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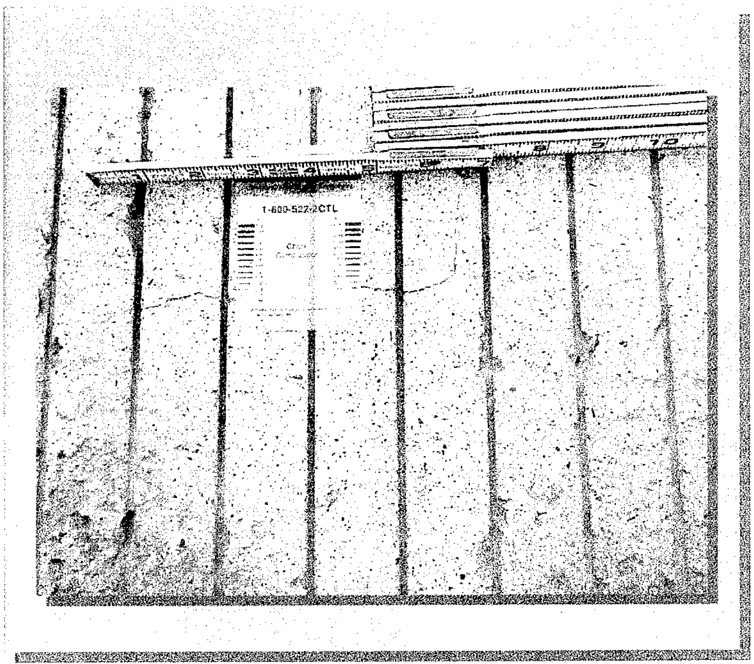

Determination of Shrinkage Characteristics of Concretes with Type K Cement, Mineral and Chemical Additives

Prepared by the University of New Hampshire, Department of Civil Engineering in cooperation with the U.S. DOT, Federal Highway Administration

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16. Abstract

Laboratory and field work was conducted to evaluate shrinkage and cracking tendency of various concrete mix designs.

Two simple span bridge decks were placed on Rt. 101 over Bloody Brook in Exeter NH during the summer of 1997. One of these decks was made with Type k cement and the other was a control mix design using a Type II low alkali cement with a 50 percent substitution of ground granulated blast furnace slag (GGBFS). Both decks were instrumented with thermocouples and strain gauges bonded to reinforcement in the transverse and longitudinal directions. Relative humidity, wind speed, wind direction, solar intensity, rainfall, air and deck temperature, and reinforcement strain were monitored. Neither deck developed visible cracks in the first year of monitoring.

Laboratory testing using modified cracking tendency rings and prisms showed that some mix designs are effective in reducing shrinkage. An expansive Type k cement was found to offset drying shrinkage. A shrinkage reducing admixture (SRA) at a mid dosage was shown to significantly reduce drying shrinkage. Water reducers, silica fume and high alkali cement were shown to in general increase shrinkage. Low alkali cement, GGBFS and SRA were shown to in general lower shrinkage.

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Cracking, durability, shrinkage,
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Final Report

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And
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CHAPTER I

INTRODUCTION

This country spends over 5 billion dollars a year designing and maintaining our roads and bridges. (1) Currently there are over 100 billion dollars in overdue bridge repairs that are multiplying into much more expensive fixes due to neglect. Increasing the life expectancy of the materials used in construction can reduce much of this cost. Increasing the life span of concrete structures exposed to deicing salts is a direct function of slowing the corrosion rate of the steel reinforcement. Chloride ion penetration greatly increases the rate of corrosion.

Reducing non-structural cracking of concrete caused primarily by drying shrinkage is one method of modifying the rate of corrosion. Free shrinkage is the length change that concrete would be expected to undergo without external restraint. Free shrinkage can be directly related to in place drying shrinkage and cracking tendency.

