but somewhere between $0.71f_{ci}$ and $0.87f_{ci}$ the increase in prestress losses becomes too great to merit an increase beyond $0.71f_{ci}$. Additional findings of this section of the research program are summarized below.

1. With good quality control measures, most precast/prestressed concrete facilities have the capabilities to produce to HPC. HPC was produced using locally available materials and produced using a concrete mixer that was manufactured in 1926.
2. Other than the reduction in compressive strength, the addition of entrained air did not affect the prestress losses when compared to similar non-air entrained girders.
3. The AASHTO prediction equations over estimated the total losses for all girders. The AASHTO equations predicted losses that were approximately 50 percent greater than the measured losses for Girders 1 and 4. For all girders, the PCI equations estimated losses that were more accurate than the AASHTO equations. Pessiki et al (1996), Roller et al (1995), Roller et al (2000), and Idriss (2001) reported that the AASHTO prediction equations overestimate the prestress losses.
4. For all girders, the AASHTO equations underestimated the losses due to elastic shortening. The overestimation was most likely due to changes in concrete temperature between readings and the high release stress and lower modulus of elasticity for girders with release stresses greater than $0.70f_{ci}$.
5. For all girders, the AASHTO equations underestimated the losses due to shrinkage. However, these measured losses may not truly represent the shrinkage losses of the girders. The losses were measured from 2 x 2 x 11 in. concrete prisms. Unlike the girders, the shrinkage prisms did not contain any reinforcement, which would restrain the concrete and reduce shrinkage. The

addition of reinforcement would assist in preventing the girders from shrinking as much as the non-reinforced shrinkage prisms. Another possible explanation for the differences between the predicted and measured shrinkage is the quantity of cement used in the production of the girders. The AASHTO equation does not account for the quantity of cement used in the girders.

6. When compared to the PCI equations, the AASHTO equations predicted creep losses that were much greater (22 to 29 percent). The AASHTO equations do not account for the modulus of elasticity. This is the main reason as to why the AASHTO equations estimated losses that were greater than those predicted by the PCI equations.
7. Increasing the allowable compressive stress at release resulted in greater prestress losses. For all girders, the prestress losses increased with raising the release stress. The results support increasing the release limit to $0.71f_{ci}$, but benefits of a higher release stress might be negated by greater prestress losses.

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

7.1 INTRODUCTION

As stated in Chapter 1, the research program investigated the need for air entrainment in high performance concrete (HPC) and its effect on HPC bridge girders. In addition to investigating the freeze-thaw durability of HPC, the research program also examined the permeability of HPC by using the Rapid Chloride Ion Penetrability test (RCIP). This portion of the research examined the permeability of mixtures with and without air entrainment and also mixtures with and without pozzolans. Another objective of the research examined the prestress losses in concrete with and without air entrainment. This portion of the study also included investigating the allowable release stress of prestressed concrete girders. The significant findings of each portion of the research program are summarized in the following sections.

7.2 DEVELOPING HIGH PERFORMANCE CONCRETE

The results of this section of the research program showed that HPC can be produced with materials that are native to Oklahoma. All materials used in the research program are currently being used by batch plants and prestressing facilities in the area. Concrete mixtures were cast that achieved a one-day compressive strength of more than 9000 psi and a 28-day compressive strength of more than 14,000 psi. A minimum w/cm of 0.26 and a maximum HRWR dosage of 15 fl oz/cwt were established as the lower and upper limits needed to produce concrete mixtures with sufficient workability. The amount of slump loss negated the use of an accelerator. The findings of this section of the research program are summarized below.

1. There were some differences observed between cement manufacturers. Cements intended for use in HPC should be used in trial batching before use in the concrete structure.
2. For most mixtures increases in cement content resulted in greater slumps for mixtures with like w/cm. The results showed that a minimum cement content of 900 lb/yd^3 was necessary for a concrete mixture with a w/cm equal to and less than 0.26 to have sufficient workability. For the mixtures tested, cement contents of 800 lb/yd^3 and greater produce workable concrete at w/cm of 0.28 and greater. Results from these mixtures showed that a cement content of 900 pounds per cubic yard of concrete showed the most promise because of its workability (slumps greater than 7 in.) and compressive strength (one day strengths greater than 7000 psi).
3. The addition of HRWR increased workability (Figure 2.4 of Chapter 2). Too much HRWR delayed set times and decreased early age strength (Figure 2.9 of Chapter 2). At w/cm between 0.26 and 0.30, a dosage rate

of 15 fl oz/cwt was necessary to produce concrete with sufficient workability. Lower dosages of HRWR were possible with the use of a WR but only at higher w/cm. Based on workability and compressive strength, mixtures with a w/cm of 0.26 and HRWR of 15 fl oz/cwt displayed the most promise of producing concrete with the highest compressive with sufficient workability.

4. The addition of a calcium nitrite accelerator increased the one-day compressive strength. For most mixtures, the increase in compressive strength was between 500 and 1500 psi. Even though all mixtures benefited from the addition of the accelerator (calcium nitrite accelerator), the substantial slump loss proved to be too great to benefit from the use of a calcium nitrite accelerator.
5. Based on compressive strength and workability, the results of research program indicated that for the aggregates and cements tested, the optimum cement content was 900 lb/yd^3 (results found in section 2.5.2.3). The optimum w/cm was 0.26. This mix proportion will be used to cast the precast/prestressed bridge girders that will be examined in later phases of the research program.

7.3 DEVELOPING HIGH PERFORMANCE CONCRETE WITH ENTRAINED AIR

This portion of the research focused on the development of HPC with air entrainment using locally available materials. With one of the mixtures developed, two prestressed concrete beams were cast. Once again, not only were the mixtures to have high compressive strength, they must also be cast with local materials and must have enough workability to be placed. Another task of the research was to document the effects of air entrainment of the compressive strength of the concrete. The findings of the research are summarized below.

1. Higher dosages of AEA are required at w/cm below 0.30 (Figure 3.2). At w/cm greater than 0.30, the dosage rates required were within the manufacturer's recommendation. Mixtures with w/cm of 0.26 and 0.28 required additional mixing time to achieve the desired air contents. Trial batching is recommended at lower w/cm to determine the correct dosages of admixtures.
2. For most w/cm, the compressive strength decreased as the total air content increased. By 56 days of age, most mixtures followed the "2 – 5 percent decrease in compressive strength for every 1 percent increase in entrained air" rule of thumb (Table 3.16).
3. Air entrained HPC mixtures with adequate workability and high compressive strength can be developed using locally available materials.

4. Based on compressive strength and workability, the results of research program indicated that for the aggregates and cements tested, the optimum cement content was 900 lb/yd^3. The optimum w/cm was 0.26 (results can be found in section 3.5.2 of Chapter 3). This mix proportion will be used to cast the precast/prestressed bridge girders that will be examined in later phases of the research program.

7.4 EXAMINING THE FREEZE-THAW DURABILITY OF HIGH PERFORMANCE CONCRETE

The experimental program was designed to examine two objectives. The first objective was to determine if air entrainment is necessary for freeze-thaw durability in HPC. The second objective is to determine the maximum w/cm where air entrainment is not needed. Results from the research show that air entrainment may not be necessary to achieve adequate freeze-thaw durability for concrete mixtures with w/cm less than 0.36. From the results in Table 4.9 from Chapter 4, it also appears that a total air content of 4 percent is enough to provide freeze-thaw durability for mixtures with a w/cm between 0.36 and 0.50. However, all mixtures examined in the research contained the same type of coarse aggregate, which was known to have adequate durability. The results from this research apply only to the materials used in the research. Mixtures containing aggregates with poor freeze-thaw durability may not perform as well as the mixtures tested in this research program. Before one can specify non-air entrained concrete in a harsh environment, specimens containing the materials to be used in the concrete should to be tested to determine its freeze-thaw durability. If freeze-thaw testing is not available, air entrainment should be employed in the concrete.

7.5 EXAMINING THE PERMEABILITY OF HIGH PERFORMANCE CONCRETE WITH AND WITHOUT ENTRAINED AIR

This section of the research program investigated the permeability of high performance concrete (HPC) with and without entrained air. Another purpose was to additional data on the effectiveness of the RCIP Test (ASTM C 1202) as a means to measure concrete penetrability. The findings of the research are summarized below.

1. The addition of entrained air had no noticeable effect the permeability of the mixtures. The findings are consistent with those of other researchers (Mindess et al, 1981; Neville, 1997; Kosmatka et al, 1994).
2. To achieve low permeability concrete (based on ASTM C 1202 classifications), mixtures should be cast at a w/cm equal to or less than 0.34. The results are similar with Pinto et al (2001) who found the a w/cm of 0.35 was necessary to produce with low permeability based on ASTM C 1202.
3. For mixtures not containing any mineral admixtures, the RCIP test appears to be effective in measuring concrete's chloride ion penetrability. As shown in Figure 5.4 and 5.5, the concrete's penetrability increases as the

w/cm increase, which is expected. All mixtures containing only portland cement follow this trend.

4. Once mineral admixtures are introduced, this trend is no longer valid. When compared to similar control mixtures, silica fume reduced the chloride ion penetrability by more than 75 percent.
5. Since the RCIP test does not directly measure permeability, other factors can influence the parameter the test actually does measure, which is concrete resistivity. Therefore, other factors (such as pore fluid chemistry) may influence the parameter actually measured, leading to a basic need for users to understand what the test actually measures so that results can be put into proper perspective. When testing concrete containing mineral admixtures, one should compare the results to an identical mixture without mineral admixtures. This comparison with mixtures containing only portland cement may provide a better idea as to the true permeability of the concrete.

7.6 EXAMINING THE PRESTRESS LOSSES AND ALLOWABLE COMPRESSIVE STRESS AT RELEASE OF BRIDGE GIRDERS WITH AND WITHOUT AIR ENTRAINMENT

One objective of the research program was to demonstrate that the use HPC is possible in the local prestressed concrete industry. Girders were cast with mixtures that had a w/cm of 0.26. The mixtures attained a one day strength of 8700 psi (for Girders 1 and 4) and reached over 15,000 psi at one year (for Girders 1 and 4). It must be noted that these mixtures did not contain any special chemical or mineral admixtures. The girders were cast with materials that Coreslab uses in their normal day-to-day operations. The casting of the girders also re-emphasized the effects that air entrainment has on reducing the compressive strength of concrete. At one day of age, the difference between the two mixtures was over 2500 psi, and at later ages (14, 28, and 56 days of age) this difference was approximately 3000 psi.

Another task of the research program was to examine the prestress losses of girders with and without air entrainment. Other than the increase in losses due to the decrease in compressive strength, entrained air did not have any noticeable effect on the prestress losses. Increasing the allowable compressive at release was also investigated. Current codes limit the release stress to $0.60f_{ci}$. Another goal of the research was to provide additional data to determine of this limit could be raised. Based on the results, the allowable compressive stress limit can be raised, but the prestress losses would increase. The results from the research support increasing the limit to $0.71f_{ci}$, but somewhere between $0.71f_{ci}$ and $0.87f_{ci}$ the increase in prestress losses becomes too great to merit an increase beyond $0.71f_{ci}$. Additional findings of this section of the research program are summarized below.

1. With good quality control measures, most precast/prestressed concrete facilities have the capabilities to produce to HPC. HPC was produced using locally available materials and produced using a concrete mixer that was manufactured in 1926.
2. The addition of entrained air decreased the modulus of elasticity at all ages when compared to the non-air entrained mixture (Table 6.5). Prediction equations can be used to estimate the modulus of elasticity based on the compressive strength of the concrete.
3. Other than the reduction in compressive strength and modulus of elasticity, the addition of entrained air did not affect the prestress losses when compared to similar non-air entrained girders.
4. The AASHTO prediction equations over estimated the total losses for all girders. The AASHTO equations predicted losses that were approximately 50 percent greater than the measured losses for Girders 1 and 4. For all girders, the PCI equations estimated losses that were more accurate than the AASHTO equations.
5. For all girders, the AASHTO equations underestimated the losses due to elastic shortening (Table 6.14).. The overestimation was most likely due to changes in concrete temperature between readings and the high release stress.

6. For all girders, the AASHTO equations underestimated the losses due to shrinkage (Table 6.16). However, these measured losses may not truly represent the shrinkage losses of the girders. The losses were measured from 2 x 2 x 11 in. concrete prisms. Unlike the girders, the shrinkage prisms did not contain any reinforcement, which would restrain the concrete and reduce shrinkage. The addition of reinforcement would assist in preventing the girders from shrinking as much as the non-reinforced shrinkage prisms. Another possible explanation for the differences between the predicted and measured shrinkage is the quantity of cement used in the production of the girders. The AASHTO equation does not account for the quantity of cement used in the girders.
7. When compared to the PCI equations, the AASHTO equations predicted creep losses that were much greater (22 to 29 percent). The AASHTO equations do not account for the modulus of elasticity. This is the main reason as to why the AASHTO equations estimated losses that were greater than those predicted by the PCI equations.
8. Increasing the allowable compressive stress at release resulted in greater prestress losses. For all girders, the prestress losses increased with raising the release stress. The results support increasing the release limit to $0.71f_{ci}$, but benefits of a higher release stress might be negated by greater prestress losses.

7.7 RECOMMENDATIONS

Further research needs to examine the freeze-thaw durability of concrete containing aggregates that are known to have poor freeze-thaw durability. For the aggregates used in this research program, the results have shown that air entrainment is not required at w/cm equal to and less than 0.36. Is this statement still valid for concretes containing poor aggregates? Further research should also be done to determine if a total air content of four percent provides adequate freeze-thaw durability at w/cm greater than 0.50. The air void system of the hardened concrete should also be examined to determine the spacing factors and the air contents of the hardened concrete.

Additional research in the area of prestress losses and allowable compressive stresses at release is needed. The results from this research show that AASHTO prediction equations overestimate the prestress losses of girders with release stresses at or near $0.6f_{ci}$. Further research is needed to investigate the prestress losses of members with release stresses greater than $0.6f_{ci}$. Finally, HPC bridge girders should be cast and instrumented to monitor the prestress losses over the life of the structure.

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APPENDIX A

Appendix A includes the individual test results and statistical data for the results from Chapter 2. Appendix A also includes plots of the 90 percent confidence intervals.

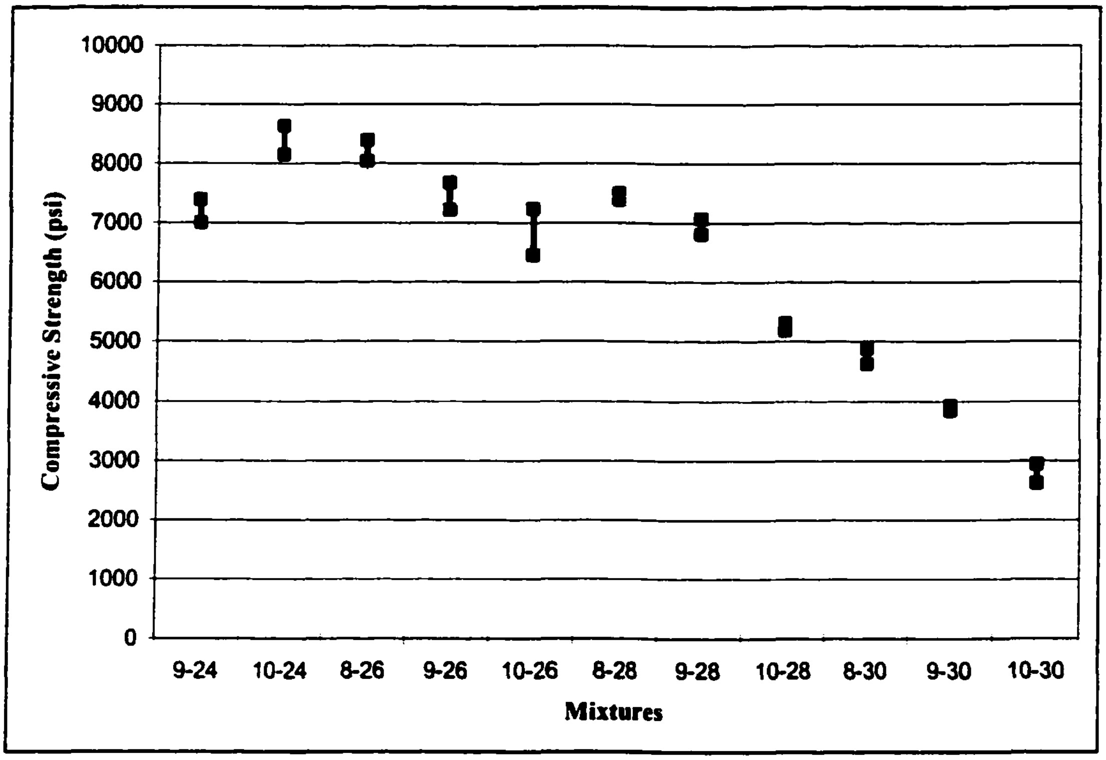

Figure A.1 One-Day Compressive Strengths for Optimizing Cement Content &w/cm.

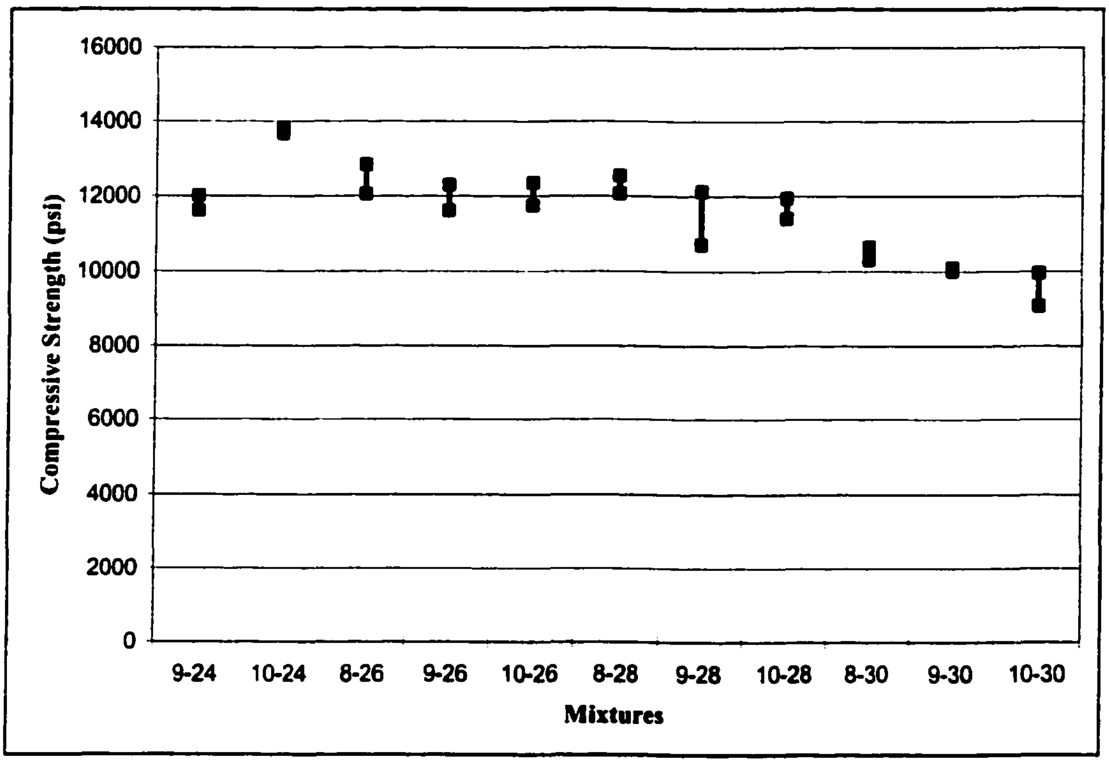

Figure A.2 28 Day Compressive Strengths for Optimizing Cement Content & w/cm.

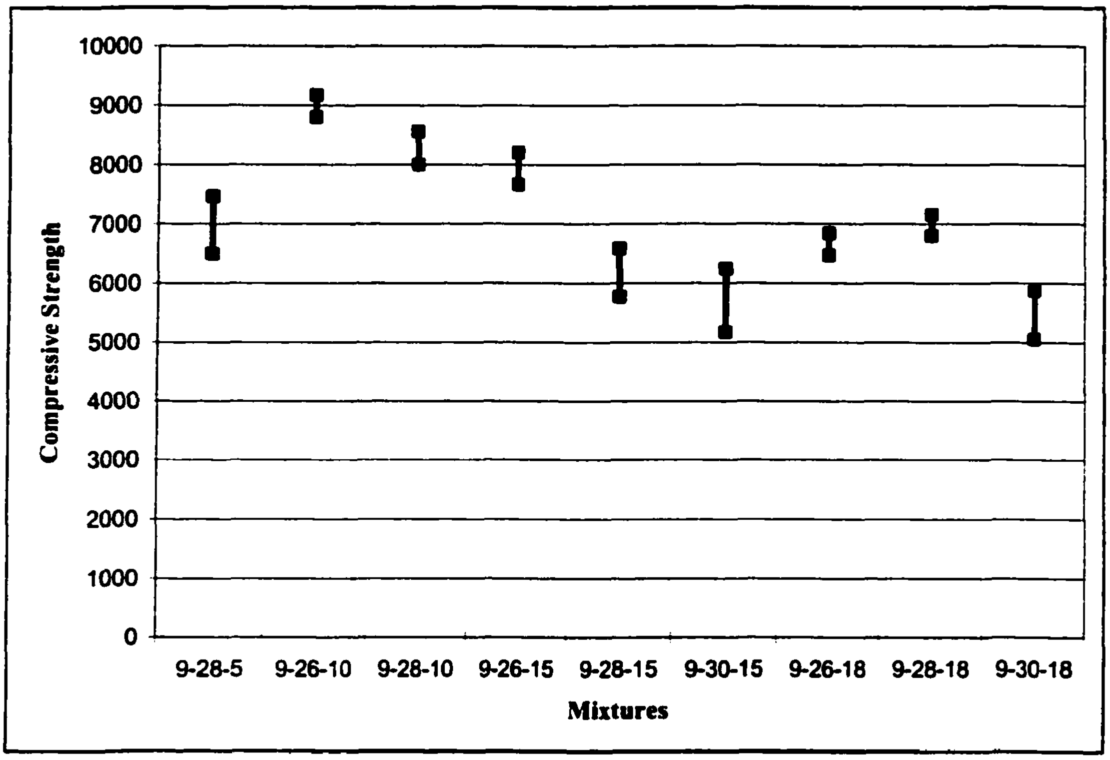

Figure A.3 One-Day Compressive Strength for Optimizing HRWR Dosage.

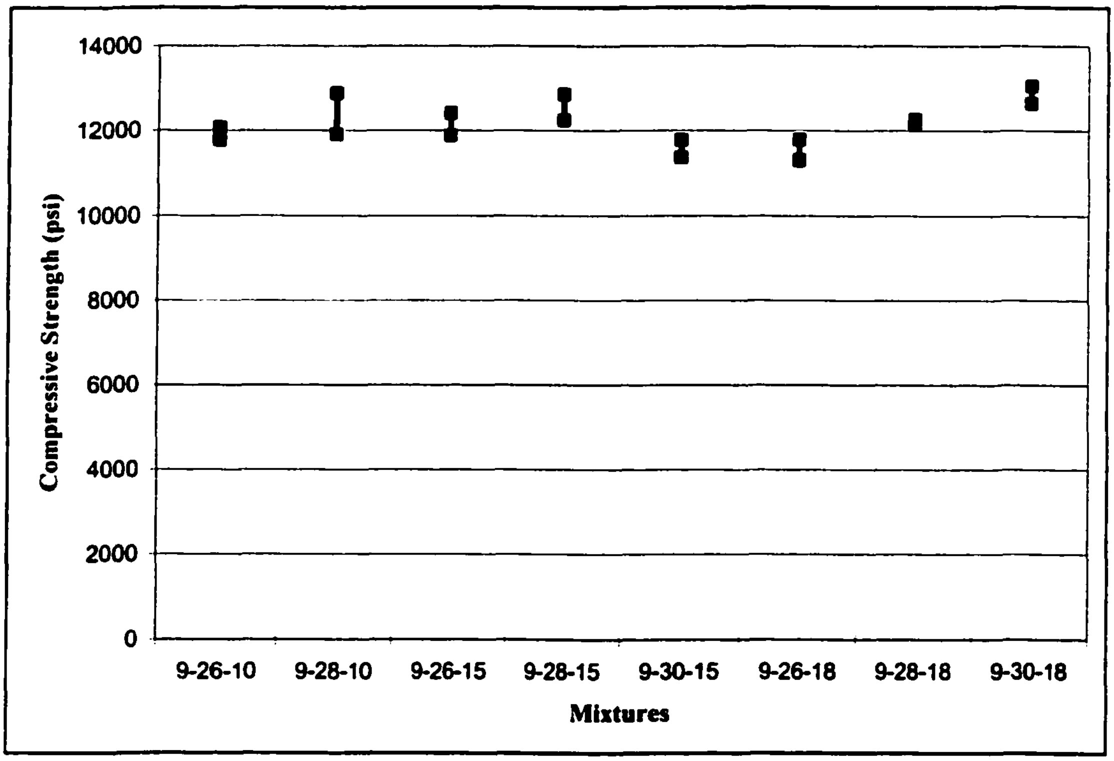

Figure A.4 28-Day Compressive Strength for Optimizing HRWR Dosage.

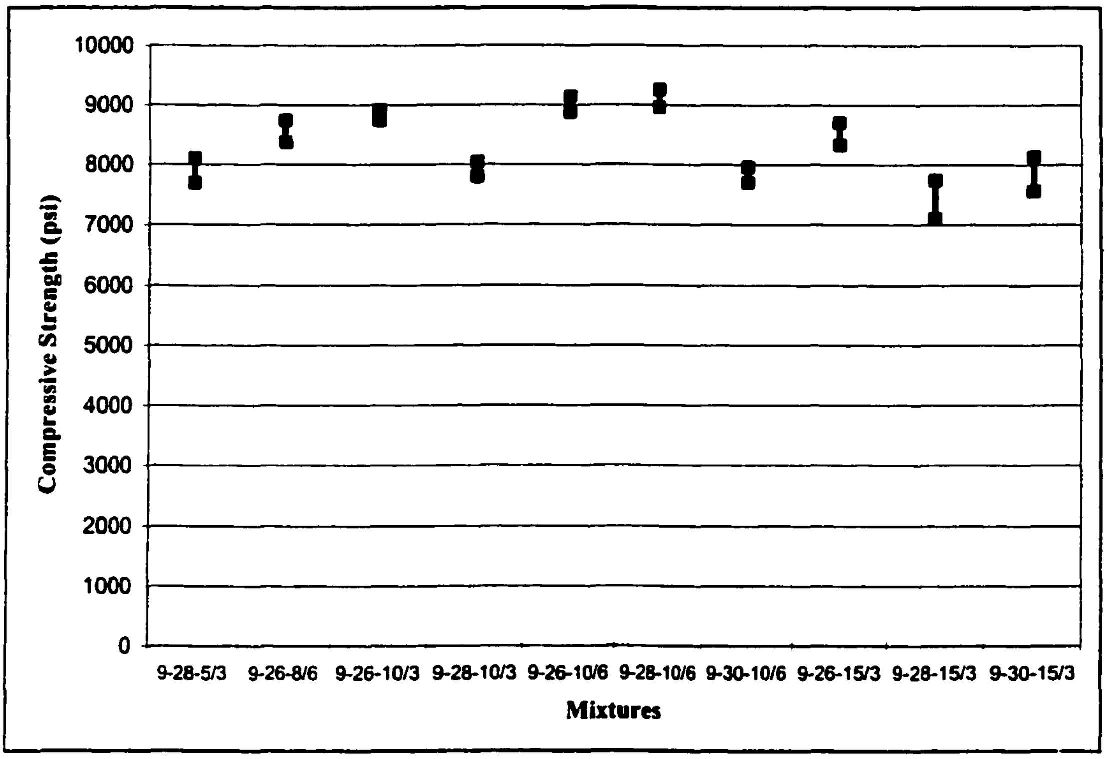

Figure A.5 One-Day Compressive Strength for Optimizing HRWR and WR Dosage.

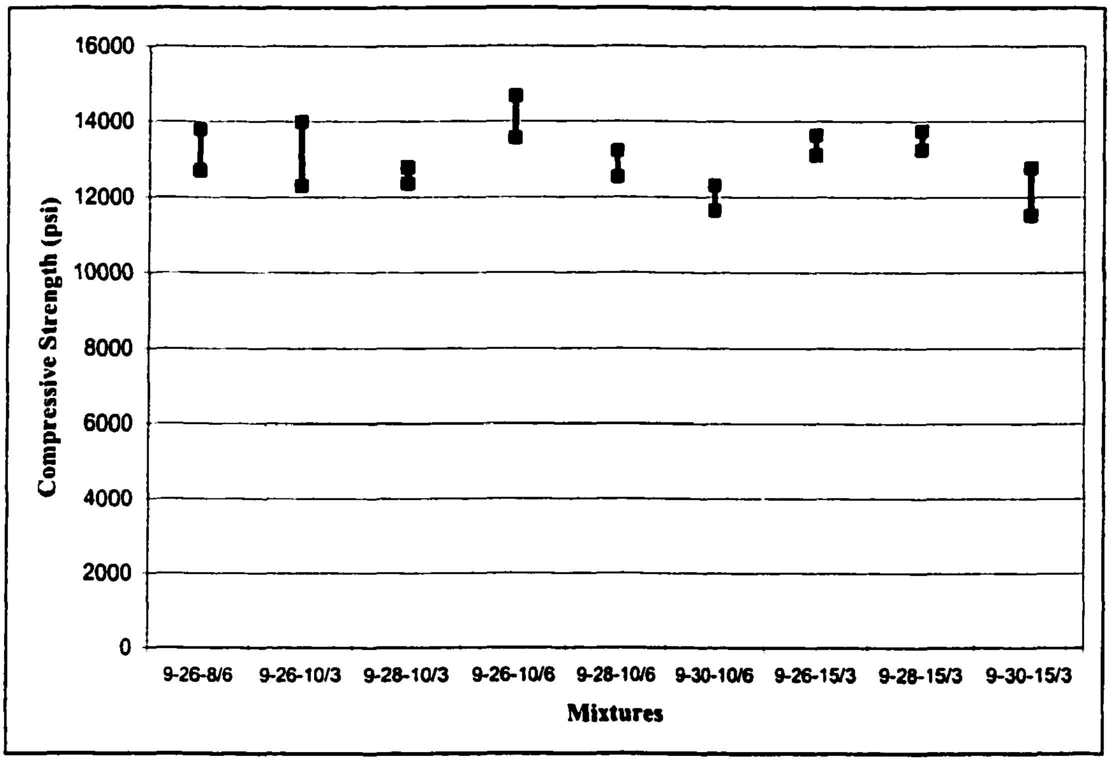

Figure A.6 28-Day Compressive Strength for Optimizing HRWR and WR Dosage.

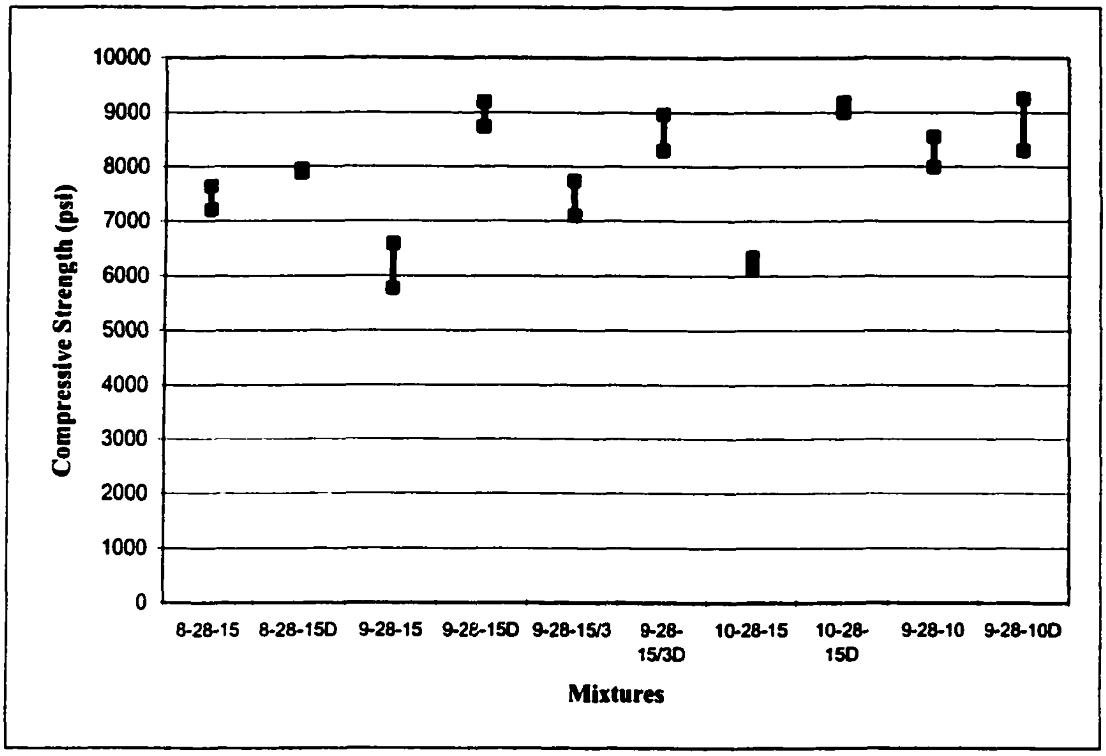

Figure A.7 One-Day Compressive Strength Results for Examining Accelerator Dosage.

Table A.1 Individual Results & Statistical Data for 1 day Compressive Strength Tests.

Mixture Designation	One Day Compressive Strength (psi) Cylinder 1	Cylinder 2	Cylinder 3	Average (psi)	Std. Dev. (psi)	90 Percent Confidence Intervals Lower Limit	Upper Limit	Coefficient of Variation
9-24	7432	6928	7220	7193	207	6997	7390	2.87
10-24	8654	8467	8041	8387	257	8144	8631	3.06
8-26	8015	8159	8465	8213	188	8035	8391	2.28
9-26	7524	7119	7698	7447	243	7217	7677	3.26
10-26	6258	7146	7123	6842	413	6450	7235	6.04
8-28	7384	7501		7443	59	7374	7511	0.79
9-28	6755	7077	6983	6938	135	6810	7067	1.95
10-28	5308	5184		5246	62	5174	5318	1.18
8-30	4678	4632	4934	4748	133	4622	4874	2.80
9-30	3946	3847	3865	3886	43	3845	3927	1.11
10-30	2645	2687	3015	2782	165	2625	2939	5.95
9-26-18	6800	6800	6387	6662	195	6477	6847	2.92
9-28-18	6739	7168	7040	6982	180	6812	7153	2.58
9-30-18	4859	5849	5678	5462	432	5052	5872	7.91
9-26-15	7842	7639	8313	7931	282	7663	8199	3.56
9-28-15	5715	6081	6755	6184	431	5775	6593	6.97
9-30-15	5818	4968	6347	5711	568	5172	6250	9.95
9-26-10	9257	8789	8919	8988	197	8801	9176	2.19
9-28-10	8157	8680	7998	8278	291	8002	8555	3.52
9-28-5	7682	6773	6501	6985	505	6506	7465	7.23
8-28-15	7506	7654	7115	7425	227	7209	7641	3.06
10-28-15	6207	6391	6127	6242	111	6137	6347	1.77
9-26-8/6	8584	8306	8776	8555	193	8372	8739	2.26
9-26-10/6	8821	9145	9036	9001	135	8873	9129	1.50
9-26-10/3	8949	8750	8776	8825	88	8741	8909	1.00
9-26-10/3	8821	9145	9036	9001	135	8873	9129	1.50
9-26-15/3	8241	8624	8684	8516	196	8330	8703	2.30
9-28-5/3	7914	8152	7628	7898	214	7695	8101	2.71
9-28-10/6	9055	9314	8941	9103	156	8955	9252	1.71
9-28-10/3	7764	7921	8075	7920	127	7799	8041	1.60
9-28-10/3	8812	8758	7801	8457	464	8016	8898	5.49
9-28-15/3	7653	6951	7666	7423	334	7106	7741	4.50
9-30-10/6	7820	7673	7996	7830	132	7704	7955	1.69
9-30-15/3	7771	8243	7522	7845	299	7561	8129	3.81
8-28-15D	7950	7896		7923	27	7892	7954	0.34
9-28-15D	8659	9246	8984	8963	240	8735	9191	2.68
10-28-15D	9016	9043	9228	9096	94	9006	9185	1.04
9-28-10D	8273	8598	9460	8777	501	8301	9253	5.71
9-30-10D	8080	8750	8364	8398	275	8137	8659	3.27
9-28-15/3D	8325	8448	9118	8630	348	8299	8961	4.04

Table A.2 Individual Results & Statistical Data for 28 D Compressive Strength Tests.

Mixture	28 Day Compressive Strength (psi)			Average	Std. Dev.	90 Percent Confidence Intervals		Coefficient of
Designation	Cylinder 1	Cylinder 2	Cylinder 3	(psi)	(psi)	Lower Limit	Upper Limit	Variation
9-24	12045	11847	11538	11810	209	11612	12008	1.77
10-24	13846	13741	13651	13746	80	13670	13822	0.58
8-26	12487	11925	12938	12450	414	12056	12844	3.33
9-26	12058	12320	11459	11946	360	11603	12288	3.02
10-26	12021	12456	11683	12053	316	11753	12354	2.63
8-28	12002	12617	12291	12303	251	12065	12542	2.04
9-28	11519	12267	10453	11413	744	10706	12120	6.52
10-28	11846	11262	11895	11668	288	11395	11941	2.46
8-30	10536	10664	10243	10481	176	10314	10648	1.68
9-30	10025	10026	10108	10053	39	10016	10090	0.39
10-30	9469	10102	8963	9511	466	9069	9954	4.90
9-26-18	11654	11201	11778	11544	248	11309	11780	2.15
9-28-18	12280	12215	12120	12205	66	12143	12267	0.54
9-30-18	13004	12984	12540	12843	214	12639	13046	1.67
9-26-15	11938	12540	11954	12144	280	11878	12410	2.31
9-28-15	12550	12934	12149	12544	320	12240	12849	2.55
9-30-15	11874	11369	11484	11576	216	11370	11781	1.87
9-26-10	12143	11847	11788	11926	155	11778	12074	1.30
9-28-10	12984	12458	11740	12394	510	11910	12878	4.11
8-28-15	13161	12479	13403	13014	391	12643	13386	3.01
9-28-15	13309	13393	13903	13535	262	13286	13784	1.94
10-28-15	10209	9973		10091	118	9954	10228	1.17
9-26-8/6	13156	12598	13987	13247	571	12705	13789	4.31
9-26-10/6	13457	14875	14012	14115	583	13561	14669	4.13
9-26-10/3	13984	13548	11918	13150	889	12306	13994	6.76
9-26-15/3	13647	12987	13478	13371	280	13105	13637	2.09
9-28-10/6	12983	12385	13239	12869	358	12529	13209	2.78
9-28-10/3	12631	12809	12273	12571	223	12359	12783	1.77
9-28-10/3	11021	11743	12182	11649	479	11194	12103	4.11
9-28-15/3	13365	13259	13871	13498	267	13245	13752	1.98
9-30-10/6	12064	12361	11519	11981	349	11650	12312	2.91
9-30-15/3	11253	12841	12356	12150	664	11519	12781	5.47
9-28-15D	12786	13511	13428	13242	324	12934	13549	2.45
10-28-15D	13616	13971	14029	13872	183	13699	14045	1.32
9-28-15/3D	12819	12687	13129	12878	185	12702	13054	1.44

Table A.3. Gradation for Fine Aggregate.

Sieve Size	Percent Passing by Weight
3/8 in.	100.0
No. 4	99.1
No. 8	94.0
No. 16	81.5
No. 30	55.3
No. 50	22.9
No. 100	4.8

Table A.4. Gradation for Coarse Aggregate.

Sieve Size	Percent Passing by Weight
1/2 in.	100.0
3/8 in.	94.2
No. 4	16.4
No. 8	3.6
No. 16	1.1
No. 100	0.6

APPENDIX B

Appendix B contains the individual test results and statistical data of Chapter 3.
Appendix B also includes plots of the 90 percent confidence intervals.

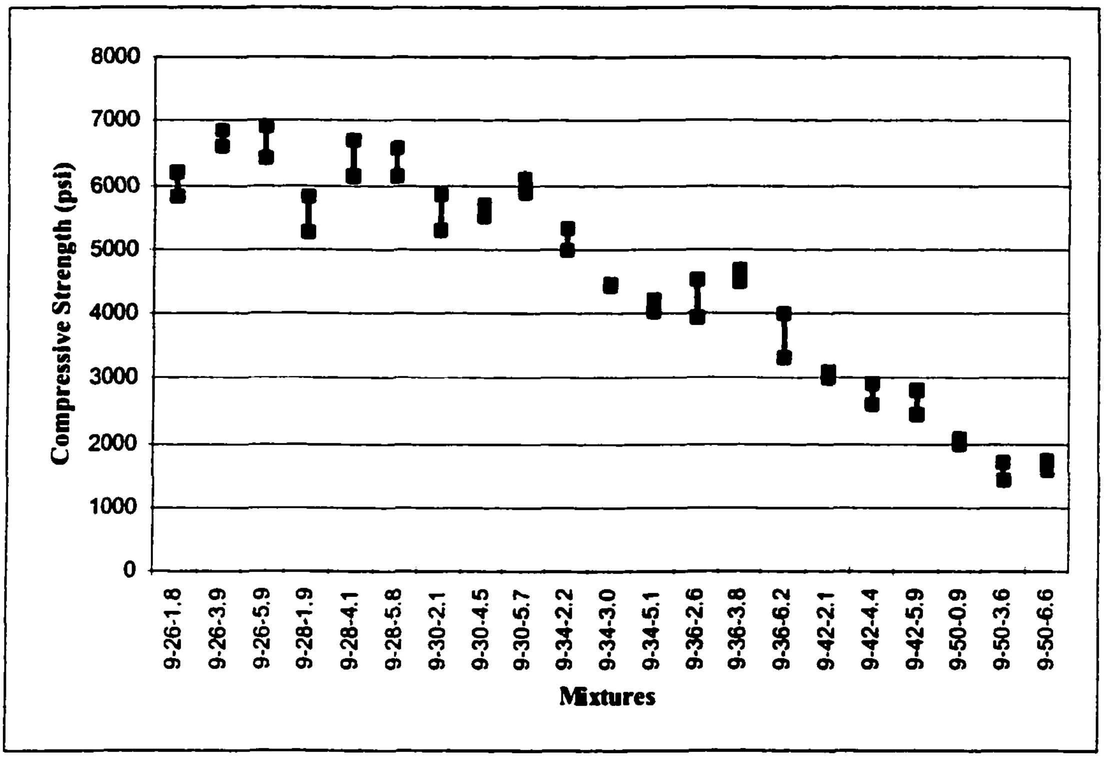

Figure B.1 90 % C.I. for One Day Compressive Strength Results.

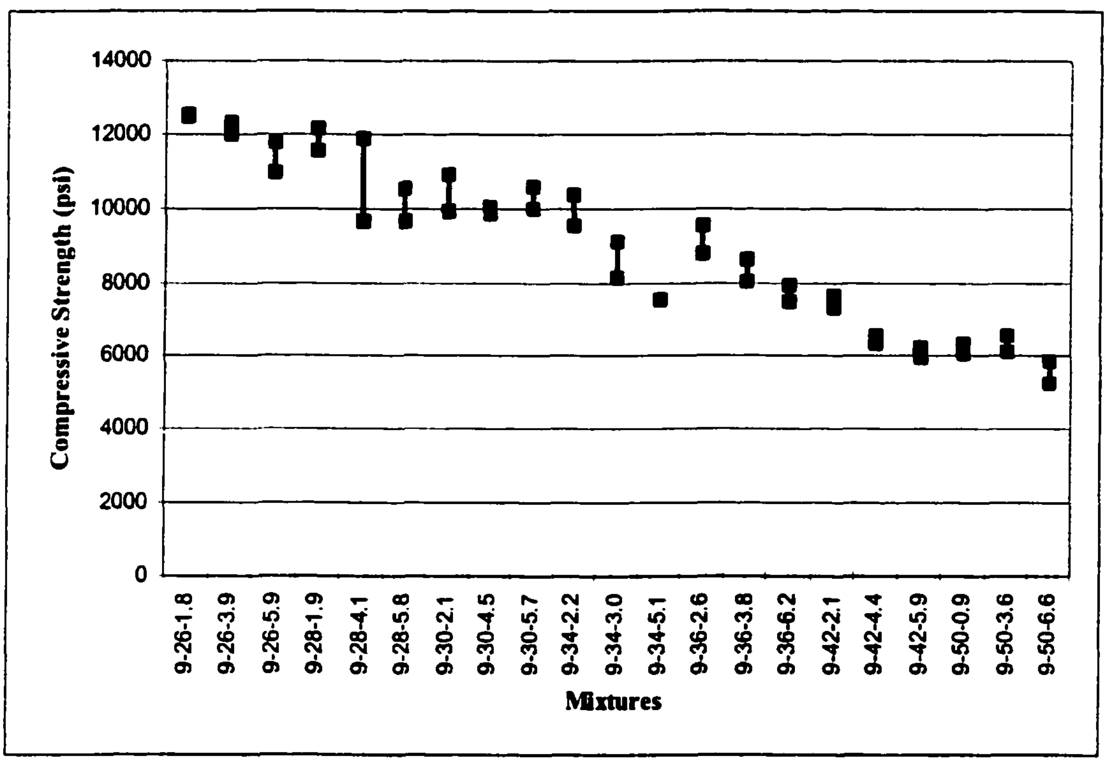

Figure B.2 90 % C.I. for 28-Day Compressive Strength Results.

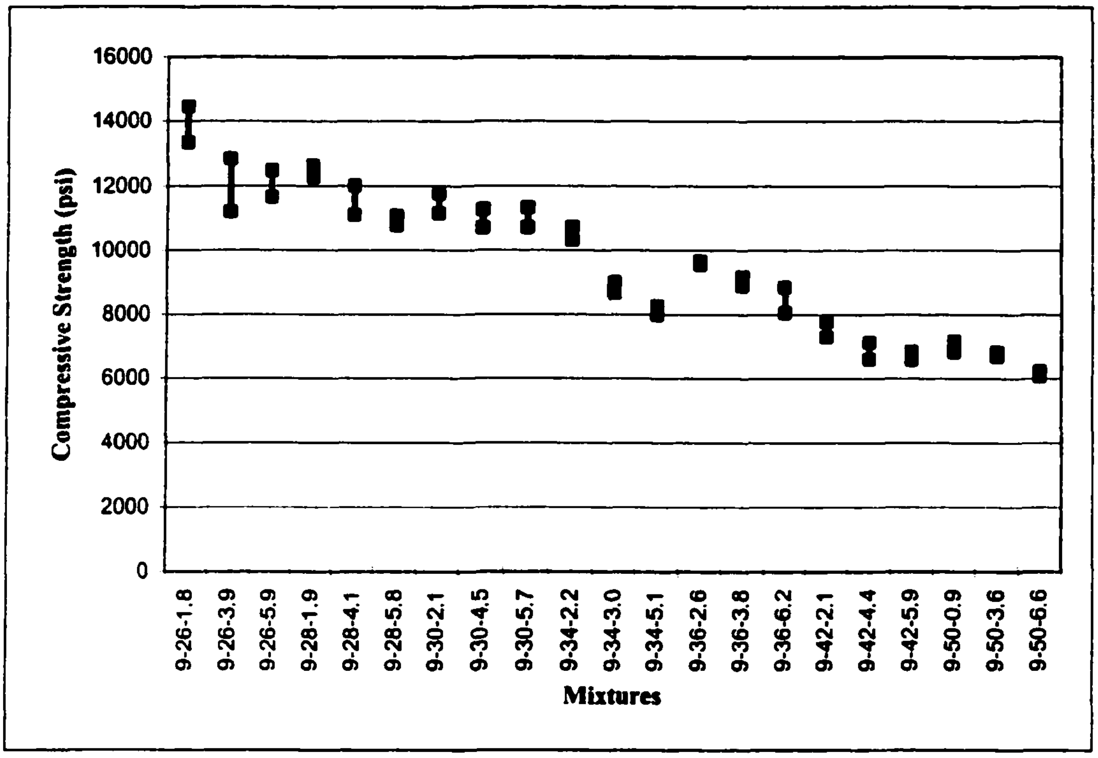

Figure B.3 90 % C.I. for 56-Day Compressive Strength Results.

Table B.1 Individual Results & Statistical Data for 1 day Compressive Strength Tests.

Mixture Designation	One Day Compressive Strength (psi) Cylinder 1	Cylinder 2	Cylinder 3	Average (psi)	Std. Dev. (psi)	90 Percent Confidence Intervals Lower Limit	Upper Limit	Coefficient of Variation
9-26-1.8	5986	6263	5787	6012	195	5827	6197	3.25
9-26-2.1	6568	6082	6761	6470	286	6199	6742	4.42
9-26-2.4	6884	6778	6817	6826	44	6785	6868	0.64
9-26-3.8	4678	5096	4722	4832	188	4654	5010	3.88
9-26-3.9	6553	6824	6831	6736	129	6613	6859	1.92
9-26-5.6	4397	4659	4617	4558	115	4449	4667	2.52
9-26-5.9	6369	6664	6987	6673	252	6434	6913	3.78
9-28-1.9	5823	5653	5142	5539	289	5265	5814	5.22
9-28-4.1	6268	6813	6162	6414	285	6143	6685	4.45
9-28-5.8	6104	6322	6657	6361	227	6145	6577	3.58
9-30-1.1	5685	6303	6465	6151	336	5832	6470	5.46
9-30-2.1	5917	5596	5221	5578	284	5308	5848	5.10
9-30-4.5	5687	5460	5693	5613	108	5510	5716	1.93
9-30-4.9	4735	4555	4449	4580	118	4468	4692	2.58
9-30-5.7	6127	5996	5882	6002	100	5907	6097	1.67
9-30-6.8	5493	5771	5882	5715	164	5560	5871	2.86
9-34-2.2	5416	5074	4994	5161	183	4988	5335	3.55
9-34-3.0	4458	4450	4446	4451	5	4447	4456	0.11
9-34-5.1	4275	4038	4070	4128	105	4028	4227	2.54
9-36-2.6	3798	4506	4362	4222	306	3932	4512	7.24
9-36-3.8	4461	4571	4715	4582	104	4484	4681	2.27
9-36-6.2	3786	4019	3170	3658	358	3318	3998	9.79
9-42-2.1	3107	2990	3043	3047	48	3001	3092	1.57
9-42-1.8	2606	2576	2883	2688	138	2557	2820	5.14
9-42-3.5	2855	3006	3001	2954	70	2887	3021	2.37
9-42-4.4	2947	2753	2547	2749	163	2594	2904	5.94
9-42-5.9	2897	2620	2392	2636	206	2440	2832	7.83
9-50-0.9	2026	1970	2078	2025	44	1983	2067	2.18
9-50-3.6	1761	1462	1459	1561	142	1426	1695	9.08
9-50-6.6	1554	1638	1750	1647	80	1571	1724	4.87

Table B.2 Individual Results & Statistical Data for 7 day Compressive Strength Tests.

Mixture Designation	Seven Day Compressive Strength (psi) Cylinder 1	Cylinder 2	Cylinder 3	Average (psi)	Std. Dev. (psi)	90 Percent Confidence Intervals Lower Limit	Upper Limit	Coefficient of Variation (%)
9-26-1.8	11938	11644	11297	11626	262	11378	11875	2.25
9-26-2.1	11038	10492	10835	10788	225	10574	11002	2.09
9-26-2.4	11432	11040	10646	11039	321	10735	11344	2.91
9-26-3.8	10282	10664	10381	10442	162	10289	10596	1.55
9-26-3.9	10716	10680	10373	10590	154	10444	10736	1.45
9-26-5.6	9169	9623	9013	9268	259	9023	9514	2.79
9-26-5.9	9732	9960	10072	9921	141	9787	10056	1.43
9-28-1.9	11082	10646	10227	10652	349	10320	10983	3.28
9-28-4.1	9542	9494	8382	9139	536	8630	9648	5.86
9-28-5.8	9685	9284	9587	9519	171	9357	9681	1.79
9-30-1.1	10146	9515	10484	10048	402	9667	10430	4.00
9-30-2.1	9222	8742	9200	9055	221	8845	9265	2.44
9-30-4.5	8347	8012	9580	8646	674	8006	9287	7.80
9-30-4.9	7137	7287	7458	7294	131	7169	7419	1.80
9-30-5.7	8767	9500	9136	9134	299	8850	9419	3.28
9-30-6.8	8728	8506	8541	8592	97	8499	8684	1.13
9-34-2.2	8672	8656	8948	8759	134	8631	8886	1.53
9-34-3.0	7237	6958	7075	7090	114	6981	7199	1.61
9-34-5.1	6682	6973	6560	6738	173	6574	6903	2.57
9-36-2.6	6866	7422	7336	7208	244	6976	7440	3.39
9-36-3.8	6942	6836	7117	6965	116	6855	7075	1.66
9-36-6.2	6613	6810	6950	6791	138	6660	6922	2.04
9-42-2.1	5979	5953	5879	5937	42	5897	5977	0.71
9-42-1.8	6761	6967	6544	6757	173	6593	6921	2.56
9-42-3.5	5086	5582	5508	5392	218	5185	5599	4.05
9-42-4.4	4628	4880	5306	4938	280	4672	5204	5.67
9-42-5.9	4720	4707	4501	4643	100	4547	4738	2.16
9-50-0.9	4762	4623	4794	4726	74	4656	4797	1.57
9-50-3.6	5114	5133	5418	5222	139	5090	5354	2.66
9-50-6.6	4169	4245	4548	4321	164	4165	4476	3.79

Table B.3 Individual Tests & Statistical Data for 28 day Compressive Strength Tests.

Mixture Designation	28 Day Compressive Strength (psi) Cylinder 1	Cylinder 2	Cylinder 3	Average (psi)	Std. Dev. (psi)	90 Percent Confidence Intervals Lower Limit	Upper Limit	Coefficient of Variation (%)
9-26-1.8	12473	12498	12579	12517	45	12474	12560	0.36
9-26-2.1	10971	11767	12019	11586	447	11162	12010	3.86
9-26-2.4	11945	12187	12434	12189	200	11999	12378	1.64
9-26-3.8	11515	11150	11140	11268	174	11103	11434	1.55
9-26-3.9	12302	12245	11931	12159	163	12004	12314	1.34
9-26-5.6	10725	10593	10368	10562	147	10422	10702	1.40
9-26-5.9	11134	11984	11010	11376	433	10965	11787	3.81
9-28-1.9	11972	12167	11453	11864	301	11578	12150	2.54
9-28-4.1	9339	12151	10811	10767	1148	9676	11858	10.67
9-28-5.8	10474	10308	9460	10081	444	9659	10502	4.41
9-30-1.1	12790	12301	11481	12191	540	11678	12704	4.43
9-30-2.1	10554	10970	9767	10430	499	9957	10904	4.78
9-30-4.5	9830	10052	10003	9962	95	9871	10052	0.96
9-30-4.9	8019	8642	8345	8335	254	8094	8577	3.05
9-30-5.7	10491	9826	10499	10272	315	9972	10572	3.07
9-30-6.8	9732	9802	9953	9829	92	9741	9917	0.94
9-34-2.2	9437	9879	10491	9936	432	9525	10346	4.35
9-34-3.0	7943	9139	8826	8636	506	8155	9117	5.86
9-34-5.1	7570	7516	7534	7540	22	7519	7561	0.30
9-36-2.6	8640	9564	9321	9175	391	8804	9546	4.26
9-36-3.8	8720	7986	8323	8343	300	8058	8628	3.60
9-36-6.2	7411	7835	7857	7701	205	7506	7896	2.67
9-42-2.1	7712	7367	7358	7479	165	7322	7636	2.20
9-42-1.8	8434	8765	8875	8691	187	8513	8869	2.16
9-42-3.5	6785	6569	6107	6487	283	6218	6756	4.36
9-42-4.4	6313	6525	6564	6467	110	6363	6572	1.71
9-42-5.9	6039	6280	6009	6109	121	5994	6225	1.99
9-50-0.9	6201	6417	6028	6215	159	6064	6366	2.56
9-50-3.6	6443	6563	6015	6340	235	6117	6564	3.71
9-50-6.6	5806	5765	5098	5556	325	5248	5865	5.84

Table B.4 Individual Tests & Statistical Data for 56 day Compressive Strength Tests.

Mixture Designation	56 Day Compressive Strength (psi)			Average (psi)	Std. Dev. (psi)	90 Percent Confidence Intervals		Coefficient of Variation (%)
	Cylinder 1	Cylinder 2	Cylinder 3			Lower Limit	Upper Limit	
9-26-1.8	14218	14399	13080	13899	584	13345	14453	4.20
9-26-2.4	13140	12993	12754	12962	159	12811	13113	1.23
9-26-3.8	12174	12165	11927	12089	114	11980	12197	0.95
9-26-3.9	10854	12352	12875	12027	856	11214	12840	7.12
9-26-5.6	10976	11670	11232	11293	287	11021	11565	2.54
9-26-5.9	11674	12662	11889	12075	424	11672	12478	3.51
9-28-1.9	12223	12679	12463	12455	186	12278	12632	1.50
9-28-4.1	12153	10989	11573	11572	475	11120	12023	4.11
9-28-5.8	10901	10767	11151	10940	159	10789	11091	1.45
9-30-1.1	13221	13971	13597	13596	306	13306	13887	2.25
9-30-2.1	11676	11735	11020	11477	324	11169	11785	2.82
9-30-4.5	10587	11312	11094	10998	304	10709	11286	2.76
9-30-4.9	8883	8899	8844	8875	23	8853	8897	0.26
9-30-5.7	10908	10713	11476	11032	324	10725	11340	2.93
9-30-6.8	10002	10101	10169	10091	69	10026	10156	0.68
9-34-2.2	10403	10363	10831	10532	212	10331	10733	2.01
9-34-3.0	9054	8861	8603	8839	185	8664	9015	2.09
9-34-5.1	8314	7972	8080	8122	143	7986	8258	1.76
9-36-2.6	9616	9695	9547	9619	60	9562	9677	0.63
9-36-3.8	9169	8832	9122	9041	149	8899	9183	1.65
9-36-6.2	8208	9044	8106	8453	420	8054	8852	4.97
9-42-2.1	7320	7411	7876	7536	244	7304	7767	3.23
9-42-1.8	8943	9065	9140	9049	81	8972	9126	0.90
9-42-3.5	6689	6833	6774	6765	59	6709	6821	0.87
9-42-4.4	7253	6627	6706	6862	278	6598	7126	4.06
9-42-5.9	6655	6577	6895	6709	135	6580	6838	2.02
9-50-0.9	6926	6821	7234	6994	175	6827	7160	2.51
9-50-3.6	6662	6767	6837	6755	72	6687	6824	1.06
9-50-6.6	6073	6260	6102	6145	82	6067	6223	1.34

APPENDIX C

Appendix C contains the individual test results and statistical data of Chapter 4. Appendix C also includes plots of the 90 percent confidence intervals.

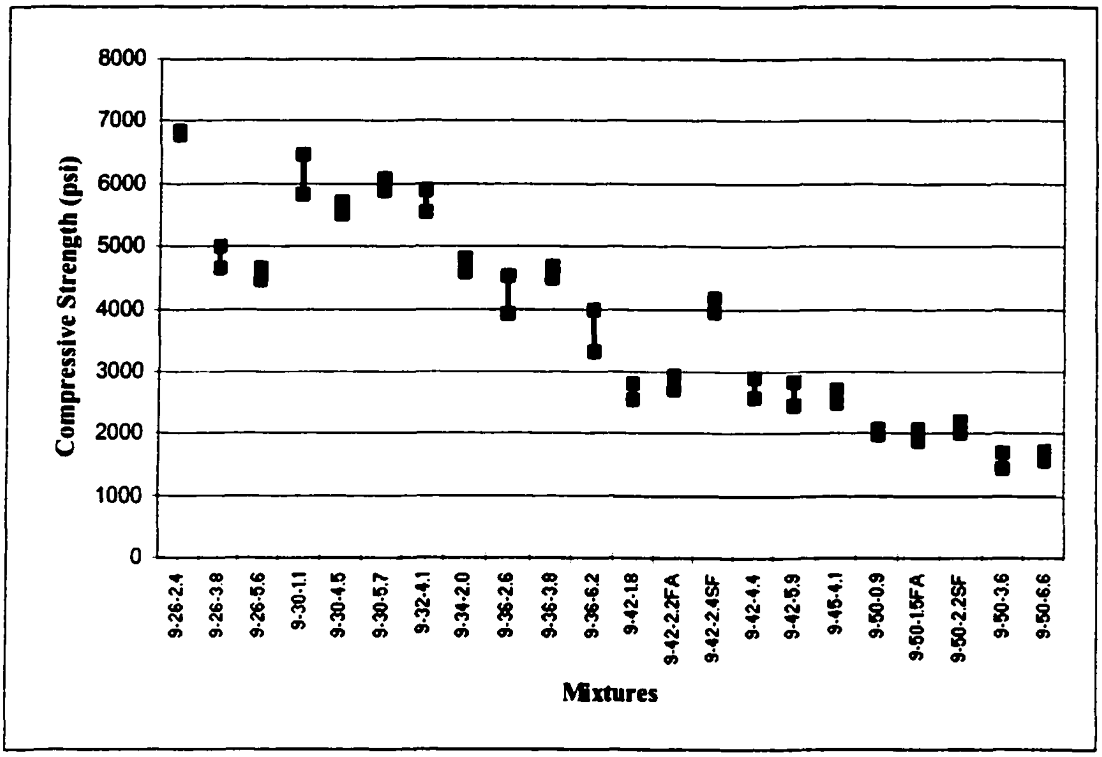

Figure C.1 90 Percent Confidence Intervals for One Day Compressive Strength.

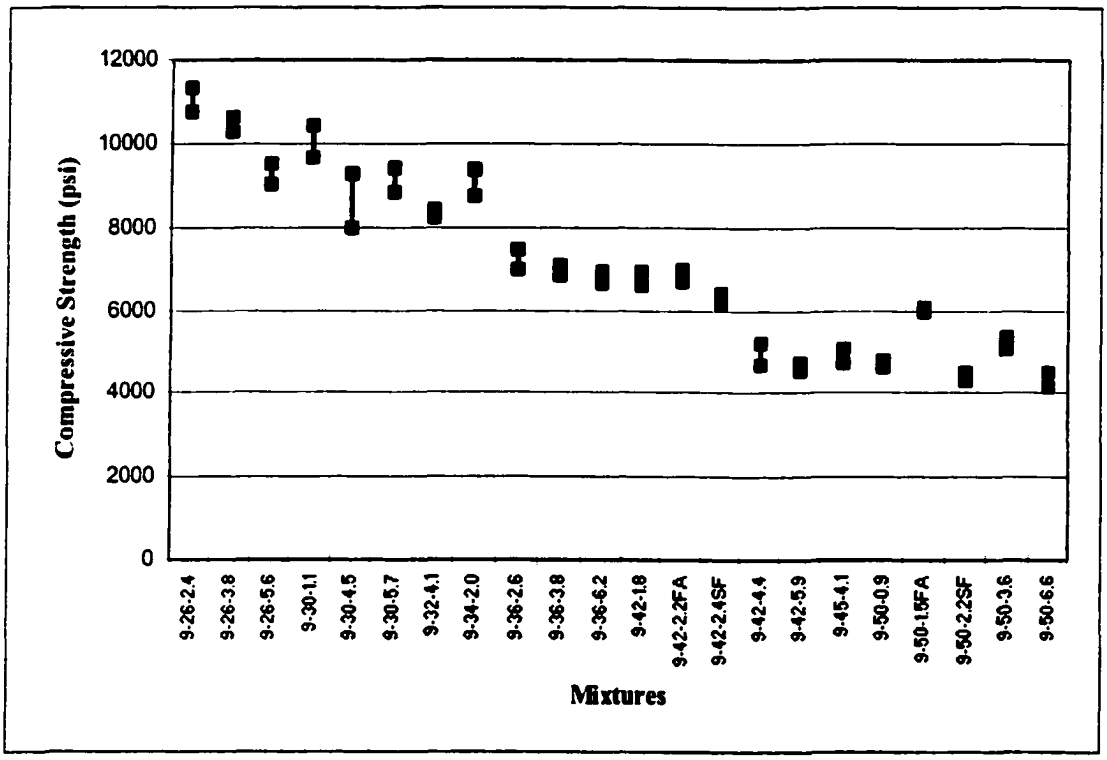

Figure C.2 90 Percent Confidence Intervals for Seven Day Compressive Strength.

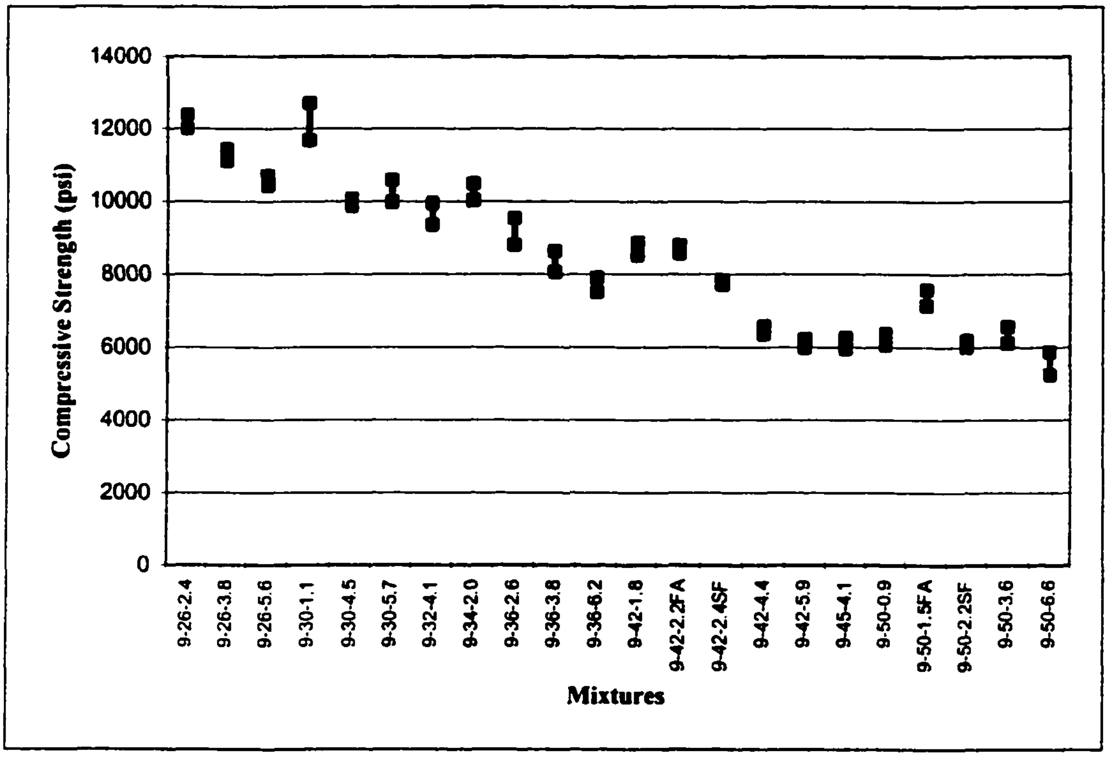

Figure C.3 90 Percent Confidence Intervals for 28 Day Compressive Strength.

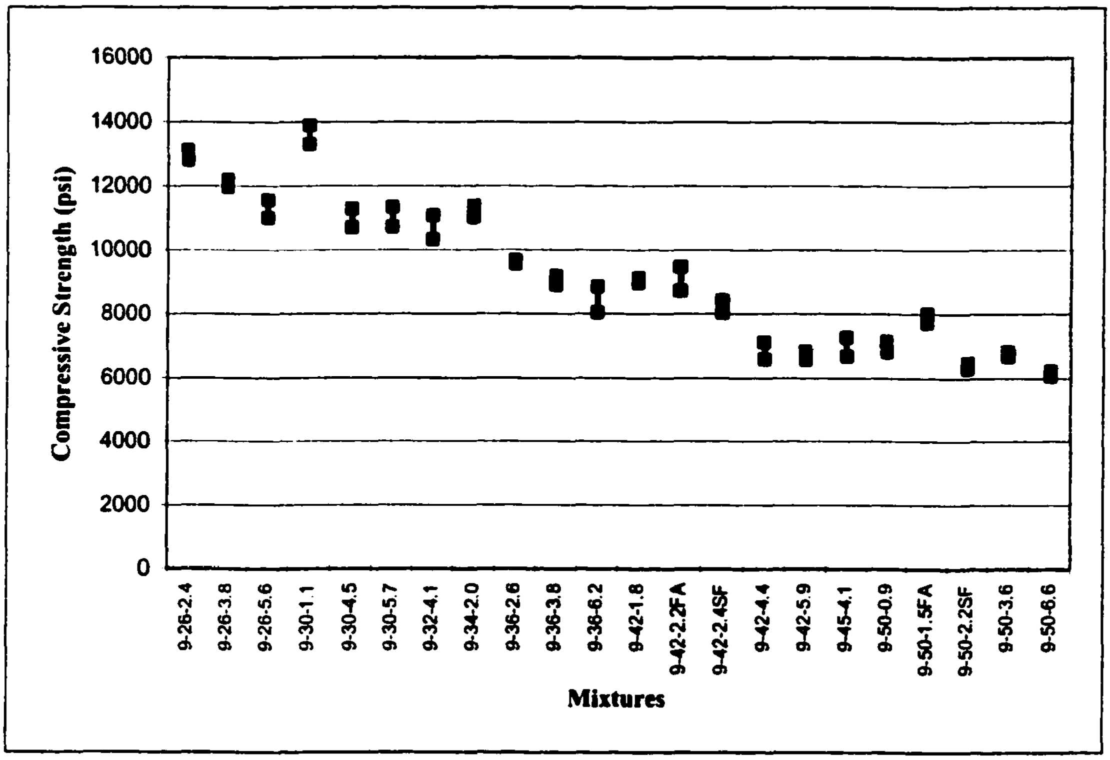

Figure C.4 90 Percent Confidence Intervals for 56 Day Compressive Strength.

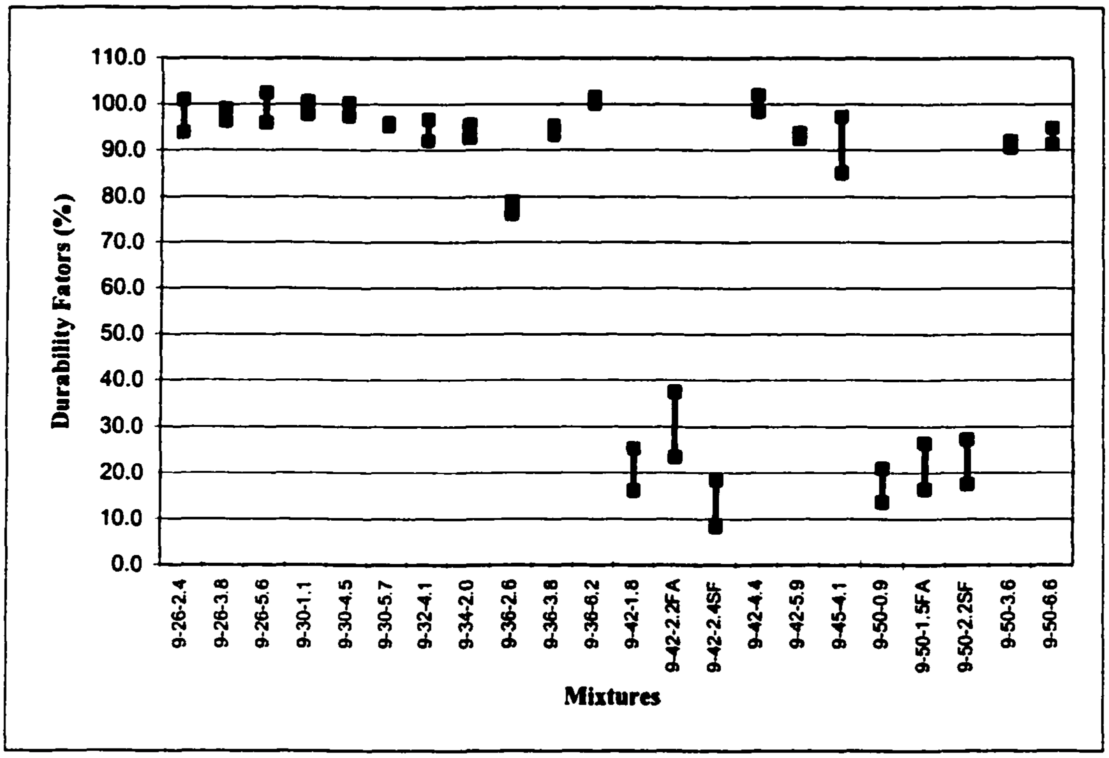

Figure C.5 90 Percent Confidence Intervals for Durability Factors.

Table C.1 Individual Results and Statistical Data for One Day Compressive Strength Tests.

Mixture Designation	One Day Compressive Strength (psi)			Average (psi)	Std. Dev. (psi)	90 Percent Confidence Intervals		Coefficient of Variation
	Cylinder 1	Cylinder 2	Cylinder 3			Lower Limit	Upper Limit	
9-26-2.4	6884	6778	6817	6826	44	6785	6868	0.64
9-26-3.8	4678	5096	4722	4832	188	4654	5010	3.88
9-26-5.6	4397	4659	4617	4558	115	4449	4667	2.52
9-30-1.1	5685	6303	6465	6151	336	5832	6470	5.46
9-30-4.5	5687	5460	5693	5613	108	5510	5716	1.93
9-30-5.7	6127	5996	5882	6002	100	5907	6097	1.67
9-32-4.1	5978	5557	5640	5725	182	5552	5898	3.18
9-34-2.2	4540	4727	4839	4702	123	4585	4819	2.62
9-36-2.6	3798	4506	4362	4222	306	3932	4512	7.24
9-36-3.8	4461	4571	4715	4582	104	4484	4681	2.27
9-36-6.2	3786	4019	3170	3658	358	3318	3998	9.79
9-42-1.8	2606	2576	2883	2688	138	2557	2820	5.14
9-42-2.2FA	2973	2787	2720	2827	107	2725	2928	3.79
9-42-2.4SF	3951	4226	4039	4072	115	3963	4181	2.82
9-42-4.4	2947	2753	2547	2749	163	2594	2904	5.94
9-42-5.9	2897	2620	2392	2636	206	2440	2832	7.83
9-45-4.1	2614	2729	2451	2598	114	2490	2706	4.39
9-50-0.9	2026	1970	2078	2025	44	1983	2067	2.18
9-50-1.5FA	2091	1844	1987	1974	101	1878	2070	5.13
9-50-2.2SF	2015	2238	2069	2107	95	2017	2198	4.51
9-50-3.6	1761	1462	1459	1561	142	1426	1695	9.08
9-50-6.6	1554	1638	1750	1647	80	1571	1724	4.87

Table C.2 Individual Results and Statistical Data for Seven Day Compressive Strength Tests.

Mixture Designation	Seven Day Compressive Strength (psi)			Average (psi)	Std. Dev. (psi)	90 Percent Confidence Intervals		Coefficient of Variation (%)
	Cylinder 1	Cylinder 2	Cylinder 3			Lower Limit	Upper Limit	
9-26-2.4	11432	11040	10646	11039	321	10735	11344	2.91
9-26-3.8	10282	10664	10381	10442	162	10289	10596	1.55
9-26-5.6	9169	9623	9013	9268	259	9023	9514	2.79
9-30-1.1	10146	9515	10484	10048	402	9667	10430	4.00
9-30-4.5	8347	8012	9580	8646	674	8006	9287	7.80
9-30-5.7	8767	9500	9136	9134	299	8850	9419	3.28
9-32-4.1	8348	8446	8162	8319	118	8207	8431	1.42
9-34-2.0	8748	8957	9530	9078	331	8764	9392	3.64
9-36-2.6	6866	7422	7336	7208	244	6976	7440	3.39
9-36-3.8	6942	6836	7117	6965	116	6855	7075	1.66
9-36-6.2	6613	6810	6950	6791	138	6660	6922	2.04
9-42-1.8	6761	6967	6544	6757	173	6593	6921	2.56
9-42-2.2FA	6914	6961	6637	6837	143	6702	6973	2.09
9-42-2.4SF	6361	6096	6406	6288	137	6158	6418	2.18
9-42-4.4	4628	4880	5306	4938	280	4672	5204	5.67
9-42-5.9	4720	4707	4501	4643	100	4547	4738	2.16
9-45-4.1	4746	4830	5121	4899	161	4746	5052	3.28
9-50-0.9	4762	4623	4794	4726	74	4656	4797	1.57
9-50-1.5FA	6077	6021	5983	6027	39	5990	6064	0.64
9-50-2.2SF	4443	4251	4481	4392	101	4296	4487	2.29
9-50-3.6	5114	5133	5418	5222	139	5090	5354	2.66
9-50-6.6	4169	4245	4548	4321	164	4165	4476	3.79

Table C.3 Individual Results and Statistical Data for 28 Day Compressive Strength Tests.

Mixture Designation	28 Day Compressive Strength (psi)			Average (psi)	Std. Dev. (psi)	90 Percent Confidence Intervals		Coefficient of Variation (%)
	Cylinder 1	Cylinder 2	Cylinder 3			Lower Limit	Upper Limit	
9-26-2.4	11945	12187	12434	12189	200	11999	12378	1.64
9-26-3.8	11515	11150	11140	11268	174	11103	11434	1.55
9-26-5.6	10725	10593	10368	10562	147	10422	10702	1.40
9-30-1.1	12790	12301	11481	12191	540	11678	12704	4.43
9-30-4.5	9830	10052	10003	9962	95	9871	10052	0.96
9-30-5.7	10491	9826	10499	10272	315	9972	10572	3.07
9-32-4.1	9311	9583	10046	9647	303	9359	9935	3.15
9-34-2.0	10428	10398	9919	10248	233	10027	10470	2.28
9-36-2.6	8640	9564	9321	9175	391	8804	9546	4.26
9-36-3.8	8720	7986	8323	8343	300	8058	8628	3.60
9-36-6.2	7411	7835	7857	7701	205	7506	7896	2.87
9-42-1.8	8434	8765	8875	8691	187	8513	8869	2.16
9-42-2.2FA	8545	8665	8840	8683	121	8568	8798	1.39
9-42-2.4SF	7696	7786	7839	7774	59	7718	7830	0.76
9-42-4.4	6313	6525	6564	6467	110	6363	6572	1.71
9-42-5.9	6039	6280	6009	6109	121	5994	6225	1.99
9-45-4.1	5912	6138	6280	6110	152	5966	6254	2.48
9-50-0.9	6201	6417	6028	6215	159	6064	6366	2.56
9-50-1.5FA	7078	7354	7645	7359	232	7139	7579	3.15
9-50-2.2SF	5961	6174	6170	6102	99	6007	6196	1.63
9-50-3.6	6443	6563	6015	6340	235	6117	6564	3.71
9-50-6.6	5806	5765	5098	5556	325	5248	5865	5.84

Table C.4 Individual Results and Statistical Data for 56 Day Compressive Strength Tests.

Mixture Designation	56 Day Compressive Strength (psi)			Average (psi)	Std. Dev. (psi)	90 Percent Confidence Intervals		Coefficient of Variation (%)
	Cylinder 1	Cylinder 2	Cylinder 3			Lower Limit	Upper Limit	
9-26-2.4	13140	12993	12754	12962	159	12811	13113	1.23
9-26-3.8	12174	12165	11927	12089	114	11980	12197	0.95
9-26-5.6	10976	11670	11232	11293	287	11021	11565	2.54
9-30-1.1	13221	13971	13597	13596	306	13306	13887	2.25
9-30-4.5	10587	11312	11094	10998	304	10709	11286	2.76
9-30-5.7	10908	10713	11476	11032	324	10725	11340	2.93
9-32-4.1	10763	10188	11160	10704	399	10325	11083	3.73
9-34-2.0	11152	11006	11446	11201	183	11028	11375	1.63
9-36-2.6	9616	9695	9547	9619	60	9562	9677	0.63
9-36-3.8	9169	8832	9122	9041	149	8899	9183	1.65
9-36-6.2	8208	9044	8106	8453	420	8054	8852	4.97
9-42-1.8	8943	9065	9140	9049	81	8972	9126	0.90
9-42-2.2FA	9389	8538	9360	9096	395	8721	9470	4.34
9-42-2.4SF	8511	8026	8150	8229	206	8034	8424	2.50
9-42-4.4	7253	6627	6706	6862	278	6598	7126	4.06
9-42-5.9	6655	6577	6895	6709	135	6580	6838	2.02
9-45-4.1	6978	7371	6603	6984	314	6686	7282	4.49
9-50-0.9	6926	6821	7234	6994	175	6827	7160	2.51
9-50-1.5FA	8062	7827	7723	7871	142	7736	8005	1.80
9-50-2.2SF	6500	6391	6263	6385	97	6293	6477	1.52
9-50-3.6	6662	6767	6837	6755	72	6687	6824	1.06
9-50-6.6	6073	6260	6102	6145	82	6067	6223	1.34

Table C.5 Individual Results and Statistical Data for the Freeze-Thaw Tests.

Mixture Designation	Durability Factor (%) Specimens				Average DF (%)	Std. Dev. (%)	90 Percent Confidence Intervals		Coefficient of Variation (%)
	1	2	3	4			Lower Limit	Upper Limit	
9-26-2.4	96.0	104.7	95.5	93.7	97.5	4.3	94.0	101.0	4.37
9-26-3.8	100.1	97.6	95.4	97.7	97.7	1.7	96.3	99.1	1.70
9-26-5.6	96.1	96.4	98.7	105.8	99.3	3.9	96.0	102.5	3.94
9-30-1.1	100.7	99.7	96.4	99.9	99.2	1.6	97.8	100.5	1.66
9-30-4.5	97.6	96.8	100.8	100.3	98.9	1.7	97.5	100.3	1.73
9-30-5.6	96.0	95.2	95.6		95.6	0.3	95.3	95.9	0.34
9-32-4.1	89.5	96.5	95.7	95.8	94.4	2.8	92.0	96.7	3.00
9-34-2.0	94.4	94.6	91.5	96.3	94.2	1.7	92.8	95.6	1.83
9-36-2.6	75.0	79.4	78.1	77.8	77.6	1.6	76.3	78.9	2.07
9-36-3.8	94.2	96.3	93.0	94.0	94.4	1.2	93.4	95.4	1.27
9-36-6.2	100.6	100.6	99.9	102.3	100.9	0.9	100.1	101.6	0.88
9-42-1.8	16.5	18.3	17.9	30.3	20.8	5.6	16.2	25.3	26.77
9-42-2.2FA	18.5	27.5	41.7	34.2	30.5	8.5	23.4	37.5	28.04
9-42-2.4SF	4.5	18.8	19.3	10.8	13.4	6.1	8.3	18.4	45.86
9-42-4.4	101.5	96.8	102.4	100.1	100.2	2.1	98.5	101.9	2.12
9-42-5.9	94.2	93.4	92.0	93.0	93.2	0.8	92.5	93.8	0.85
9-45-4.1	99.5	79.4	93.1	93.1	91.3	7.3	85.2	97.3	8.04
9-50-0.9	10.2	19.6	17.2	22.2	17.3	4.5	13.6	21.0	25.81
9-50-1.5FA	26.7	26.1	21.6	11.5	21.5	6.1	16.5	26.5	28.34
9-50-2.2SF	28.5	27.9	18.3	15.0	22.4	5.9	17.6	27.3	26.29
9-50-3.6	90.1	91.0	92.4	91.8	91.3	0.9	90.6	92.0	0.95
9-50-6.6	92.4	94.9	95.1	89.9	93.1	2.1	91.3	94.8	2.28

APPENDIX D

Appendix D includes individual tests results and statistical data from Chapter 5. Appendix D also contains plots of the 90 percent confidence intervals.

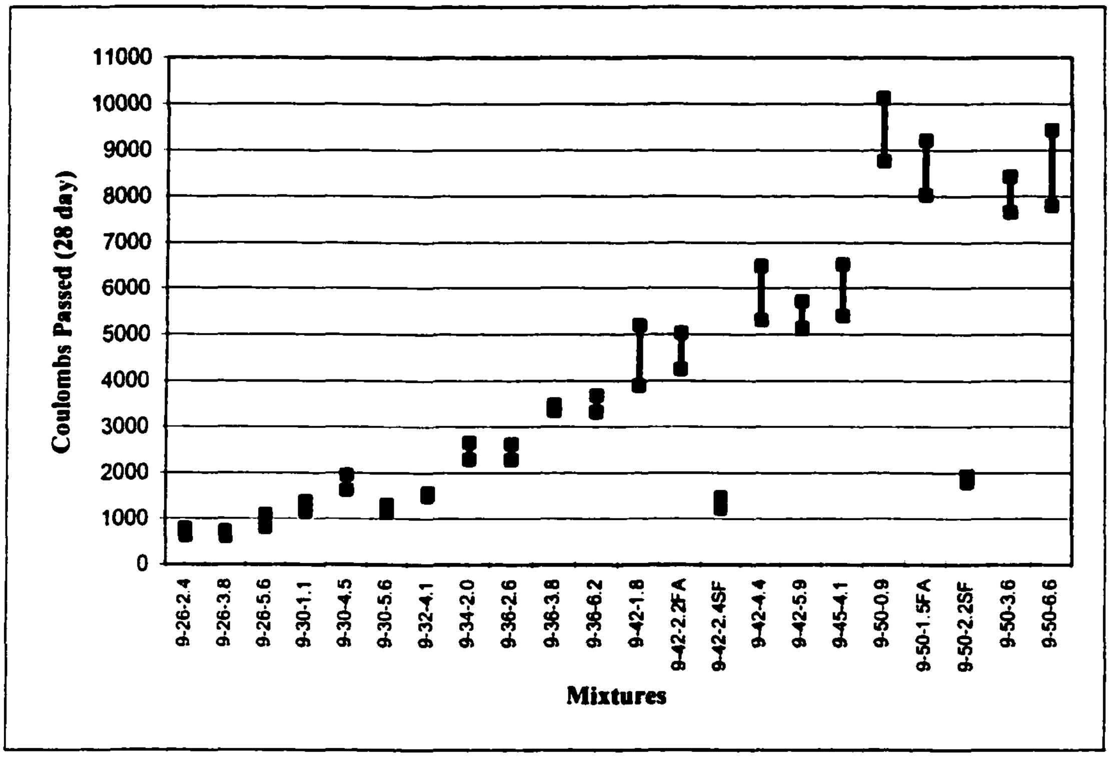

Figure D.1 90 Percent Confidence Intervals for 28 Day RCIP.

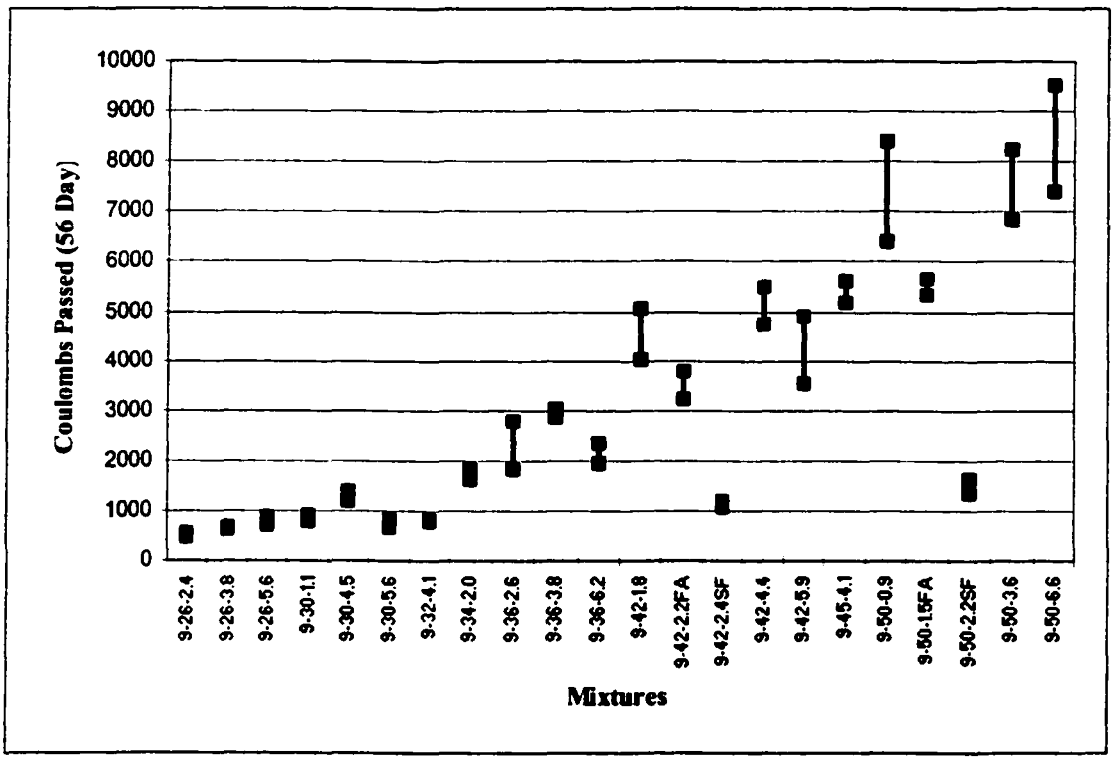

Figure D.2 90 Percent Confidence Intervals for 56 Day RCIP.

Table D.1 Individual Tests and Statistical Data for 28 Day RCIP.

Mixture Designation	Number of Coulombs Passed Specimens 1	2	3	4	Average (coulombs)	Std. Dev. (coulombs)	90 Percent Confidence Intervals Lower Limit	Upper Limit	Coefficient of Variation (%)
9-26-2.4	573	657	846	765	710	104	625	796	14.61
9-26-3.8	558	738	645	742	671	76	608	733	11.30
9-26-5.6	1249	813	908	855	956	172	815	1098	18.02
9-30-1.1	1437	1274	1070	1298	1270	131	1162	1378	10.32
9-30-4.5	1743	2049	1854	1508	1789	196	1628	1949	10.93
9-30-5.6	1083	1320	1326	1159	1222	105	1136	1308	8.55
9-32-4.1	1515	1612	1524	1483	1534	48	1494	1573	3.12
9-34-2.0	2623	2146	2383	2733	2471	226	2285	2657	9.16
9-36-2.6	2290	2812	2340	2371	2453	209	2281	2625	8.52
9-36-3.8	3353	3327	3452	3533	3416	82	3349	3484	2.40
9-36-6.2	3282	3560	3837	3324	3501	221	3319	3683	6.32
9-42-1.8	3484	4912	4187	5602	4546	791	3895	5197	17.41
9-42-2.2FA	4588	4309	5435	4223	4639	479	4245	5033	10.33
9-42-2.4SF	1238	1506	1149	1478	1343	153	1217	1468	11.38
9-42-4.4	5496	5565	7150	5409	5905	721	5312	6498	12.21
9-42-5.9	5278	5102	6029	5313	5431	355	5139	5722	6.53
9-45-4.1	6621	5169	5409	6679	5970	686	5405	6534	11.49
9-50-0.9	9411	10760	9134	8493	9450	827	8770	10129	8.75
9-50-1.5FA	9160	9471	7724	8138	8623	716	8034	9212	8.30
9-50-2.2SF	1964	1936	1791	1780	1868	83	1800	1936	4.44
9-50-3.6	7358	8599	7912	8357	8057	473	7668	8445	5.87
9-50-6.6	7318	9867	8066	9239	8623	993	7806	9439	11.51

Table D.2 Individual Tests and Statistical Data for 56 Day RCIP Tests.

Mixture Designation	Number of Coulombs Passed Specimens				Average (coulombs)	Std. Dev. (coulombs)	90 Percent Confidence Intervals		Coefficient of Variation (%)
	1	2	3	4			Lower Limit	Upper Limit	
9-26-2.4	456	497	543	584	520	48	480	560	9.25
9-26-3.8	698	623	683	637	660	31	635	686	4.71
9-26-5.6	904	722	713	855	799	83	730	867	10.38
9-30-1.1	746	963	895	793	849	85	779	919	10.00
9-30-4.5	1088	1299	1358	1377	1281	115	1186	1375	8.97
9-30-5.6	804	816	784	621	756	79	691	821	10.44
9-32-4.1	822	863	803	755	811	39	779	843	4.79
9-34-2.0	1553	1883	1759	1759	1739	118	1641	1836	6.81
9-36-2.6	2023	3327	1828	2097	2319	590	1833	2804	25.46
9-36-3.8	2808	2951	3112	2923	2949	109	2859	3038	3.68
9-36-6.2	1771	2343	2178	2372	2166	240	1969	2363	11.07
9-42-1.8	5339	3640	4343	4880	4551	633	4030	5071	13.91
9-42-2.2FA	3236	3175	3613	4033	3514	343	3232	3797	9.77
9-42-2.4SF	1206	1164	1152	1013	1134	73	1074	1193	6.40
9-42-4.4	5190	4433	5727	5088	5110	460	4731	5488	9.00
9-42-5.9	5623	3759	3881	3602	4216	818	3543	4889	19.41
9-45-4.1	5508	5673	5057		5413	260	5199	5627	4.81
9-50-0.9	6467	9144	6619		7410	1228	6400	8420	16.57
9-50-1.5FA	5322	5830	5376	5434	5491	200	5326	5655	3.64
9-50-2.2SF	1269	1542	1642		1484	158	1355	1614	10.62
9-50-3.6	7298	8690	6328	7879	7549	861	6841	8257	11.40
9-50-6.6	6623	10193	8299	8751	8467	1274	7419	9514	15.04

APPENDIX E

Appendix E contains the individual tests results for Chapter 6.

The following pages contain the prestress loss calculations (AASHTO and PCI) for Girder/Beam 1.

BEAM 1

Section Properties

Sb =	899.63	in^3
Area =	163.25	in^2
y =	13.75	in.
cgs =	6.4	in.
e =	7.35	

Strand Properties

fse =	185	ksi
Area =	0.217	in^2
Fse =	401.45	kips
Total number of strands =	10	

Concrete Properties

fci =	8.696	ksi
fc =	10	ksi
Total Air Content =	2%	
Unit Weight =	151.1	lb/ft^3

Number of Strands (1)	Distance from bottom fiber (2)	1*2
3	2	6
3	4	12
1	6	6
1	8	8
1	10	10
1	22	22

fbot=-Fse(1/A+e/Sb)

Fse = 401.45

1/A = 0.006125574

e/sb = 0.008170025

fbot = -5.738968511 ksi

fbot/fci = -0.659954981	**-66.00 %**

AASHTO Losses

ELASTIC SHORTENING

Modulus of Elasticity

concrete =	5600 ksi (1 day)
strand =	28500 ksi
n =	5.09

Strands

total # =	10	
total area =	2.17 in^2	
f_{pu} =	270 ksi	
tension after stressing =	204.25 ksi	

elongation of the strands (in)

1	8.750
2	8.750
3	8.750
4	8.250
5	8.500
6	8.500
7	8.625
8	8.500
9	8.625
10	8.750
average elongation =	8.600 in.
length of strand =	100 ft

$f_{cgp} = F/A + Fe^2/I - M_G e/I$

F (Ppi) =	443.22 kips	
M_G =	0 at ends	
M_G =	148.00 k - in	at center
w =	171.30 lb/ft	
f_{cgp} =	4.65 ksi	at ends
f_{cgp} =	4.56 ksi	at center

$\Delta f_{pES} = (E_p/E_{ci})f_{cgp}$ section 5.9.5.2.3a

Δf_{pES} =	23.67 ksi	at ends
Δf_{pES} =	23.22 ksi	at center

CREEP

$\Delta f_{pCR} = 12.0 f_{cgp} - 7.0 \Delta f_{cdp}$

Δf_{cdp} =	0 ksi	
f_{cgp} =	4.65 ksi	at ends
f_{cgp} =	4.56 ksi	at center

Δf_{pCR} =	55.81 ksi	at ends
Δf_{pCR} =	54.75 ksi	at center

SHRINKAGE

$\Delta f_{pSR} = (17.0 - 0.150\ H)$

H = 65 %

Δf_{pSR} =	7.25

RELAXATION

$\Delta f_{pR2} = (20 - 0.4\Delta f_{pES} - 0.2(\Delta f_{pSR} + \Delta f_{pCR}))*0.30$

Δf_{pES} =	23.67 ksi	at ends
Δf_{pES} =	23.22 ksi	at center
Δf_{pSR} =	7.25	
Δf_{pCR} =	55.81 ksi	at ends
Δf_{pCR} =	54.75 ksi	at center

Δf_{pR2} =	-0.62368 ksi	at ends
Δf_{pR2} =	-0.50665 ksi	at center

$\Delta f_{pR1} = (\log(24.0t)/40.0)(f_{pj}/f_{py} - 0.55)f_{pj}$

t =	1 day
f_{pj} =	204.25 ksi
f_{py} =	270 ksi

Δf_{pR1} =	1.46 ksi

TOTAL LOSSES

$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1} + \Delta f_{pR2}$

Δf_{pT} =	87.56 at ends
Δf_{pT} =	86.17 at center

PCI Losses

ELASTIC SHORTENING

Modulus of Elasticity

concrete =	5600 ksi (1 day)
concrete =	6000 ksi (28 days)
strand =	28500 ksi
$n_{one\ day}$ =	5.09
$n_{28\ days}$ =	4.75

Strands

total # =	10	
total area =	2.17 in^2	
tension after stressing =	204.25 ksi	
elongation of the strands (in)		
1	8.750	
2	8.750	
3	8.750	
4	8.250	
5	8.500	
6	8.500	
7	8.625	
8	8.500	
9	8.625	
10	8.750	
average =	8.600	in.
length of strand =	100	ft.

Ppi = 443.22 kips
Kcir = 0.895

$f_{cir} = K_{cir}(P_{pi}(1/A_g + e^2/I_g)) - f_g$

e =	7.35 in.	
A_g =	163.25 in^2	
I_g =	12370 in^4	
f_g =	0 ksi	at ends
f_g =	0.088 ksi	at center
f_{cir} =	4.16 ksi	at ends
f_{cir} =	4.07 ksi	at center

$f_g = M_G e/I$

w =	171.30 lb/ft
M_G =	148.00 k - in

$ES = K_{es} n f_{cir}$

K_{es} = 1.0

ES =	**21.18 ksi**	at ends

CREEP

$CR = K_{cr} n(f_{cir} - f_{cds})$

K_{cr} =	2.0 for p/s members	
f_{cir} =	4.16 ksi	at ends
f_{cir} =	4.07 ksi	at center
f_{cds} =	0 ksi	

CR =	39.54 ksi	at ends
CR =	38.71 ksi	at center

SHRINKAGE

$SH = 8.2 \cdot 10^{-6} K_{sh} E_s (1 - 0.06V/S)(100 - RH)$

RH =	65 %
Volume =	47016 in^3
Surface Area =	28224 in^2
V/A =	1.67
K_{sh} =	1
E_s =	28500 ksi

SH =	7.36 ksi

ES =	**20.74 ksi** at center

RELAXATION

RE = (K_{re} - J(SH + CR + ES))C

K_{re} = 5

J = 0.040

SH = 7.36 ksi

CR = 39.54 ksi at ends

CR = 38.71 ksi at center

ES = 21.18 ksi at ends

ES = 20.74 ksi at center

C = 1.05

f_{pi} = 204.25 ksi

f_{pu} = 270 ksi

f_{pi}/f_{pu} = 0.76

RE =	2.39 ksi at ends
RE =	2.44 ksi at center

Total Losses

ES =	21.18 ksi	at ends
ES =	20.74 ksi	at center
CR =	39.54 ksi	at ends
CR =	38.71 ksi	at center
SH =	7.36 ksi	
RE =	2.39 ksi	at ends
RE =	2.44 ksi	at center

Total Losses at Ends =	**70.48 ksi**
Total Losses at Center =	**69.25 ksi**

The following pages contain the prestress loss calculations (AASHTO and PCI) for Girder/Beam 2.

BEAM 2

Section Properties

Sb = 899.63 in^3

Area = 163.25 in^2

y = 13.75 in.

cgs = 6 in.

e = 7.75

Strand Properties

fse = 185 ksi

Area = 0.217 in^2

Fse = 361.305 kips

Total number of strands = 9

Concrete Properties

fci = 6.127 ksi

fc = 10 ksi

Total Air Content = 6%

Unit Weight = 146.9 lb/ft^3

Number of Strands (1)	Distance from bottom fiber (2)	1*2
3	2	6
3	4	12
1	6	6
1	8	8
0	10	0
1	22	22

fbot = -Fse(1/A+e/Sb)

Fse = 361.305

1/A = 0.006125574

e/sb = 0.008614653

fbot = -5.325717703 ksi

fbot/fci = -0.869221104	**-86.92 %**

AASHTO Losses

ELASTIC SHORTENING

Modulus of Elasticity

concrete =	4700 ksi (1 day)	
strand =	28500 ksi	
n =	6.06	

Strands

total # =	9	
total area =	1.953 in^2	
f_{pu} =	270 ksi	
tension after stressing =	202.20 ksi	

elongation of the strands (in)

1	8.250
2	8.500
3	8.500
4	8.625
5	8.625
6	8.750
7	8.500
8	8.500
9	
10	8.375

average elongation =	8.514 in.
length of strand =	100 ft

$f_{cgp} = F/A + Fe^2/I - M_G e/I$

F (Ppi) =	394.91 kips	
M_G =	0 at ends	
M_G =	143.89 k - in	at center
w =	166.54 lb/ft	
f_{cgp} =	4.34 ksi	at ends
f_{cgp} =	4.25 ksi	at center

$\Delta f_{pES} = (E_p/E_{ci}) f_{cgp}$ section 5.9.5.2.3a

Δf_{pES} =	26.30 ksi	at ends
Δf_{pES} =	25.75 ksi	at center

CREEP

$\Delta f_{pCR} = 12.0 f_{cgp} - 7.0 \Delta f_{cdp}$

Δf_{cdp} =	0 ksi	
f_{cgp} =	4.34 ksi	at ends
f_{cgp} =	4.25 ksi	at center

Δf_{pCR} =	52.04 ksi	at ends
Δf_{pCR} =	50.96 ksi	at center

SHRINKAGE

$\Delta f_{pSR} = (17.0 - 0.150\ H)$

H = 65 %

Δf_{pSR} =	7.25

RELAXATION

$\Delta f_{pR2} = (20 - 0.4\Delta f_{pES} - 0.2(\Delta f_{pSR} + \Delta f_{pCR}))*0.30$

Δf_{pES} =	26.30 ksi	at ends
Δf_{pES} =	25.75 ksi	at center
Δf_{pSR} =	7.25	
Δf_{pCR} =	52.04 ksi	at ends
Δf_{pCR} =	50.96 ksi	at center

Δf_{pR2} =	-0.71276 ksi	at ends
Δf_{pR2} =	-0.58226 ksi	at center

$\Delta f_{pR1} = (\log(24.0t)/40.0)(f_{pj}/f_{py} - 0.55)f_{pj}$

t =	1 day
f_{pj} =	202.20 ksi
f_{py} =	270 ksi

Δf_{pR1} =	1.39 ksi

TOTAL LOSSES

$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1} + \Delta f_{pR2}$

Δf_{pT} =	86.26	at ends
Δf_{pT} =	84.76	at center

PCI Losses

ELASTIC SHORTENING

Modulus of Elasticity

concrete =	4700 ksi (1 day)
concrete =	5500 ksi (28days)
strand =	28500 ksi
$n_{one\ day}$ =	6.06
$n_{28\ days}$ =	5.18

Strands

total # =	9
total area =	1.953 in^2
tension after stressing =	202.20 ksi

elongation of the strands (in)

1	8.250
2	8.500
3	8.500
4	8.625
5	8.625
6	8.750
7	8.500
8	8.500
9	
10	8.375
average =	8.514 in.
length of strand =	100 ft.

Ppi = 394.91 kips
Kcir = 0.886

$$f_{cir} = K_{cir}(P_{pi}(1/A_g + e^2/I_g)) - f_g$$

e =	7.75 in.	
A_g =	163.25 in^2	
I_g =	12370 in^4	
f_g =	0 ksi	at ends
f_g =	0.090 ksi	at center
f_{cir} =	3.84 ksi	at ends
f_{cir} =	3.75 ksi	at center

$f_g = M_G e/I$

w =	166.54 lb/ft
M_G =	143.89 k - in

$$ES = K_{es} n f_{cir}$$

K_{es} = 1.0

ES =	**23.30 ksi**	at ends
ES =	**22.75 ksi**	at center

CREEP

$CR = K_{cr} n(f_{cir} - f_{cds})$

K_{cr} =	2.0 for p/s members	
f_{cir} =	3.84 ksi	at ends
f_{cir} =	3.75 ksi	at center
f_{cds} =	0 ksi	

CR =	39.82 ksi	at ends
CR =	38.88 ksi	at center

SHRINKAGE

$SH = 8.2{*}10^{-6} K_{sh} E_s (1 - 0.06V/S)(100 - RH)$

RH =	65 %
Volume =	47016 in^3
Surface Area =	28224 in^2
V/A =	1.67
K_{sh} =	1
E_s =	28500 ksi

SH =	7.36 ksi

RELAXATION

$RE = (K_{re} - J(SH + CR + ES))C$

$K_{re} =$ 5

$J =$ 0.040

$SH =$ 7.36 ksi

$CR =$ 39.82 ksi at ends

$CR =$ 38.88 ksi at center

$ES =$ 23.30 ksi at ends

$ES =$ 22.75 ksi at center

$C =$ 1.00

$f_{pi} =$ 202.20 ksi

$f_{pu} =$ 270 ksi

$f_{pi}/f_{pu} =$ 0.75

RE =	2.18 ksi at ends
RE =	2.24 ksi at center

Total Losses

ES =	23.30 ksi	at ends
ES =	22.75 ksi	at center
CR =	39.82 ksi	at ends
CR =	38.88 ksi	at center
SH =	7.36 ksi	
RE =	2.18 ksi	at ends
RE =	2.24 ksi	at center

Total Losses at Ends =	**72.66 ksi**
Total Losses at Center =	**71.24 ksi**

The following pages contain the prestress loss calculations (AASHTO and PCI) for Girder/Beam 3.

BEAM 3

Section Properties

Sb = 899.63 in^3

Area = 163.25 in^2

y = 13.75 in.

cgs = 7 in.

e = 6.75

Strand Properties

fse = 185 ksi

Area = 0.217 in^2

Fse = 321.16 kips

Total number of strands = 8

Concrete Properties

fci = 6.127 ksi

fc = 10 ksi

Total Air Content = 6%

Unit Weight = 146.9 lb/ft^3

Number of Strands (1)	Distance from bottom fiber (2)	1*2
3	2	6
1	4	4
1	6	6
1	8	8
1	10	10
1	22	22

fbot = -Fse(1/A+e/Sb)

Fse = 321.16

1/A = 0.006125574

e/sb = 0.007503085

fbot = -4.376980084 ksi

fbot/fci = -0.714375728	**-71.44 %**

AASHTO Losses

ELASTIC SHORTENING

Modulus of Elasticity

concrete =	4700 ksi (1 day)	
strand =	28500 ksi	
n =	6.06	

Strands

total # =	8	
total area =	1.736 in^2	
f_{pu} =	270 ksi	
tension after stressing =	200.76 ksi	

elongation of the strands (in)

1	8.250
2	8.500
3	8.500
4	
5	8.625
6	
7	8.500
8	8.500
9	8.375
10	8.375

average elongation =	8.453 in.
length of strand =	100 ft

$f_{cgp} = F/A + Fe^2/I - M_G e/I$

F (Ppi) =	348.52 kips	
M_G =	0 at ends	
M_G =	143.89 k - in	at center
w =	166.5377 lb/ft	
f_{cgp} =	3.42 ksi	at ends
f_{cgp} =	3.34 ksi	at center

$\Delta f_{pES} = (E_p/E_{ci}) f_{cgp}$ section 5.9.5.2.3a

Δf_{pES} =	20.73 ksi	at ends
Δf_{pES} =	20.25 ksi	at center

CREEP

$\Delta f_{pCR} = 12.0 f_{cgp} - 7.0 \Delta f_{cdp}$

Δf_{cdp} =	0 ksi	
f_{cgp} =	3.42 ksi	at ends
f_{cgp} =	3.34 ksi	at center

Δf_{pCR} =	41.02 ksi	at ends
Δf_{pCR} =	40.08 ksi	at center

SHRINKAGE

$\Delta f_{pSR} = (17.0 - 0.150\ H)$

H = 65 %

Δf_{pSR} =	7.25

RELAXATION

$\Delta f_{pR2} = (20 - 0.4\Delta f_{pES} - 0.2(\Delta f_{pSR} + \Delta f_{pCR}))*0.30$

Δf_{pES} =	20.73 ksi	at ends
Δf_{pES} =	20.25 ksi	at center
Δf_{pSR} =	7.25	
Δf_{pCR} =	41.02 ksi	at ends
Δf_{pCR} =	40.08 ksi	at center

Δf_{pR2} =	0.62 ksi	at ends
Δf_{pR2} =	0.73 ksi	at center

$\Delta f_{pR1} = (\log(24.0t)/40.0)(f_{pj}/f_{py} - 0.55)f_{pj}$

t =	1 day
f_{pj} =	200.76 ksi
f_{py} =	270 ksi

Δf_{pR1} =	1.34 ksi

TOTAL LOSSES

$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1} + \Delta f_{pR2}$

Δf_{pT} =	70.96 at ends
Δf_{pT} =	69.66 at center

PCI Losses

ELASTIC SHORTENING

Modulus of Elasticity

concrete =	4700 ksi (1 day)	
concrete =	5500 ksi (28 days)	
strand =	28500 ksi	
$n_{one\ day}$ =	6.06	
$n_{28\ days}$ =	5.18	

Strands

total # =	8	
total area =	1.736 in^2	
tension after stressing =	200.76 ksi	

elongation of the strands (in)

1	8.250
2	8.500
3	8.500
4	
5	8.625
6	
7	8.500
8	8.500
9	8.375
10	8.375
average =	8.453 in.
length of strand =	100 ft.

Ppi = 348.52 kips
Kcir = 0.91

$$f_{cir} = K_{cir}(P_{pi}(1/A_g + e^2/I_g)) - f_g$$

e =	6.75 in.	
A_g =	163.25 in^2	
I_g =	12370 in^4	
f_g =	0 ksi	at ends
f_g =	0.079 ksi	at center
f_{cir} =	3.11 ksi	at ends
f_{cir} =	3.03 ksi	at center

$$f_g = M_G e/I$$

w =	166.54 lb/ft
M_G =	143.89 k - in

$$ES = K_{es} n f_{cir}$$

K_{es} = 1.0

ES =	**18.86 ksi**	at ends
ES =	**18.39 ksi**	at center

CREEP

$$CR = K_{cr} n(f_{cir} - f_{cds})$$

K_{cr} =	2.0 for p/s members	
f_{cir} =	3.11 ksi	at ends
f_{cir} =	3.03 ksi	at center
f_{cds} =	0 ksi	

CR =	32.24 ksi	at ends
CR =	31.43 ksi	at center

SHRINKAGE

$$SH = 8.2 \cdot 10^{-6} K_{sh} E_s (1 - 0.06V/S)(100 - RH)$$

RH =	65 %
Volume =	47016 in^3
Surface Area =	28224 in^2
V/A =	1.67
K_{sh} =	1
E_s =	28500 ksi

SH =	7.36 ksi

RELAXATION

$RE = (K_{re} - J(SH + CR + ES))C$

K_{re} =	5
J =	0.040
SH =	7.36 ksi
CR =	32.24 ksi at ends
CR =	31.43 ksi at center
ES =	18.86 ksi at ends
ES =	18.39 ksi at center
C =	0.95
f_{pi} =	200.76 ksi
f_{pu} =	270 ksi
f_{pi}/f_{pu} =	0.74

RE =	2.53 ksi at ends
RE =	2.58 ksi at center

Total Losses

ES =	18.86 ksi	at ends
ES =	18.39 ksi	at center
CR =	32.24 ksi	at ends
CR =	31.43 ksi	at center
SH =	7.36 ksi	
RE =	2.53 ksi	at ends
RE =	2.58 ksi	at center

Total Losses at Ends =	**61.00 ksi**
Total Losses at Center =	**59.75 ksi**

The following pages contain the prestress loss calculations (AASHTO and PCI) for Girder/Beam 4.

BEAM 4

Section Properties

Sb = 899.63 in^3
Area = 163.25 in^2
y = 13.75 in.
cgs = 5.75 in.
e = 8

Strand Properties

fse = 185 ksi
Area = 0.217 in^2
Fse = 321.16 kips
Total number of strands = 8

Concrete Properties

fci = 8.696 ksi
fc = 10 ksi
Total Air Content = 2%
Unit Weight = 151.1 lb/ft^3

Number of Strands (1)	Distance from bottom fiber (2)	1*2
3	2	6
3	4	12
1	6	6
0	8	0
0	10	0
1	22	22

fbot = -Fse(1/A+e/Sb)

Fse = 321.16
1/A = 0.006125574
e/sb = 0.008892545

fbot = -4.823219093 ksi

fbot/fci = -0.55464801	**-55.46 %**

AASHTO Losses

ELASTIC SHORTENING

Modulus of Elasticity

concrete =	5600 ksi (1 day)	
strand =	28500 ksi	
n =	5.09	

Strands

total # =	8	
total area =	1.736 in^2	
f_{pu} =	270 ksi	
tension after stressing =	204.47 ksi	

elongation of the strands (in)

1	8.750
2	8.750
3	8.750
4	8.250
5	8.500
6	8.500
7	8.625
8	
9	
10	8.750

average elongation =	8.609 in.
length of strand =	100 ft

$f_{cgp} = F/A + Fe^2/I - M_Ge/I$

F (Ppi) =	354.96 kips	
M_G =	0 at ends	
M_G =	148.00 k - in	at center
w =	171.30 lb/ft	
f_{cgp} =	4.01 ksi	at ends
f_{cgp} =	3.92 ksi	at center

$\Delta f_{pES} = (E_p/E_{ci})f_{cgp}$ section 5.9.5.2.3a

Δf_{pES} =	20.41 ksi	at ends
Δf_{pES} =	19.93 ksi	at center

CREEP

$\Delta f_{pCR} = 12.0f_{cgp} - 7.0\Delta f_{cdp}$

Δf_{cdp} =	0 ksi	
f_{cgp} =	4.01 ksi	at ends
f_{cgp} =	3.92 ksi	at center

Δf_{pCR} =	48.13 ksi	at ends
Δf_{pCR} =	46.98 ksi	at center

SHRINKAGE

$\Delta f_{pSR} = (17.0 - 0.150\ H)$

H = 65 %

Δf_{pSR} =	7.25

RELAXATION

Δf_{pR2} = (20 - 0.4Δf_{pES} - 0.2(Δf_{pSR} + Δf_{pCR}))*0.30

Δf_{pES} =	20.41 ksi	at ends
Δf_{pES} =	19.93 ksi	at center
Δf_{pSR} =	7.25	
Δf_{pCR} =	48.13 ksi	at ends
Δf_{pCR} =	46.98 ksi	at center

Δf_{pR2} =	0.23 ksi	at ends
Δf_{pR2} =	0.36 ksi	at center

Δf_{pR1} = (log(24.0t)/40.0)(f_{pj}/f_{py} - 0.55)f_{pj}

t =	1 day
f_{pj} =	204.4727 ksi
f_{py} =	270 ksi

Δf_{pR1} =	1.46 ksi

TOTAL LOSSES

Δf_{pT} = Δf_{pES} + Δf_{pSR} + Δf_{pCR} + Δf_{pR1} + Δf_{pR2}

Δf_{pT} =	77.48 at ends
Δf_{pT} =	75.97 at center

PCI Losses

ELASTIC SHORTENING

Modulus of Elasticity

concrete =	5600 ksi (l day)
concrete =	6000 ksi (28 days)
strand =	28500 ksi
$n_{one\ day}$ =	5.09
$n_{28\ days}$ =	4.75

Strands

total # =	8
total area =	1.736 in^2
tension after stressing =	204.47 ksi

elongation of the strands (in)

1	8.750
2	8.750
3	8.750
4	8.250
5	8.500
6	8.500
7	8.625
8	
9	
10	8.750
average =	8.609 in.
length of strand =	100 ft.

Ppi = 354.96 kips

Kcir = 0.91

$$f_{cir} = K_{cir}(P_{pi}(1/A_g + e^2/I_g)) - f_g$$

e =	8 in.	
A_g =	163.25 in^2	
I_g =	12370 in^4	
f_g =	0 ksi	at ends
f_g =	0.096 ksi	at center
f_{cir} =	3.65 ksi	at ends
f_{cir} =	3.55 ksi	at center

$f_g = M_G e/I$

w =	171.30 lb/ft
M_G =	148.00 k - in

$$ES = K_{es} n f_{cir}$$

K_{es} = 1.0

ES =	**18.58 ksi**	at ends
ES =	**18.09 ksi**	at center

CREEP

$CR = K_{cr} n(f_{cir} - f_{cds})$

K_{cr} =	2.0 for p/s members	
f_{cir} =	3.65 ksi	at ends
f_{cir} =	3.55 ksi	at center
f_{cds} =	0 ksi	

CR =	34.67 ksi	at ends
CR =	33.76 ksi	at center

SHRINKAGE

$SH = 8.2 * 10^{-6} K_{sh} E_s (1 - 0.06V/S)(100 - RH)$

RH =	65 %
Volume =	47016 in^3
Surface Area =	28224 in^2
V/A =	1.67
K_{sh} =	1
E_s =	28500 ksi

SH =	7.36 ksi

RELAXATION

$RE = (K_{re} - J(SH + CR + ES))C$

K_{re} = 5

J = 0.040

SH = 7.36 ksi

CR = 34.67 ksi at ends

CR = 33.76 ksi at center

ES = 18.58 ksi at ends

ES = 18.09 ksi at center

C = 1.05

f_{pi} = 204.47 ksi

f_{pu} = 270 ksi

f_{pi}/f_{pu} = 0.76

RE =	2.70 ksi at ends
RE =	2.76 ksi at center

Total Losses

ES =	18.58 ksi	at ends
ES =	18.09 ksi	at center
CR =	34.67 ksi	at ends
CR =	33.76 ksi	at center
SH =	7.36 ksi	
RE =	2.70 ksi	at ends
RE =	2.76 ksi	at center

Total Losses at Ends =	**63.32 ksi**
Total Losses at Center =	**61.98 ksi**

The following pages contain the strain measurements for Girder/Beam 1.

Beam 1

		Before Cutting 12/20/00				After Cutting 12/20/00							
		East		West		East		West					
North											Strain	Microstrain	Losses (ksi)
Top	60 - 68	770	766	814	816	768	768	812	811	1.75	1.42E-05	14	0.40
	68 - 76	655	658	186	190	591	592	109	111	71.50	0.000579	579	16.51
Bottom	60 - 68	783	778	787	787	661	662	664	663	121.25	0.000982	982	27.99
	68 - 76	1160	1157	829	827	1035	1034	699	698	126.75	0.001027	1027	29.26
Top	144 - 152	830	826	778	771	819	819	773	774	5.00	4.05E-05	41	1.15
	152 - 160	772	768	836	833	760	760	824	823	10.50	8.51E-05	85	2.42
Bottom	144 - 152	826	825	803	803	705	705	690	688	117.25	0.00095	950	27.07
	152 - 160	751	747	50	35	629	629			120.00	0.000972	972	27.70
South													0.00
Top	60 - 68	972	970	770	771	964	965	763	762	7.25	5.87E-05	59	1.67
	68 - 76	843	849	788	788	795	794	731	731	54.25	0.000439	439	12.52
Bottom	60 - 68	792	799	1495	1501	681	681	1397	1394	108.50	0.000879	879	25.05
	68 - 76	1018	1015	757	752	899	898	636	638	117.75	0.000954	954	27.18

Distance for DEMEC to extreme fiber = 19

c.g.s without top strand = 4.67

North End		At C.G.S.		Average
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	50.94	998.94	28.47	28.98
Distance = 68 - 76	23.55	1034.45	29.48	

Center		At C.G.S.		
	Slope	Strain	Stress (ksi)	
Distance = 144 - 152	47.85	965.52	27.52	27.83
Distance = 152 - 160	47.85	987.79	28.15	

South End		At C.G.S.		
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	43.16	893.09	25.45	26.45
Distance = 68 - 76	27.07	962.71	27.44	

5-Jan-01

		East		West			Strain	Microstrain	Losses (ksi)
North									
Top	60 - 68	760	760	796	795	13.75	0.000111	111	3.17
	68 - 76	577	577	135	137	65.75	0.000533	533	15.18
Bottom	60 - 68	609	609	614	614	172.25	0.001395	1395	39.76
	68 - 76	985	984	651	652	175.25	0.00142	1420	40.46
Top	144 - 152	805	806	756	756	20.50	0.000166	166	4.73
	152 - 160	747	747	805	806	26.00	0.000211	211	6.00
Bottom	144 - 152	651	653	640	640	168.25	0.001363	1363	38.84
	152 - 160	577	576			172.50	0.001397	1397	39.82
South									
Top	60 - 68	954	953	746	746	21.00	0.00017	170	4.85
	68 - 76	778	779	713	712	71.50	0.000579	579	16.51
Bottom	60 - 68	630	629	1342	1342	161.00	0.001304	1304	37.17
	68 - 76	844	845	588	588	169.25	0.001371	1371	39.07

Distance for DEMEC to extreme fiber = 19
c.g.s without top strand = 4.67

North End	At C.G.S.			Average
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	67.57	1417.52	40.40	40.65
Distance = 68 - 76	46.68	1434.93	40.90	

Center	At C.G.S.			
	Slope	Strain	Stress (ksi)	
Distance = 144 - 152	62.99	1383.61	39.43	39.92
Distance = 152 - 160	62.99	1418.04	40.41	

South End	At C.G.S.			
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	59.68	1323.80	37.73	38.60
Distance = 68 - 76	41.67	1384.68	39.46	

18-Jan-01

		East		West			Strain	Microstrain	Losses (ksi)
North									
Top	60 - 68	764	763	806	804	7.25	5.87E-05	59	1.67
	68 - 76	583	584	110	115	74.25	0.000601	601	17.14
Bottom	60 - 68	605	605	611	612	175.50	0.001422	1422	40.51
	68 - 76	981	981	650	650	177.75	0.00144	1440	41.03
Top	144 - 152	816	817	765	766	10.25	8.3E-05	83	2.37
	152 - 160	754	754	814	813	18.50	0.00015	150	4.27
Bottom	144 - 152	650	652	641	642	168.00	0.001361	1361	38.78
	152 - 160	575	574			174.50	0.001413	1413	40.28
South									
Top	60 - 68	968	968	760	759	7.00	5.67E-05	57	1.62
	68 - 76	791	791	725	726	58.75	0.000476	476	13.56
Bottom	60 - 68	627	627	1334	1335	166.00	0.001345	1345	38.32
	68 - 76	846	846	589	589	168.00	0.001361	1361	38.78

Distance for DEMEC to extreme fiber = 19
c.g.s without top strand = 4.67

North End		At C.G.S.		Average
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	71.73	1445.22	41.19	41.32
Distance = 68 - 76	44.12	1454.34	41.45	

Center		At C.G.S.		
	Slope	Strain	Stress (ksi)	
Distance = 144 - 152	67.25	1382.99	39.42	40.17
Distance = 152 - 160	67.25	1435.64	40.92	

South End		At C.G.S.		
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	67.78	1366.97	38.96	39.09
Distance = 68 - 76	46.58	1376.17	39.22	

2-Feb-01

		East		West			Strain	Microstrain	Losses (ksi)
North									
Top	60 - 68	752	752	793	792	19.25	0.000156	156	4.44
	68 - 76	571	570	63	70	103.75	0.00084	840	23.95
Bottom	60 - 68	578	579	584	585	202.25	0.001638	1638	46.69
	68 - 76	955	955	625	626	203.00	0.001644	1644	46.86
Top	144 - 152	804	805	752	752	23.00	0.000186	186	5.31
	152 - 160	740	740	798	799	33.00	0.000267	267	7.62
Bottom	144 - 152	624	624	615	617	194.25	0.001573	1573	44.84
	152 - 160	550	550			199.00	0.001612	1612	45.94
South									
Top	60 - 68	950	948	742	743	25.00	0.000203	203	5.77
	68 - 76	778	779	715	715	70.25	0.000569	569	16.22
Bottom	60 - 68	604	603	1311	1311	189.50	0.001535	1535	43.75
	68 - 76	821	822	565	566	192.00	0.001555	1555	44.32

Distance for DEMEC to extreme fiber = 19
c.g.s without top strand = 4.67

North End		**At C.G.S.**		**Average**
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	78.02	1663.97	47.42	47.34
Distance = 68 - 76	42.31	1658.26	47.26	

Center		**At C.G.S.**		
	Slope	Strain	Stress (ksi)	
Distance = 144 - 152	73.01	1597.52	45.53	46.08
Distance = 152 - 160	73.01	1635.99	46.63	

South End		**At C.G.S.**		
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	70.13	1558.09	44.41	44.61
Distance = 68 - 76	51.90	1572.33	44.81	

		19-Feb-01							
		East		West					
North							Strain	Microstrain	Losses (ksi)
Top	60 - 68	768	768	805	805	5.00	4.05E-05	41	1.15
	68 - 76	582	582	136	140	62.25	0.000504	504	14.37
Bottom	60 - 68	588	588	591	592	194.00	0.001571	1571	44.78
	68 - 76	961	961	629	629	198.25	0.001606	1606	45.77
Top	144 - 152	814	815	762	762	13.00	0.000105	105	3.00
	152 - 160	752	752	811	810	21.00	0.00017	170	4.85
Bottom	144 - 152	632	638	625	628	183.50	0.001486	1486	42.36
	152 - 160	557	555			193.00	0.001563	1563	44.55
South									
Top	60 - 68	931	935	755	758	26.00	0.000211	211	6.00
	68 - 76	783	783	720	721	65.25	0.000529	529	15.06
Bottom	60 - 68	609	610	1319	1320	182.25	0.001476	1476	42.07
	68 - 76	825	825	570	568	188.50	0.001527	1527	43.52

Distance for DEMEC to extreme fiber = 19

c.g.s without top strand = 4.67

North End		**At C.G.S.**		**Average**
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	80.57	1597.99	45.54	45.93
Distance = 68 - 76	57.98	1624.96	46.31	

Center		**At C.G.S.**		
	Slope	Strain	Stress (ksi)	
Distance = 144 - 152	72.69	1510.34	43.04	44.14
Distance = 152 - 160	72.69	1587.29	45.24	

South End		**At C.G.S.**		
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	66.61	1498.21	42.70	43.35
Distance = 68 - 76	52.54	1544.19	44.01	

		14-Mar-01							
		East		West					
North							Strain	Microstrain	Losses (ksi)
Top	60 - 68	765	765	800	794	10.50	8.51E-05	85	2.42
	68 - 76	575	576			81.00	0.000656	656	18.70
Bottom	60 - 68	576	576	583	583	204.25	0.001654	1654	47.15
	68 - 76	954	954	620	621	206.00	0.001669	1669	47.56
Top	144 - 152	810	810	760	760	16.25	0.000132	132	3.75
	152 - 160	750	750	809	807	23.25	0.000188	188	5.37
Bottom	144 - 152	626	626	615	618	193.00	0.001563	1563	44.55
	152 - 160	550	550			199.00	0.001612	1612	45.94
South									
Top	60 - 68			759	760	11.00	8.91E-05	89	2.54
	68 - 76	784	783	720	720	65.25	0.000529	529	15.06
Bottom	60 - 68	605	605	1312	1312	188.25	0.001525	1525	43.46
	68 - 76	820	820	566	565	192.75	0.001561	1561	44.50

Distance for DEMEC to extreme fiber = 19
c.g.s without top strand = 4.67

North End		At C.G.S.		Average
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	82.60	1681.68	47.93	47.99
Distance = 68 - 76	53.29	1686.19	48.06	

Center		At C.G.S.		
	Slope	Strain	Stress (ksi)	
Distance = 144 - 152	75.35	1588.17	45.26	45.96
Distance = 152 - 160	75.35	1636.77	46.65	

South End		At C.G.S.		
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	75.56	1549.76	44.17	44.59
Distance = 68 - 76	54.36	1579.21	45.01	

19-Apr-01

			East		West					
North								Strain	Microstrain	Losses (ksi)
Top	60 - 68	767	768	803	800	7.00	0.0000567	57	1.62
	68 - 76	580	581	125	125	69.50	0.00056295	563	16.04
Bottom	60 - 68	571	571	576	576	210.25	0.00170303	1703	48.54
	68 - 76	948	945	612	611	214.25	0.00173543	1735	49.46
Top	144 - 152	815	813	760	759	14.50	0.00011745	117	3.35
	152 - 160	751	750	810	809	22.25	0.00018023	180	5.14
Bottom	144 - 152	623	621	611	611	197.75	0.00160178	1602	45.65
	152 - 160	543				208.00	0.0016848	1685	48.02
South									
Top	60 - 68			761	762	9.00	0.0000729	73	2.08
	68 - 76	791	791	726	725	58.75	0.00047588	476	13.56
Bottom	60 - 68	596	597	1302	1302	197.50	0.00159975	1600	45.59
	68 - 76	815	815	562	561	197.25	0.00159773	1598	45.54

Distance for DEMEC to extreme fiber = 19
c.g.s without top strand = 4.67

North End		**At C.G.S.**		**Average**
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	86.65	1731.62	49.35	49.70
Distance = 68 - 76	61.71	1755.79	50.04	

Center		**At C.G.S.**		
	Slope	Strain	Stress (ksi)	
Distance = 144 - 152	78.12	1627.56	46.39	47.57
Distance = 152 - 160	78.12	1710.58	48.75	

South End		**At C.G.S.**		
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	80.36	1626.27	46.35	46.22
Distance = 68 - 76	59.04	1617.21	46.09	

		25-Jun-01				180 days			
		East		West					
North							Strain	Microstrain	Losses (ksi)
Top	60 - 68	760	760	790	792	16.00	0.00013	130	3.69
	68 - 76	572	570	135	135	69.25	0.000561	561	15.99
Bottom	60 - 68	550	552	558	559	229.00	0.001855	1855	52.86
	68 - 76	930	928	595	594	231.50	0.001875	1875	53.44
Top	144 - 152	808	810	753	752	20.50	0.000166	166	4.73
	152 - 160	745	742	798	798	31.50	0.000255	255	7.27
Bottom	144 - 152			591	591	212.00	0.001717	1717	48.94
	152 - 160					#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
South									
Top	60 - 68			760	755	13.00	0.000105	105	3.00
	68 - 76	785	785	716	715	66.75	0.000541	541	15.41
Bottom	60 - 68	580	578	1286	1285	214.50	0.001737	1737	49.52
	68 - 76	795	795	537	538	219.25	0.001776	1776	50.61

Distance for DEMEC to extreme fiber = 19

c.g.s without top strand = 4.67

North End		At C.G.S.		Average
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	90.81	1884.87	53.72	53.91
Distance = 68 - 76	69.17	1897.98	54.09	

Center		At C.G.S.		
	Slope	Strain	Stress (ksi)	
Distance = 144 - 152	81.64	1744.14	49.71	49.71
Distance = 152 - 160	81.64	#DIV/0!	#DIV/0!	

South End		At C.G.S.		
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	85.90	1765.80	50.33	50.78
Distance = 68 - 76	65.01	1797.38	51.23	

				21-Dec-01		360 days				
		East		West						
North							Strain	Microstrain	Losses (ksi)
Top	60 - 68	756	756	783	783	22.00	0.000178	178	5.08
	68 - 76	579	579			-156.75	-0.00127	-1270	-36.19
Bottom	60 - 68			542	545	243.50	0.001972	1972	56.21
	68 - 76			578	578	250.00	0.002025	2025	57.71
Top	144 - 152	798	795	742	744	31.50	0.000255	255	7.27
	152 - 160	738	739	799	799	33.50	0.000271	271	7.73
Bottom	144 - 152			576	575	227.50	0.001843	1843	52.52
	152 - 160					#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
South									
Top	60 - 68			748	749	22.00	0.000178	178	5.08
	68 - 76	790	789	716	715	64.50	0.000522	522	14.89
Bottom	60 - 68	563	563	1263	1263	233.75	0.001893	1893	53.96
	68 - 76	777	778	523	522	235.50	0.001908	1908	54.37

Distance for DEMEC to extreme fiber = 19
c.g.s without top strand = 4.67

North End		At C.G.S.		Average
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	94.43	2003.51	57.10	58.22
Distance = 68 - 76	173.40	2082.22	59.34	

Center		At C.G.S.		
	Slope	Strain	Stress (ksi)	
Distance = 144 - 152	83.56	1870.32	53.30	53.30
Distance = 152 - 160	83.56	#DIV/0!	#DIV/0!	

South End		At C.G.S.		
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	90.27	1923.16	54.81	54.93
Distance = 68 - 76	72.90	1931.61	55.05	

The following pages contain the strain measurements for Girder 2/Beam 2.

		Before Cutting 12/22/00				After Cutting 12/22/00							
		East		West		East		West					
North										Difference	Strain	Microstrain	Losses (ksi)
Top	60 - 68	689	689	822	824	684	684	821	821	3.50	2.835E-05	28	0.81
	68 - 76	669	668	762	761	663	662	759	759	4.25	3.443E-05	34	0.98
Bottom	60 - 68	733	733	800	801	594	594	665	664	137.50	0.0011138	1114	31.74
	68 - 76	742	742	803	803	600	601	661	661	141.75	0.0011482	1148	32.72
Top	144 - 152	957	959	835	835	952	953	826	827	7.00	0.0000567	57	1.62
	152 - 160	598	598	768	767	592	591	758	758	8.00	0.0000648	65	1.85
Bottom	144 - 152	239	237	809	810	98	95	672	673	139.25	0.0011279	1128	32.15
	152 - 160	1298	1299	774	775	1163	1164	632	636	137.75	0.0011158	1116	31.80
South													0.00
Top	60 - 68	824	824	795	795	821	820	792	791	3.50	2.835E-05	28	0.81
	68 - 76	791	791	813	813	787	786	810	810	3.75	3.038E-05	30	0.87
Bottom	60 - 68	1082	1084	773	772	951	952	644	645	129.75	0.001051	1051	29.95
	68 - 76	1072	1072	679	680	940	939	550	549	131.25	0.0010631	1063	30.30

Distance for DEMEC to extreme fiber = 19

c.g.s without top strand = 4

North End		At C.G.S.		Average
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	57.13	1170.88	33.37	33.88
Distance = 68 - 76	58.62	1206.79	34.39	

Center		At C.G.S.		
	Slope	Strain	Stress (ksi)	
Distance = 144 - 152	56.38	1184.31	33.75	33.58
Distance = 152 - 160	56.38	1172.16	33.41	

South End		At C.G.S.		
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	53.82	1104.80	31.49	31.67
Distance = 68 - 76	54.36	1117.48	31.85	

5-Jan-01

		East		West					
North						Difference	Strain	Microstrain	Losses (ksi)
Top	60 - 68	678	678	813	813	10.50	0.00008505	85	2.42
	68 - 76	654	654	742	743	16.75	0.000135675	136	3.87
Bottom	60 - 68	522	522	588	589	211.50	0.00171315	1713	48.82
	68 - 76	526	525	583	583	218.25	0.001767825	1768	50.38
Top	144 - 152	948	948	819	820	12.75	0.000103275	103	2.94
	152 - 160	585	585	755	757	12.25	0.000099225	99	2.83
Bottom	144 - 152	21	20	595	596	215.75	0.001747575	1748	49.81
	152 - 160	1091	1091	556	557	212.75	0.001723275	1723	49.11
South									
Top	60 - 68	817	816	781	783	10.25	0.000083025	83	2.37
	68 - 76	780	780	804	802	10.50	0.00008505	85	2.42
Bottom	60 - 68	875	875	566	567	207.00	0.0016767	1677	47.79
	68 - 76	866	867	468	468	208.50	0.00168885	1689	48.13

Distance for DEMEC to extreme fiber = 19

c.g.s without top strand = 4

North End		At C.G.S.		Average
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	85.69	1798.84	51.27	52.05
Distance = 68 - 76	85.90	1853.73	52.83	

Center		At C.G.S.		
	Slope	Strain	Stress (ksi)	
Distance = 144 - 152	86.54	1834.12	52.27	51.93
Distance = 152 - 160	86.54	1809.82	51.58	

South End		At C.G.S.		
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	83.88	1760.58	50.18	50.36
Distance = 68 - 76	84.41	1773.26	50.54	

18-Jan-01

		East		West		Difference	Strain	Microstrain	Losses (ksi)
North									
Top	60 - 68	673	675	812	813	12.75	0.000103275	103	2.94
	68 - 76	651	651	740	740	19.50	0.00015795	158	4.50
Bottom	60 - 68	510	510	575	574	224.50	0.00181845	1818	51.83
	68 - 76	513	513	572	573	229.75	0.001860975	1861	53.04
Top	144 - 152	939	941	820	821	16.25	0.000131625	132	3.75
	152 - 160	582	582	763	764	10.00	0.000081	81	2.31
Bottom	144 - 152	3	3	585	584	230.00	0.001863	1863	53.10
	152 - 160	1075	1077	544	543	226.75	0.001836675	1837	52.35
South									
Top	60 - 68	813	815	778	777	13.75	0.000111375	111	3.17
	68 - 76	779	780	809	809	7.75	0.000062775	63	1.79
Bottom	60 - 68	863	863	562	561	215.50	0.00174555	1746	49.75
	68 - 76	855	854	458	460	219.00	0.0017739	1774	50.56

Distance for DEMEC to extreme fiber = 19

c.g.s without top strand = 4

North End		**At C.G.S.**		**Average**
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	90.27	1908.72	54.40	55.00
Distance = 68 - 76	89.63	1950.61	55.59	

Center		**At C.G.S.**		
	Slope	Strain	Stress (ksi)	
Distance = 144 - 152	91.13	1954.13	55.69	55.32
Distance = 152 - 160	91.13	1927.80	54.94	

South End		**At C.G.S.**		
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	86.01	1831.56	52.20	52.66
Distance = 68 - 76	90.06	1863.96	53.12	

5-Feb-01

		East		West					
North						Difference	Strain	Microstrain	Losses (ksi)
Top	60 - 68	652	652	793	793	33.50	0.00027135	271	7.73
	68 - 76	633	633	720	721	38.25	0.000309825	310	8.83
Bottom	60 - 68	475	475	546	546	256.25	0.002075625	2076	59.16
	68 - 76	475	476	539	540	265.00	0.0021465	2147	61.18
Top	144 - 152	920	921	800	799	36.50	0.00029565	296	8.43
	152 - 160	562	562	738	737	33.00	0.0002673	267	7.62
Bottom	144 - 152			552	552	257.50	0.00208575	2086	59.44
	152 - 160	1042	1043	510	510	260.25	0.002108025	2108	60.08
South									
Top	60 - 68	793	792	755	755	35.75	0.000289575	290	8.25
	68 - 76	763	763	792	792	24.50	0.00019845	198	5.66
Bottom	60 - 68	828	830	528	527	249.50	0.00202095	2021	57.60
	68 - 76	822	822	427	426	251.50	0.00203715	2037	58.06

Distance for DEMEC to extreme fiber = 19

c.g.s without top strand = 4

North End		**At C.G.S.**		**Average**
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	94.96	2170.59	61.86	62.90
Distance = 68 - 76	96.67	2243.17	63.93	

Center		**At C.G.S.**		
	Slope	Strain	Stress (ksi)	
Distance = 144 - 152	94.22	2179.97	62.13	62.45
Distance = 152 - 160	94.22	2202.24	62.76	

South End		**At C.G.S.**		
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	91.13	2112.08	60.19	60.51
Distance = 68 - 76	96.77	2133.92	60.82	

19-Feb-01

		East		West					
North						Difference	Strain	Microstrain	Losses (ksi)
Top	60 - 68	676	677	812	811	12.00	0.0000972	97	2.77
	68 - 76	648	648	731	731	25.50	0.00020655	207	5.89
Bottom	60 - 68	489	488	549	549	248.00	0.0020088	2009	57.25
	68 - 76	521		550	551	242.00	0.0019602	1960	55.87
Top	144 - 152	943	944	822	821	14.00	0.0001134	113	3.23
	152 - 160	584	583	757	760	11.75	0.000095175	95	2.71
Bottom	144 - 152			562	564	246.50	0.00199665	1997	56.90
	152 - 160	1052	1053	519	518	251.00	0.0020331	2033	57.94
South									
Top	60 - 68	816	815	778	782	11.75	0.000095175	95	2.71
	68 - 76	782	782	802	803	9.75	0.000078975	79	2.25
Bottom	60 - 68	840	841	528	530	243.00	0.0019683	1968	56.10
	68 - 76	834	831	434	431	243.25	0.001970325	1970	56.15

Distance for DEMEC to extreme fiber = 19
c.g.s without top strand = 4

North End		**At C.G.S.**		**Average**
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	100.61	2109.41	60.12	59.31
Distance = 68 - 76	92.30	2052.50	58.50	

Center		**At C.G.S.**		
	Slope	Strain	Stress (ksi)	
Distance = 144 - 152	99.12	2095.77	59.73	60.25
Distance = 152 - 160	99.12	2132.22	60.77	

South End		**At C.G.S.**		
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	98.59	2066.89	58.91	58.95
Distance = 68 - 76	99.54	2069.87	58.99	

14-Mar-01

		East		West		Difference	Strain	Microstrain	Losses (ksi)
North									
Top	60 - 68	662	663	804	804	22.75	0.000184275	184	5.25
	68 - 76	632	632	707	706	45.75	0.000370575	371	10.56
Bottom	60 - 68	468	470	534	535	265.00	0.0021465	2147	61.18
	68 - 76			536	536	267.00	0.0021627	2163	61.64
Top	144 - 152	929	928	810	810	27.25	0.000220725	221	6.29
	152 - 160	568	567	748	749	24.75	0.000200475	200	5.71
Bottom	144 - 152			548	545	263.00	0.0021303	2130	60.71
	152 - 160	1030	1029	506	505	269.00	0.0021789	2179	62.10
South									
Top	60 - 68	802	800	766	766	26.00	0.0002106	211	6.00
	68 - 76	772	772	799	795	17.50	0.00014175	142	4.04
Bottom	60 - 68	825	824	523	522	254.25	0.002059425	2059	58.69
	68 - 76	819	819	420	421	256.00	0.0020736	2074	59.10

Distance for DEMEC to extreme fiber = 19
c.g.s without top strand = 4

North End		At C.G.S.		Average
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	103.28	2249.78	64.12	64.22
Distance = 68 - 76	94.32	2257.02	64.33	

Center		At C.G.S.		
	Slope	Strain	Stress (ksi)	
Distance = 144 - 152	100.50	2230.80	63.58	64.27
Distance = 152 - 160	100.50	2279.40	64.96	

South End		At C.G.S.		
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	97.31	2156.73	61.47	61.73
Distance = 68 - 76	101.68	2175.28	62.00	

		19-Apr-01							
		East		West					
North						Difference	Strain	Microstrain	Losses (ksi)
Top	60 - 68	666	665	813	813	16.75	0.000136	136	3.87
	68 - 76	640	639	720	715	36.50	0.000296	296	8.43
Bottom	60 - 68	461	462	529	528	271.75	0.002201	2201	62.73
	68 - 76			532	532	271.00	0.002195	2195	62.56
Top	144 - 152	937	937	816	815	20.25	0.000164	164	4.67
	152 - 160	577	575	756	756	16.75	0.000136	136	3.87
Bottom	144 - 152			540	539	270.00	0.002187	2187	62.33
	152 - 160	1021	1022	501	502	275.00	0.002228	2228	63.48
South									0.00
Top	60 - 68	809	808	768	767	21.50	0.000174	174	4.96
	68 - 76	777	777	804	805	11.25	9.11E-05	91	2.60
Bottom	60 - 68	817	817	519	519	259.75	0.002104	2104	59.96
	68 - 76	813	813	415	415	261.75	0.00212	2120	60.42

Distance for DEMEC to extreme fiber = 19

c.g.s without top strand = 4

North End		**At C.G.S.**		**Average**
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	108.71	2309.89	65.83	65.62
Distance = 68 - 76	99.97	2295.07	65.41	

Center		**At C.G.S.**		
	Slope	Strain	Stress (ksi)	
Distance = 144 - 152	106.47	2293.47	65.36	65.94
Distance = 152 - 160	106.47	2333.97	66.52	

South End		**At C.G.S.**		
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	101.57	2205.54	62.86	63.16
Distance = 68 - 76	106.79	2226.97	63.47	

			25-Jun-01			180				
		East		West						
North						Difference	Strain	Microstrain	Losses (ksi)
Top	60 - 68			790	796	30.00	0.000243	243	6.93
	68 - 76	630	630	709	710	45.25	0.000367	367	10.45
Bottom	60 - 68	445	445	510	508	289.75	0.002347	2347	66.89
	68 - 76			510	510	293.00	0.002373	2373	67.64
Top	144 - 152			810	811	24.50	0.000198	198	5.66
	152 - 160	565	565	745	744	28.00	0.000227	227	6.46
Bottom	144 - 152			518	516	292.50	0.002369	2369	67.52
	152 - 160	1000	1000	475	475	299.00	0.002422	2422	69.02
South									0.00
Top	60 - 68	802	802	761	763	27.50	0.000223	223	6.35
	68 - 76	771	775	803	803	14.00	0.000113	113	3.23
Bottom	60 - 68	800	800	500	500	277.75	0.00225	2250	64.12
	68 - 76	790	790	400	395	282.00	0.002284	2284	65.10

Distance for DEMEC to extreme fiber = 19
c.g.s without top strand = 4

North End		At C.G.S.		Average
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	110.74	2457.71	70.04	70.35
Distance = 68 - 76	105.62	2478.92	70.65	

Center		At C.G.S.		
	Slope	Strain	Stress (ksi)	
Distance = 144 - 152	114.25	2483.50	70.78	71.53
Distance = 152 - 160	114.25	2536.15	72.28	

South End		At C.G.S.		
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	106.69	2356.46	67.16	67.76
Distance = 68 - 76	114.25	2398.45	68.36	

		21-Dec-01				360			
		East		West					
North						Difference	Strain	Microstrain	Losses (ksi)
Top	60 - 68			798	799	24.50	0.000198	198	5.66
	68 - 76	620	620	712	714	48.50	0.000393	393	11.20
Bottom	60 - 68	435	434	492	491	303.75	0.00246	2460	70.12
	68 - 76			502	503	300.50	0.002434	2434	69.37
Top	144 - 152			803	802	32.50	0.000263	263	7.50
	152 - 160	551	551	737	737	38.75	0.000314	314	8.95
Bottom	144 - 152			508	507	302.00	0.002446	2446	69.72
	152 - 160	987	987	464	464	311.00	0.002519	2519	71.79
South									0.00
Top	60 - 68	790	790			34.00	0.000275	275	7.85
	68 - 76	766	765			25.50	0.000207	207	5.89
Bottom	60 - 68	785	785	483	483	293.75	0.002379	2379	67.81
	68 - 76	781	782	384	384	293.00	0.002373	2373	67.64

Distance for DEMEC to extreme fiber = 19
c.g.s without top strand = 4

North End		**At C.G.S.**		**Average**
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	119.05	2579.42	73.51	72.97
Distance = 68 - 76	107.43	2541.48	72.43	

Center		**At C.G.S.**		
	Slope	Strain	Stress (ksi)	
Distance = 144 - 152	114.89	2561.09	72.99	74.03
Distance = 152 - 160	114.89	2633.99	75.07	

South End		**At C.G.S.**		
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	110.74	2490.11	70.97	70.93
Distance = 68 - 76	114.04	2487.34	70.89	

The following pages contain the strain measurements for Girder/Beam 3.

			Before Cutting 12/22/00				After Cutting 12/22/00							
			East		West		East		West					
North											Difference	Strain	Microstrain	Losses (ksi)
	Top	60 - 68	724	723	815	815	714	713	806	804	10.00	0.000081	81	2.31
		68 - 76	746	746	881	881	737	736	872	872	9.25	7.49E-05	75	2.14
	Bottom	60 - 68	779	777	871	871	671	671	766	765	106.25	0.000861	861	24.53
		68 - 76	770	771	791	792	676	676	692	689	97.75	0.000792	792	22.57
	Top	144 - 152	794	793	644	644	781	779	630	631	13.50	0.000109	109	3.12
		152 - 160	804	803	975	972	793	790	961	962	12.00	9.72E-05	97	2.77
	Bottom	144 - 152	777	776	915	915	676	675	812	812	102.00	0.000826	826	23.55
		152 - 160	843	842	787	785	745	745	684	684	99.75	0.000808	808	23.03
South														
	Top	60 - 68	768	768	747	745	756	757	736	735	11.00	8.91E-05	89	2.54
		68 - 76	732	731	767	767	723	723	756	757	9.50	7.7E-05	77	2.19
	Bottom	60 - 68	614	614	774	774	513	512	623	674	113.50	0.000919	919	26.20
		68 - 76	782	781	953	954	682	683	847	846	103.00	0.000834	834	23.78

Distance for DEMEC to extreme fiber = 19
c.g.s without top strand = 4.86

North End		**At C.G.S.**		**Average**
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	41.03	866.37	24.69	23.70
Distance = 68 - 76	37.73	797.06	22.72	

Center		**At C.G.S.**		
	Slope	Strain	Stress (ksi)	
Distance = 144 - 152	37.73	831.48	23.70	23.44
Distance = 152 - 160	37.41	813.21	23.18	

South End		**At C.G.S.**		
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	43.70	925.47	26.38	25.16
Distance = 68 - 76	39.86	839.88	23.94	

18-Jan-01

		East		West		Difference	Strain	Microstrain	Losses (ksi)
North									
Top	60 - 68	708	707	798	798	16.50	0.00013365	134	3.81
	68 - 76	725	723	861	861	21.00	0.0001701	170	4.85
Bottom	60 - 68	597	595	683	682	185.25	0.001500525	1501	42.76
	68 - 76	595	593	609	609	179.50	0.00145395	1454	41.44
Top	144 - 152	766	766	618	618	26.75	0.000216675	217	6.18
	152 - 160	777	776	948	949	26.00	0.0002106	211	6.00
Bottom	144 - 152	601	604	737	737	176.00	0.0014256	1426	40.63
	152 - 160	663	664	605	605	180.00	0.001458	1458	41.55
South									
Top	60 - 68	746	747	719	720	24.00	0.0001944	194	5.54
	68 - 76	717	718	751	751	15.00	0.0001215	122	3.46
Bottom	60 - 68	429	429	595	596	181.75	0.001472175	1472	41.96
	68 - 76	604	605	768	768	181.25	0.001468125	1468	41.84

Distance for DEMEC to extreme fiber = 19
c.g.s without top strand = 4.86

North End	Slope	At C.G.S. Strain	Stress (ksi)	Average
Distance = 60 - 68	71.94	1510.60	43.05	42.38
Distance = 68 - 76	67.57	1463.41	41.71	

Center	Slope	At C.G.S. Strain	Stress (ksi)	
Distance = 144 - 152	63.63	1434.51	40.88	41.35
Distance = 152 - 160	65.65	1467.19	41.81	

South End	Slope	At C.G.S. Strain	Stress (ksi)	
Distance = 60 - 68	67.25	1481.59	42.23	42.17
Distance = 68 - 76	70.88	1478.05	42.12	

5-Feb-01

			East		West					
North							Difference	Strain	Microstrain	Losses (ksi)
Top	60 - 68	687	686	776	775		38.25	0.000309825	310	8.83
	68 - 76	704	704	842	841		40.75	0.000330075	330	9.41
Bottom	60 - 68	567	568	651	653		214.75	0.001739475	1739	49.58
	68 - 76	568	564	579	578		208.75	0.001690875	1691	48.19
Top	144 - 152	745	744	595	594		49.25	0.000398925	399	11.37
	152 - 160	755	755	923	923		49.50	0.00040095	401	11.43
Bottom	144 - 152	571	571	704	704		208.25	0.001686825	1687	48.07
	152 - 160	635	633	574	574		210.25	0.001703025	1703	48.54
South										
Top	60 - 68	727	729	697	696		44.75	0.000362475	362	10.33
	68 - 76	694	694	731	730		37.00	0.0002997	300	8.54
Bottom	60 - 68	401	401	565	565		211.00	0.0017091	1709	48.71
	68 - 76	577	577	735	735		211.50	0.00171315	1713	48.82

Distance for DEMEC to extreme fiber = 19

c.g.s without top strand = 4.86

North End		At C.G.S.		Average
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	75.24	1750.01	49.88	49.18
Distance = 68 - 76	71.62	1700.90	48.48	

Center		At C.G.S.		
	Slope	Strain	Stress (ksi)	
Distance = 144 - 152	67.78	1696.31	48.34	48.58
Distance = 152 - 160	68.53	1712.62	48.81	

South End		At C.G.S.		
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	70.88	1719.02	48.99	49.06
Distance = 68 - 76	74.39	1723.56	49.12	

19-Feb-01

		East		West		Difference	Strain	Microstrain	Losses (ksi)
North									
Top	60 - 68	705	706	795	795	19.00	0.0001539	154	4.39
	68 - 76	726	724	860	861	20.75	0.000168075	168	4.79
Bottom	60 - 68	579	583	662	662	203.00	0.0016443	1644	46.86
	68 - 76	594	596	585	589	190.00	0.001539	1539	43.86
Top	144 - 152	762	761	614	614	31.00	0.0002511	251	7.16
	152 - 160	775	775	943	943	29.50	0.00023895	239	6.81
Bottom	144 - 152	588	589	719	719	192.00	0.0015552	1555	44.32
	152 - 160	640	639	581	581	204.00	0.0016524	1652	47.09
South									
Top	60 - 68	745	745	711	710	29.25	0.000236925	237	6.75
	68 - 76	715	715	744	744	19.75	0.000159975	160	4.56
Bottom	60 - 68	410	410	573	572	202.75	0.001642275	1642	46.80
	68 - 76	587	586	742	744	202.75	0.001642275	1642	46.80

Distance for DEMEC to extreme fiber = 19

c.g.s without top strand = 4.86

North End		**At C.G.S.**		**Average**
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	78.44	1655.28	47.18	45.66
Distance = 68 - 76	72.15	1549.10	44.15	

Center		**At C.G.S.**		
	Slope	Strain	Stress (ksi)	
Distance = 144 - 152	68.64	1564.81	44.60	45.99
Distance = 152 - 160	74.39	1662.81	47.39	

South End		**At C.G.S.**		
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	73.97	1652.63	47.10	47.11
Distance = 68 - 76	78.02	1653.20	47.12	

14-Mar-01

		East		West					
North						Difference	Strain	Microstrain	Losses (ksi)
Top	60 - 68	695	694	784	781	30.75	0.000249	249	7.10
	68 - 76	710	710	854	854	31.50	0.000255	255	7.27
Bottom	60 - 68	565	566	650	649	217.00	0.001758	1758	50.09
	68 - 76	575	574	573	572	207.50	0.001681	1681	47.90
Top	144 - 152	746	747	604	604	43.50	0.000352	352	10.04
	152 - 160	830	834	935	935	5.00	4.05E-05	41	1.15
Bottom	144 - 152	572	571	708	708	206.00	0.001669	1669	47.56
	152 - 160	635	635	570	569	212.00	0.001717	1717	48.94
South									
Top	60 - 68	737	739	703	705	36.00	0.000292	292	8.31
	68 - 76	709	707	739	737	26.25	0.000213	213	6.06
Bottom	60 - 68	395	394	562	561	216.00	0.00175	1750	49.86
	68 - 76	576	574	736	735	212.25	0.001719	1719	49.00

Distance for DEMEC to extreme fiber = 19
c.g.s without top strand = 4.86

North End		At C.G.S.		Average
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	79.40	1768.82	50.41	49.31
Distance = 68 - 76	75.03	1691.25	48.20	

Center		At C.G.S.		
	Slope	Strain	Stress (ksi)	
Distance = 144 - 152	69.28	1678.30	47.83	48.56
Distance = 152 - 160	88.25	1729.55	49.29	

South End		At C.G.S.		
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	76.74	1760.34	50.17	49.74
Distance = 68 - 76	79.29	1730.33	49.31	

		19-Apr-01							
		East		West					
North						Difference	Strain	Microstrain	Losses (ksi)
Top	60 - 68	705	705	791	789	21.75	0.000176	176	5.02
	68 - 76	711	715	860	860	27.00	0.000219	219	6.23
Bottom	60 - 68	560	560	645	645	222.00	0.001798	1798	51.25
	68 - 76	575	576	565	566	210.50	0.001705	1705	48.59
Top	144 - 152	750	752	609	608	39.00	0.000316	316	9.00
	152 - 160	771	772	941	941	32.25	0.000261	261	7.44
Bottom	144 - 152	561	564	701	700	214.25	0.001735	1735	49.46
	152 - 160	622	624	563	564	221.00	0.00179	1790	51.02
South									
Top	60 - 68	745	743	703	703	33.50	0.000271	271	7.73
	68 - 76	710	710	740	740	24.25	0.000196	196	5.60
Bottom	60 - 68	394	395	555	557	218.75	0.001772	1772	50.50
	68 - 76	570	570	728	729	218.25	0.001768	1768	50.38

Distance for DEMEC to extreme fiber =	19
c.g.s without top strand =	4.86

North End		At C.G.S.		Average
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	85.37	1810.15	51.59	50.25
Distance = 68 - 76	78.23	1716.00	48.91	

Center		At C.G.S.		
	Slope	Strain	Stress (ksi)	
Distance = 144 - 152	74.71	1745.88	49.76	50.55
Distance = 152 - 160	80.47	1801.37	51.34	

South End		At C.G.S.		
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	78.98	1782.93	50.81	50.76
Distance = 68 - 76	82.71	1779.40	50.71	

			25-Jun-01			180				
		East		West						
North						Difference	Strain	Microstrain	Losses (ksi)	
Top	60 - 68	685	685	772	770	41.25	0.000334	334	9.52
	68 - 76	705	705	840	839	41.25	0.000334	334	9.52
Bottom	60 - 68	539	533	627	624	243.75	0.001974	1974	56.27
	68 - 76			545	545	246.50	0.001997	1997	56.90
Top	144 - 152	745	745	597	598	47.50	0.000385	385	10.97
	152 - 160	762	762	925	925	45.00	0.000365	365	10.39
Bottom	144 - 152	545	545	676	676	235.25	0.001906	1906	54.31
	152 - 160	610	607	543	540	239.25	0.001938	1938	55.23
South									
Top	60 - 68	731	731	700	700	41.50	0.000336	336	9.58
	68 - 76	701	699			31.50	0.000255	255	7.27
Bottom	60 - 68	375	375	533	533	240.00	0.001944	1944	55.40
	68 - 76	552	550	705	705	239.50	0.00194	1940	55.29

Distance for DEMEC to extreme fiber = 19

c.g.s without top strand = 4.86

North End		At C.G.S.		Average
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	86.33	1986.46	56.61	56.93
Distance = 68 - 76	87.50	2008.90	57.25	

Center		At C.G.S.		
	Slope	Strain	Stress (ksi)	
Distance = 144 - 152	80.04	1916.73	54.63	55.09
Distance = 152 - 160	82.81	1949.52	55.56	

South End		At C.G.S.		
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	84.62	1955.85	55.74	55.69
Distance = 68 - 76	88.67	1952.36	55.64	

		21-Dec-01				360			
		East		West					
North						Difference	Strain	Microstrain	Losses (ksi)
Top	60 - 68	686	682	774	774	40.25	0.000326	326	9.29
	68 - 76	700	699	846	846	40.75	0.00033	330	9.41
Bottom	60 - 68	527	527	614		253.00	0.002049	2049	58.41
	68 - 76			532		259.00	0.002098	2098	59.79
Top	144 - 152	740	738	597	597	50.75	0.000411	411	11.72
	152 - 160	758	758	929	929	45.00	0.000365	365	10.39
Bottom	144 - 152	535	539	663	662	246.00	0.001993	1993	56.79
	152 - 160			529	528	257.50	0.002086	2086	59.44
South									
Top	60 - 68	738	737	694	692	41.75	0.000338	338	9.64
	68 - 76	698	700			32.50	0.000263	263	7.50
Bottom	60 - 68	355	358	521	521	255.25	0.002068	2068	58.92
	68 - 76	537	535	694	694	252.50	0.002045	2045	58.29

Distance for DEMEC to extreme fiber = 19

c.g.s without top strand = 4.86

North End		At C.G.S.		Average
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	90.70	2062.00	58.77	59.46
Distance = 68 - 76	93.04	2110.93	60.16	

Center		At C.G.S.		
	Slope	Strain	Stress (ksi)	
Distance = 144 - 152	83.24	2004.25	57.12	58.46
Distance = 152 - 160	90.59	2098.43	59.81	

South End		At C.G.S.		
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	91.02	2080.27	59.29	58.98
Distance = 68 - 76	93.79	2058.38	58.66	

The following pages contain the strain measurements for Girder/Beam 4.

		Before Cutting 12/20/00				After Cutting 12/20/00							
		East		West		East		West					
North										Difference	Strain	Microstrain	Losses (ksi)
Top	60 - 68	828	826	693	691	776	778	633	634	54.25	0.00043943	439	12.52
	68 - 76	813	809	876	872	807	809	874	875	1.25	1.0125E-05	10	0.29
Bottom	60 - 68	889	887	789	788	780	779	688	688	104.50	0.00084645	846	24.12
	68 - 76	841	838	786	786	735	734	685	687	102.50	0.00083025	830	23.66
Top	144 - 152	732	732	783	782	729	728	779	779	3.50	0.00002835	28	0.81
	152 - 160	530	529	596	593	532	532	591	590	0.75	6.075E-06	6	0.17
Bottom	144 - 152	787	784	786	784	683	682	688	688	100.00	0.00081	810	23.09
	152 - 160	790	789	778	783	689	687	681	684	99.75	0.00080798	808	23.03
South													
Top	60 - 68	765	765	790	788	762	763	784	787	3.00	0.0000243	24	0.69
	68 - 76	807	809	788	785	792	794	766	765	18.00	0.0001458	146	4.16
Bottom	60 - 68	943	944	787	786	835	835	688	688	103.50	0.00083835	838	23.89
	68 - 76	794	795	790	790	687	688	687	688	104.75	0.00084848	848	24.18

Distance for DEMEC to extreme fiber = 19

c.g.s without top strand = 3.48

North End	**At C.G.S.**			**Average**
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	21.42	879.01	25.05	25.29
Distance = 68 - 76	43.16	895.86	25.53	
Center	**At C.G.S.**			
	Slope	Strain	Stress (ksi)	
Distance = 144 - 152	41.14	872.53	24.87	24.86
Distance = 152 - 160	42.21	872.13	24.86	
South End	**At C.G.S.**			
	Slope	Strain	Stress (ksi)	
Distance = 60 - 68	42.84	903.47	25.75	25.77
Distance = 68 - 76	36.98	904.69	25.78	

		05-Jan-01							
		East		West					
North						Difference	Strain	Microstrain	Losses (ksi)
Top	60 - 68	763	763	615	615	70.50	0.00057105	571	16.27
	68 - 76	804	803	867	866	7.50	0.00006075	61	1.73
Bottom	60 - 68	736	738	640	639	150.00	0.001215	1215	34.63
	68 - 76	694	694	636	638	147.25	0.001192725	1193	33.99
Top	144 - 152	722	721	772	771	10.75	0.000087075	87	2.48
	152 - 160	524	524	577	577	11.50	0.00009315	93	2.65
Bottom	144 - 152	638	638	638	638	147.25	0.001192725	1193	33.99
	152 - 160	644	643	651	640	140.50	0.00113805	1138	32.43
South									
Top	60 - 68	755	750	774	774	13.75	0.000111375	111	3.17
	68 - 76	788	787	760	757	24.25	0.000196425	196	5.60
Bottom	60 - 68	797	797	638	639	147.25	0.001192725	1193	33.99
	68 - 76	645	647	635	637	151.25	0.001225125	1225	34.92
		Distance for DEMEC to extreme fiber =				19			
			c.g.s without top strand =			3.48			
				North End		At C.G.S.			Average
						Slope	Strain	Stress (ksi)	
				Distance = 60 - 68		33.89	1266.52	36.10	36.33
				Distance = 68 - 76		59.58	1283.28	36.57	
				Center		At C.G.S.			
						Slope	Strain	Stress (ksi)	
				Distance = 144 - 152		58.19	1281.18	36.51	35.67
				Distance = 152 - 160		54.99	1221.64	34.82	
				South End		At C.G.S.			
						Slope	Strain	Stress (ksi)	
				Distance = 60 - 68		56.91	1279.23	36.46	36.86
				Distance = 68 - 76		54.14	1307.42	37.26	

		18-Jan-01							
		East		West					
North						Difference	Strain	Microstrain	Losses (ksi)
Top	60 - 68	769	770	630	631	59.50	0.000482	482	13.74
	68 - 76	805	806	874	872	3.25	2.63E-05	26	0.75
Bottom	60 - 68	735	734	638	637	152.25	0.001233	1233	35.15
	68 - 76	689	689	634	635	151.00	0.001223	1223	34.86
Top	144 - 152	727	726	776	776	6.00	4.86E-05	49	1.39
	152 - 160	529	529	586	587	4.25	3.44E-05	34	0.98
Bottom	144 - 152	633	633	639	640	149.00	0.001207	1207	34.40
	152 - 160	639	641	636	634	147.50	0.001195	1195	34.05
South									
Top	60 - 68	758	758	786	786	5.00	4.05E-05	41	1.15
	68 - 76	790	790	762	763	21.00	0.00017	170	4.85
Bottom	60 - 68	789	787	640	640	151.00	0.001223	1223	34.86
	68 - 76	648	647	637	637	150.00	0.001215	1215	34.63
		Distance for DEMEC to extreme fiber =				19			
			c.g.s without top strand =			3.48			
				North End		At C.G.S.			Average
						Slope	Strain	Stress (ksi)	
				Distance = 60 - 68		39.54	1293.33	36.86	37.22
				Distance = 68 - 76		62.99	1318.84	37.59	
				Center		At C.G.S.			
						Slope	Strain	Stress (ksi)	
				Distance = 144 - 152		60.96	1299.56	37.04	36.87
				Distance = 152 - 160		61.07	1287.58	36.70	
				South End		At C.G.S.			
						Slope	Strain	Stress (ksi)	
				Distance = 60 - 68		62.24	1317.71	37.55	37.28
				Distance = 68 - 76		54.99	1298.59	37.01	

		02-Feb-01							
		East		West					
North						Difference	Strain	Microstrain	Losses (ksi)
Top	60 - 68	760	759	619	618	70.50	0.000571	571	16.27
	68 - 76	800	799	867	867	9.25	7.49E-05	75	2.14
Bottom	60 - 68	714	714	615	615	173.75	0.001407	1407	40.11
	68 - 76	669	668	612	612	172.50	0.001397	1397	39.82
Top	144 - 152	720	720	767	770	13.00	0.000105	105	3.00
	152 - 160	520	520	575	575	14.50	0.000117	117	3.35
Bottom	144 - 152	612	611	614	614	172.50	0.001397	1397	39.82
	152 - 160	617	615	610	613	171.25	0.001387	1387	39.53
South									
Top	60 - 68	748	749	775	773	15.75	0.000128	128	3.64
	68 - 76	780	784	756	756	28.25	0.000229	229	6.52
Bottom	60 - 68	766	767	618	616	173.25	0.001403	1403	39.99
	68 - 76	624	622	610	610	175.75	0.001424	1424	40.57
		Distance for DEMEC to extreme fiber =				19			
			c.g.s without top strand =			3.48			
				North End		At C.G.S.			Average
						Slope	Strain	Stress (ksi)	
				Distance = 60 - 68		44.02	1474.28	42.02	42.43
				Distance = 68 - 76		69.60	1503.04	42.84	
				Center		At C.G.S.			
						Slope	Strain	Stress (ksi)	
				Distance = 144 - 152		68.00	1500.61	42.77	42.60
				Distance = 152 - 160		66.83	1488.70	42.43	
				South End		At C.G.S.			
						Slope	Strain	Stress (ksi)	
				Distance = 60 - 68		67.14	1505.39	42.90	43.10
				Distance = 68 - 76		62.88	1519.16	43.30	

		19-Feb-01							
		East		West					
North						Difference	Strain	Microstrain	Losses (ksi)
Top	60 - 68	782	781	630	634	52.75	0.000427	427	12.18
	68 - 76	815	813	885	885	-7.00	-5.67E-05	-57	-1.62
Bottom	60 - 68	724	723	631	631	161.00	0.001304	1304	37.17
	68 - 76	680	680	623	623	161.25	0.001306	1306	37.22
Top	144 - 152	732	734	782	785	-1.00	-8.1E-06	-8	-0.23
	152 - 160	537	538	587	587	-0.25	-2.03E-06	-2	-0.06
Bottom	144 - 152	617	617	629	625	163.25	0.001322	1322	37.69
	152 - 160	623	625	621	618	163.25	0.001322	1322	37.69
South									
Top	60 - 68	762	762	788	790	1.50	1.22E-05	12	0.35
	68 - 76	786	785	771	771	19.00	0.000154	154	4.39
Bottom	60 - 68	775	774	627	627	164.25	0.00133	1330	37.92
	68 - 76	631	629	618	615	169.00	0.001369	1369	39.01
		Distance for DEMEC to extreme fiber =				19			
			c.g.s without top strand =			3.48			
				North End		**At C.G.S.**			**Average**
						Slope	Strain	Stress (ksi)	
				Distance = 60 - 68		46.15	1374.25	39.17	39.75
				Distance = 68 - 76		71.73	1415.15	40.33	
				Center		**At C.G.S.**			
						Slope	Strain	Stress (ksi)	
				Distance = 144 - 152		70.02	1428.76	40.72	40.71
				Distance = 152 - 160		69.70	1428.27	40.71	
				South End		**At C.G.S.**			
						Slope	Strain	Stress (ksi)	
				Distance = 60 - 68		69.38	1435.89	40.92	41.35
				Distance = 68 - 76		63.95	1466.10	41.78	

		14-Mar-01							
		East		West					
North						Difference	Strain	Microstrain	Losses (ksi)
Top	60 - 68	771	769	627	627	61.00	0.000494	494	14.08
	68 - 76	801	802	869	868	7.50	6.08E-05	61	1.73
Bottom	60 - 68	710	710	620	621	173.00	0.001401	1401	39.94
	68 - 76	665	665	615	615	172.75	0.001399	1399	39.88
Top	144 - 152	724	724	774	775	8.00	6.48E-05	65	1.85
	152 - 160	525	525	583	582	8.25	6.68E-05	67	1.90
Bottom	144 - 152	605	605	611	611	177.25	0.001436	1436	40.92
	152 - 160	615	615	607	604	174.75	0.001415	1415	40.34
South									
Top	60 - 68	756	758	781	781	8.00	6.48E-05	65	1.85
	68 - 76	784	782	760	760	25.75	0.000209	209	5.94
Bottom	60 - 68	763	761	617	616	175.75	0.001424	1424	40.57
	68 - 76	616	617	615	610	177.75	0.00144	1440	41.03
		Distance for DEMEC to extreme fiber =				19			
			c.g.s without top strand =			3.48			
				North End		**At C.G.S.**			**Average**
						Slope	Strain	Stress (ksi)	
				Distance = 60 - 68		47.75	1473.88	42.01	42.47
				Distance = 68 - 76		70.45	1506.36	42.93	
				Center		**At C.G.S.**			
						Slope	Strain	Stress (ksi)	
				Distance = 144 - 152		72.15	1545.40	44.04	43.73
				Distance = 152 - 160		70.98	1523.37	43.42	
				South End		**At C.G.S.**			
						Slope	Strain	Stress (ksi)	
				Distance = 60 - 68		71.51	1532.28	43.67	43.76
				Distance = 68 - 76		64.80	1538.27	43.84	

		19-Apr-01							
		East		West					
North						Difference	Strain	Microstrain	Losses (ksi)
Top	60 - 68	782	782	635	638	50.25	0.000407025	407	11.60
	68 - 76	812	813	883	885	-5.75	-0.000046575	-47	-1.33
Bottom	60 - 68	706	705	610	609	180.75	0.001464075	1464	41.73
	68 - 76	662	662	607	609	177.75	0.001439775	1440	41.03
Top	144 - 152	725	725	779	780	5.00	0.0000405	41	1.15
	152 - 160	539	538	588	586	-0.75	-0.000006075	-6	-0.17
Bottom	144 - 152	601	603	606	605	181.50	0.00147015	1470	41.90
	152 - 160	614	615	590	595	181.50	0.00147015	1470	41.90
South									
Top	60 - 68	759	759	789	790	2.75	0.000022275	22	0.63
	68 - 76	798	795	766	767	15.75	0.000127575	128	3.64
Bottom	60 - 68	758	756	610	610	181.50	0.00147015	1470	41.90
	68 - 76	615	614	613		179.00	0.0014499	1450	41.32
		Distance for DEMEC to extreme fiber =				19			
			c.g.s without top strand =			3.48			
				North End		**At C.G.S.**			**Average**
						Slope	Strain	Stress (ksi)	
				Distance = 60 - 68		55.63	1548.64	44.14	44.28
				Distance = 68 - 76		78.23	1558.68	44.42	
				Center		**At C.G.S.**			
						Slope	Strain	Stress (ksi)	
				Distance = 144 - 152		75.24	1584.52	45.16	45.21
				Distance = 152 - 160		77.70	1588.25	45.27	
				South End		**At C.G.S.**			
						Slope	Strain	Stress (ksi)	
				Distance = 60 - 68		76.20	1585.98	45.20	44.77
				Distance = 68 - 76		69.60	1555.69	44.34	

		25-Jun-01				180 days			
		East		West					
North						Difference	Strain	Microstrain	Losses (ksi)
Top	60 - 68	778	775	620	625	60.00	0.000486	486	13.85
	68 - 76	795	792	865	864	13.50	0.00010935	109	3.12
Bottom	60 - 68	688	688	595	595	196.75	0.00159368	1594	45.42
	68 - 76	645	645	589	590	195.50	0.00158355	1584	45.13
Top	144 - 152	700	698	768	770	23.25	0.00018833	188	5.37
	152 - 160	553	550	576	580	-2.75	-2.228E-05	-22	-0.63
Bottom	144 - 152	584	585	588	588	199.00	0.0016119	1612	45.94
	152 - 160	590	585	588	585	198.00	0.0016038	1604	45.71
South									
Top	60 - 68	753	755	792	790	4.50	0.00003645	36	1.04
	68 - 76			759	760	27.00	0.0002187	219	6.23
Bottom	60 - 68	740	740	595	595	197.50	0.00159975	1600	45.59
	68 - 76	601	600			194.00	0.0015714	1571	44.78
		Distance for DEMEC to extreme fiber =				19			
			c.g.s without top strand =			3.48			
				North End		**At C.G.S.**			**Average**
						Slope	Strain	Stress (ksi)	
				Distance = 60 - 68		58.30	1682.29	47.95	48.22
				Distance = 68 - 76		77.59	1701.49	48.49	
				Center		**At C.G.S.**			
						Slope	Strain	Stress (ksi)	
				Distance = 144 - 152		74.93	1725.79	49.18	49.30
				Distance = 152 - 160		85.58	1733.89	49.42	
				South End		**At C.G.S.**			
						Slope	Strain	Stress (ksi)	
				Distance = 60 - 68		82.28	1724.81	49.16	48.51
				Distance = 68 - 76		71.19	1679.62	47.87	

		21-Dec-01				360			
		East		West					
North						Difference	Strain	Microstrain	Losses (ksi)
Top	60 - 68	781	778	621	620	59.50	0.000482	482	13.74
	68 - 76	779	779	853	852	26.75	0.000217	217	6.18
Bottom	60 - 68	679	678	581	583	208.00	0.001685	1685	48.02
	68 - 76	634	634	580	579	206.00	0.001669	1669	47.56
Top	144 - 152	679	675	769	770	34.00	0.000275	275	7.85
	152 - 160	518	521	580	578	12.75	0.000103	103	2.94
Bottom	144 - 152	575	574	577	579	209.00	0.001693	1693	48.25
	152 - 160	581	580	570	569	210.00	0.001701	1701	48.48
South									
Top	60 - 68	747	746	763	765	21.75	0.000176	176	5.02
	68 - 76			773	772	14.00	0.000113	113	3.23
Bottom	60 - 68	729	729	580	579	210.75	0.001707	1707	48.65
	68 - 76	591	588			205.00	0.001661	1661	47.32
		Distance for DEMEC to extreme fiber =				19			
			c.g.s without top strand =			3.48			
				North End		At C.G.S.			Average
						Slope	Strain	Stress (ksi)	
				Distance = 60 - 68		63.31	1781.03	50.76	50.81
				Distance = 68 - 76		76.42	1784.75	50.87	
				Center		At C.G.S.			
						Slope	Strain	Stress (ksi)	
				Distance = 144 - 152		74.61	1806.30	51.48	51.80
				Distance = 152 - 160		84.09	1828.82	52.12	
				South End		At C.G.S.			
						Slope	Strain	Stress (ksi)	
				Distance = 60 - 68		80.57	1829.55	52.14	51.50
				Distance = 68 - 76		81.43	1784.27	50.85	

Hardened Concrete Properties for the Girders/Beams.

Table E.1. Compressive Strength for Girders 1 and 4.

Compressive Strength for Girders 1 and 4 (2.3 % Air)					
Age (days)	Specimens				Average (psi)
	1	2	3	4	
1	8438	8915	8278	9154	8696
14	10026	10417	10137		10193
28	11668	10854	11163	10538	11056
56	12419	12438	12460		12439
180	13948	14719	15040	14128	14459
360	16270	15492	15392	15301	15614

Table E.2. Compressive Strength for Girders 2 and 3.

Compressive Strength for Girders 2 and 3 (6.2 % Air)					
Age (days)	Specimens				Average (psi)
	1	2	3	4	
1	6048	6207	6485	6048	6197
14	6791	7168	7306	7161	7107
28	7797	8507	8650	8608	8391
56	9480	9293	8816	9272	9215
180	10845	11007	10250	10908	10753
360	11297	11287	12134	11128	11462

Shrinkage for Girders 1 and 4.

One - Day		
Reference Bar	Prism Length	Difference
0.0793	0.0512	0.0281
0.0793	0.1049	-0.0256
0.0793	0.1095	-0.0302

14 Day				
Measurement	Difference	Change in Length	Strain	Micro-Strain
0.0471	0.0322	-0.0041	-0.00041	-410
0.1001	-0.0208	-0.0048	-0.00048	-480
0.1055	-0.0262	-0.004	-0.0004	-400

28 day				
Measurement	Difference	Change in Length	Strain	Micro-Strain
0.0451	0.0342	-0.0061	-0.00061	-610
0.0992	-0.0199	-0.0057	-0.00057	-570
0.1035	-0.0242	-0.006	-0.0006	-600

43 day				
Measurement	Difference	Change in Length	Strain	Micro-Strain
0.0447	0.0346	-0.0065	-0.00065	-650
0.0986	-0.0193	-0.0063	-0.00063	-630
0.1035	-0.0242	-0.006	-0.0006	-600

60 day				
Measurement	Difference	Change in Length	Strain	Micro-Strain
0.0447	0.0346	-0.0065	-0.00065	-650
0.0988	-0.0195	-0.0061	-0.00061	-610
0.1032	-0.0239	-0.0063	-0.00063	-630

84 day				
Measurement	Difference	Change in Length	Strain	Micro-Strain
0.0446	0.0347	-0.0066	-0.00066	-660
0.0988	-0.0195	-0.0061	-0.00061	-610
0.103	-0.0237	-0.0065	-0.00065	-650

120 day				
Measurement	Difference	Change in Length	Strain	Micro-Strain
0.0444	0.0349	-0.0068	-0.00068	-680
0.0987	-0.0194	-0.0062	-0.00062	-620
0.1026	-0.0233	-0.0069	-0.00069	-690

180 day				
Measurement	Difference	Change in Length	Strain	Micro-Strain
0.0443	0.035	-0.0069	-0.00069	-690
0.0984	-0.0191	-0.0065	-0.00065	-650
0.1025	-0.0232	-0.007	-0.0007	-700

360 day				
Measurement	Difference	Change in Length	Strain	Micro-Strain
0.0442	0.0351	-0.007	-0.0007	-700
0.0986	-0.0193	-0.0063	-0.00063	-630
0.1023	-0.023	-0.0072	-0.00072	-720

Shrinkage for Girders 2 and 3.

One - Day		
Reference Bar	Prism Length	Difference
0.0893	0.1102	-0.0209
0.0893	0.0545	0.0348
0.0893	0.1053	-0.016

14 Day				
Measurement	Difference	Change in Length	Strain	Micro-Strain
0.1057	-0.0164	-0.0045	-0.00045	-450
0.0492	0.0401	-0.0053	-0.00053	-530
0.1017	-0.0124	-0.0036	-0.00036	-360

28 day				
Measurement	Difference	Change in Length	Strain	Micro-Strain
0.1037	-0.0144	-0.0065	-0.00065	-650
0.0477	0.0416	-0.0068	-0.00068	-680
0.1002	-0.0109	-0.0051	-0.00051	-510

43 day				
Measurement	Difference	Change in Length	Strain	Micro-Strain
0.1033	-0.014	-0.0069	-0.00069	-690
0.0475	0.0418	-0.007	-0.0007	-700
0.0997	-0.0104	-0.0056	-0.00056	-560

60 day				
Measurement	Difference	Change in Length	Strain	Micro-Strain
0.1031	-0.0138	-0.0071	-0.00071	-710
0.047	0.0423	-0.0075	-0.00075	-750
0.0995	-0.0102	-0.0058	-0.00058	-580

84 day				
Measurement	Difference	Change in Length	Strain	Micro-Strain
0.103	-0.0137	-0.0072	-0.00072	-720
0.0471	0.0422	-0.0074	-0.00074	-740
0.0996	-0.0103	-0.0057	-0.00057	-570

120 day				
Measurement	Difference	Change in Length	Strain	Micro-Strain
0.1028	-0.0135	-0.0074	-0.00074	-740
0.0469	0.0424	-0.0076	-0.00076	-760
0.0995	-0.0102	-0.0058	-0.00058	-580

180 day				
Measurement	Difference	Change in Length	Strain	Micro-Strain
0.1026	-0.0133	-0.0076	-0.00076	-760
0.0466	0.0427	-0.0079	-0.00079	-790
0.0991	-0.0098	-0.0062	-0.00062	-620

360 day				
Measurement	Difference	Change in Length	Strain	Micro-Strain
0.1026	-0.0133	-0.0076	-0.00076	-760
0.0463	0.043	-0.0082	-0.00082	-820
0.0988	-0.0095	-0.0065	-0.00065	-650

Modulus of Elasticity for Girders 1 and 4.

Age 14 days

Specimen	40% fc (lbs)	40% fc (psi)	Load (lbs) @ 0.00005	Stress (psi) @ 0.00005	Strain @ 40% fc		Change in Stress	Change in Strain	MOE (ksi)	Average MOE (ksi)
1	50224	4000	4620	368	0.000677	0.00005	3632	0.000627	5793	
1	50224	4000	4830	384	0.000678	0.00005	3616	0.000628	5757	
2	50224	4000	4365	347	0.000691	0.00005	3653	0.000641	5698	**5768**
2	50224	4000	4605	366	0.000689	0.00005	3634	0.000639	5686	
3	50224	4000	4560	363	0.000675	0.00005	3637	0.000625	5819	
3	50224	4000	4425	352	0.000673	0.00005	3648	0.000623	5855	

Age 28 days

Specimen	40% fc (lbs)	40% fc (psi)	Load (lbs) @ 0.00005	Stress (psi) @ 0.00005	Strain @ 40% fc		Change in Stress	Change in Strain	MOE (ksi)	Average MOE (ksi)
1	58650	4671	4365	347	0.000757	0.00005	4324	0.000707	6116	
1	58650	4671	4395	350	0.000747	0.00005	4321	0.000697	6200	
2	58650	4671	4920	392	0.000766	0.00005	4280	0.000716	5977	**6054**
2	58650	4671	4560	363	0.000763	0.00005	4308	0.000713	6042	
3	58650	4671	3825	304	0.000782	0.00005	4367	0.000732	5965	
3	58650	4671	4125	328	0.000771	0.00005	4343	0.000721	6023	

Age 56 days

Specimen	40% fc (lbs)	40% fc (psi)	Load (lbs) @ 0.00005	Stress (psi) @ 0.00005	Strain @ 40% fc		Change in Stress	Change in Strain	MOE (ksi)	Average MOE (ksi)
1	62425	4972	5000	398	0.000777	0.00005	4574	0.000727	6291	
1	62425	4972	5020	399	0.000774	0.00005	4572	0.000724	6315	
2	62425	4972	4460	355	0.000798	0.00005	4617	0.000748	6172	**6331**
2	62425	4972	4420	352	0.000795	0.00005	4620	0.000745	6201	
3	62425	4972	4720	376	0.000759	0.00005	4596	0.000709	6483	
3	62425	4972	4920	392	0.000752	0.00005	4580	0.000702	6524	

Age 180 days

Specimen	40% fc (lbs)	40% fc (psi)	Load (lbs) @ 0.00005	Stress (psi) @ 0.00005	Strain @ 40% fc		Change in Stress	Change in Strain	MOE (ksi)	Average MOE (ksi)
1	70052	5579	4920	392	0.000819	0.00005	5188	0.000769	6746	
1	70052	5579	4980	396	0.000815	0.00005	5183	0.000765	6775	
2	70052	5579	4760	379	0.000810	0.00005	5200	0.000760	6843	**6773**
2	70052	5579	4780	380	0.000798	0.00005	5199	0.000748	6950	
3	70052	5579	4860	387	0.000831	0.00005	5192	0.000781	6648	
3	70052	5579	5080	404	0.000825	0.00005	5175	0.000775	6677	

Age 360 days

Specimen	40% fc (lbs)	40% fc (psi)	Load (lbs) @ 0.00005	Stress (psi) @ 0.00005	Strain @ 40% fc		Change in Stress	Change in Strain	MOE (ksi)	Average MOE (ksi)
1	79760	6352	4980	396	0.0009	0.00005	5956	0.00085	7007	
1	79760	6352	4440	353	0.000905	0.00005	5999	0.000855	7016	
2	79760	6352	4940	393	0.001006	0.00005	5959	0.000956	6233	**6913**
2	79760	6352	4880	388	0.000892	0.00005	5964	0.000842	7083	
3	79760	6352	4440	353	0.000899	0.00005	5999	0.000849	7066	
3	79760	6352	5060	403	0.000891	0.00005	5950	0.000841	7075	

Modulus of Elasticity for Girders 2 and 3.

Age 14 days

Specimen	40% fc (lbs)	40% fc (psi)	Load (lbs) @ 0.00005	Stress (psi) @ 0.00005	Strain @ 40% fc		Change in Stress	Change in Strain	MOE (ksi)	Average MOE (ksi)
1	35600	2835	3990	318	0.000547	0.00005	2518	0.000497	5066	
1	35600	2835	3840	306	0.000547	0.00005	2530	0.000497	5090	
2	35600	2835	3330	265	0.000583	0.00005	2570	0.000533	4822	**4959**
2	35600	2835	3765	300	0.000574	0.00005	2536	0.000524	4839	
3	35600	2835	3960	315	0.000542	0.00005	2520	0.000492	5122	
3	35600	2835	3945	314	0.000574	0.00005	2521	0.000524	4812	

Age 28 days

Specimen	40% fc (lbs)	40% fc (psi)	Load (lbs) @ 0.00005	Stress (psi) @ 0.00005	Strain @ 40% fc		Change in Stress	Change in Strain	MOE (ksi)	Average MOE (ksi)
1	42172	3359	3960	315	0.000613	0.00005	3044	0.000563	5406	
1	42172	3359	4020	320	0.000604	0.00005	3039	0.000554	5485	
2	42172	3359	4080	325	0.000585	0.00005	3034	0.000535	5671	**5490**
2	42172	3359	4110	327	0.000585	0.00005	3032	0.000535	5667	
3	42172	3359	4125	328	0.000620	0.00005	3030	0.00057	5317	
3	42172	3359	4035	321	0.000613	0.00005	3038	0.000563	5395	

Age 56 days

Specimen	40% fc (lbs)	40% fc (psi)	Load (lbs) @ 0.00005	Stress (psi) @ 0.00005	Strain @ 40% fc		Change in Stress	Change in Strain	MOE (ksi)	Average MOE (ksi)
1	46961	3740	4040	322	0.000684	0.00005	3419	0.000634	5392	
1	46961	3740	3780	301	0.000687	0.00005	3439	0.000637	5399	
2	46961	3740	3720	296	0.000704	0.00005	3444	0.000654	5266	**5375**
2	46961	3740	3900	310	0.000697	0.00005	3430	0.000647	5301	
3	46961	3740	3940	314	0.000681	0.00005	3427	0.000631	5430	
3	46961	3740	3980	317	0.000677	0.00005	3423	0.000627	5460	

Age 180 days

Specimen	40% fc (lbs)	40% fc (psi)	Load (lbs) @ 0.00005	Stress (psi) @ 0.00005	Strain @ 40% fc		Change in Stress	Change in Strain	MOE (ksi)	Average MOE (ksi)
1	54874	4370	3500	279	0.000773	0.00005	4092	0.000723	5659	
1	54874	4370	3840	306	0.000759	0.00005	4065	0.000709	5733	
2	54874	4370	4080	325	0.000781	0.00005	4046	0.000731	5534	5534
2	54874	4370	4220	336	0.000769	0.00005	4035	0.000719	5611	
3	54874	4370	3740	298	0.000819	0.00005	4073	0.000769	5296	
3	54874	4370	4040	322	0.000804	0.00005	4049	0.000754	5370	

Age 360 days

Specimen	40% fc (lbs)	40% fc (psi)	Load (lbs) @ 0.00005	Stress (psi) @ 0.00005	Strain @ 40% fc		Change in Stress	Change in Strain	MOE (ksi)	Average MOE (ksi)
1	56712	4517	4280	341	0.000743	0.00005	4176	0.000693	6026	
1	56712	4517	4380	349	0.000738	0.00005	4168	0.000688	6058	
2	56712	4517	3620	288	0.000756	0.00005	4229	0.000706	5990	5992
2	56712	4517	3740	298	0.000748	0.00005	4219	0.000698	6045	
3	56712	4517	3660	291	0.000765	0.00005	4225	0.000715	5910	
3	56712	4517	4060	323	0.000758	0.00005	4194	0.000708	5923	

Calculations for Prestress Losses due to Temperature Change.

Thermal expansion for concrete and strand $= 6 \times 10^{-6}$ in./in./ degree F

Modulus of Elasticity of the strand = 28,500 ksi

Assume a 20 F change in temperature:

Loss = (6×10^{-6} in./in./ degree F)(28,500 ksi)(20 F)
Loss = 3.42 ksi

Assume a 30 F change in temperature:

Loss = (6×10^{-6} in./in./ degree F)(28,500 ksi)(30 F)
Loss = 5.13 ksi

Calculations for Modified Kcir for Elastic Shortening Losses.

For Girder 1:

Assume Kcir = 0.90

Calculate ES Losses

$f_{cir} = K_{cir}(P_{pi}(1/A_g + e^2/I_g)) - f_g$

$ES = K_{es}nf_{cir}$

ES = 21.30 ksi

Calculate Kcir by

Kcir = (202.5 – 21.30)/202.5

Kcir = 0.89

Try Kcir = 0.89

Calculate ES Losses

$f_{cir} = K_{cir}(P_{pi}(1/A_g + e^2/I_g)) - f_g$

$ES = K_{es}nf_{cir}$

ES = 21.06 ksi

Calculate Kcir by

Kcir = (202.5 – 21.06)/202.5

Kcir = 0.896

Try Kcir = 0.895

Calculate ES Losses

$f_{cir} = K_{cir}(P_{pi}(1/A_g + e^2/I_g)) - f_g$

$ES = K_{es}nf_{cir}$

ES = 21.18 ksi

Calculate Kcir by

Kcir = (202.5 – 21.18)/202.5

Kcir = 0.895

0.895 = 0.895 Okay

Use Kcir = 0.895

For Girder 2:

Assume Kcir = 0.90

Calculate ES Losses

$f_{cir} = K_{cir}(P_{pi}(1/A_g + e^2/I_g)) - f_g$

$ES = K_{es}nf_{cir}$

ES = 23.67 ksi

Calculate Kcir by

Kcir = (202.5 – 23.67)/202.5

Kcir = 0.88

Try Kcir = 0.88

Calculate ES Losses

$f_{cir} = K_{cir}(P_{pi}(1/A_g + e^2/I_g)) - f_g$

$ES = K_{es}nf_{cir}$

ES = 23.14 ksi

Calculate Kcir by

Kcir = (202.5 – 23.14)/202.5

Kcir = 0.886

Try Kcir = 0.886

Calculate ES Losses

$f_{cir} = K_{cir}(P_{pi}(1/A_g + e^2/I_g)) - f_g$

$ES = K_{es}nf_{cir}$

ES = 23.30 ksi

Calculate Kcir by

Kcir = (202.5 – 23.30)/202.5

Kcir = 0.885

0.886 = 0.885 Okay

Use Kcir = 0.886

For Girder 3:

Assume Kcir = 0.90

Calculate ES Losses

$f_{cir} = K_{cir}(P_{pi}(1/A_g + e^2/I_g)) - f_g$

$ES = K_{es} n f_{cir}$

ES = 18.66 ksi

Calculate Kcir by

Kcir = (202.5 – 18.66)/202.5

Kcir = 0.91

Try Kcir = 0.91

Calculate ES Losses

$f_{cir} = K_{cir}(P_{pi}(1/A_g + e^2/I_g)) - f_g$

$ES = K_{es} n f_{cir}$

ES = 18.86 ksi

Calculate Kcir by

Kcir = (202.5 – 18.86)/202.5

Kcir = 0.91

0.91 = 0.91 Okay

Use Kcir = 0.91

For Girder 4:

Assume Kcir = 0.90

Calculate ES Losses

$f_{cir} = K_{cir}(P_{pi}(1/A_g + e^2/I_g)) - f_g$

$ES = K_{es} n f_{cir}$

ES = 18.37 ksi

Calculate Kcir by

Kcir = (202.5 – 18.37)/202.5

Kcir = 0.91

Try Kcir = 0.91

Calculate ES Losses

$f_{cir} = K_{cir}(P_{pi}(1/A_g + e^2/I_g)) - f_g$

$ES = K_{es} n f_{cir}$

ES = 18.58 ksi

Calculate Kcir by

Kcir = (202.5 – 18.58)/202.5

Kcir = 0.91

0.91 = 0.91 Okay

Use Kcir = 0.91