Drying shrinkage is due to water loss in the hydrated cement paste (hcp). Losses of water from three pore size ranges in the hcp structure have an affect on shrinkage. (2) The three size ranges are:

1)	Capilllary voids	5 – 50 nm
2)	Adsorbed water	< 1.5 nm
3)	Interlayer spacing	1 – 4 nm

Capillary water is defined as water held in pores greater than 5 nm. Loss of water from capillaries ranging from 5 to 50 nm in size contributes to shrinkage since its removal induces compressive stresses on the pore walls. A simplified version of the stress caused by shrinkage can be estimated by $\sigma=(2\times\Upsilon)/r$ where Υ is the surface tension and r the radius of the pore being drained. Decreasing Υ would reduce compressive stresses and therefore reduce shrinkage.

Shrinkage reducing agents (SRA) can reduce these pressures by lowering the surface tension at the pore-pore solution interface. Surface tension is due to the difference between the attraction of surface molecules of a liquid and inner molecules which have nearest neighbors in three directions. The surface of the liquid acts like a stretched membrane with in plane forces measured in force per unit length pulling on the pore walls. For a given pore size drying shrinkage strain is linearly related to surface tension. Lowering the surface tension of pore water results in lower shrinkage values. (3) Pores greater than 50 nm contain what is known as free water and do not contribute to shrinkage when drained since the radius is large compared to surface tension.

Loss of water that is physically adsorbed onto the surface solids of the hcp is mainly responsible for drying shrinkage. Researchers have theorized that 6 layers of closely packed water molecules can be attached by hydrogen bonding. A major part of the adsorbed water can be lost by drying the hcp to 30 percent relative humidity. Some suggest that when confined between narrow spaces adsorbed water can exert a disjoining pressure on the hcp matrix. Loss of water reduces disjoining pressure and causes shrinkage. (2) Interlayer water is associated with the calcium silicate hydrate (CSH) structure, which is the binding agent of the hcp structure, and is lost only in cases of extremely low relative humidity. The CSH structure shrinks significantly when the water between the CSH layers, known as interlayer water, is lost.

Current Practice

Contractors are usually on a fast track, wanting to remove formwork as soon as possible. Curing of concrete is a secondary consideration in most jobs. Most agencies place normal concrete without significant additives and specify moist cure with burlap for three to seven days. A sealer that degrades with exposure to ultraviolet light is then sprayed on the surface to continue curing. The most commonly cited reason for cracking related to construction practices is improper curing. (4)

The contractor selected the mix used as a control in the west bound bridge deck over Bloody Brook. The major difference between a conventional bridge deck concrete mix and this mix is the substitution of Ground Granulated Blast Furnace Slag (GGBFS) for a significant portion of the cement. Substitution of GGBFS for portland cement lowers temperature gain during hydration and helps develop a less permeable matrix. Using GGBFS to produce high strength concretes tends to reduce permeability and the likelihood of thermal cracking; however, it can have an effect on drying shrinkage.

Increasing Cracking

The organization now known as the American Association of State Highway Transportation Officials (AASHTO) changed their specifications of bridge deck concrete in 1974. A bridge deck minimum compressive strength of 4500 psi, a minimum cement content of 611 pounds per cubic yard and a water cement ratio no greater than 0.45 were recommended for bridge deck concretes. (4)

A recent survey showed that 62% of DOT agencies believe that early transverse cracking is a problem, 15% of these agencies believe that every deck they pour develops early transverse cracks. Approximately 50% of all decks nationwide develop transverse cracks. Out of the agencies that consider early transverse cracking to be a problem, 93% use water reducers and only two put limits on crack width. (4)

Higher strength concretes tend to have high early strength, high early modulus of elasticity, high cement content, and a low coefficient of creep. This combination provides for high shrinkage potential, large tensile stresses for a given strain, and little possibility for stress relief through creep. These higher strength concretes have higher cement contents and therefore a higher heat of hydration, increasing the role of thermal stresses in crack formation. These mixes are typically made with smaller maximum aggregate size which also increase the shrinkage. Large aggregates with the possible exception of increased D-cracking are superior to smaller aggregates in reducing shrinkage. (5) Increasing the maximum aggregate size reduces the volume of cement paste required in a given mix and therefore reduces shrinkage. Increasing the aggregate content in general reduces shrinkage, especially if the aggregate has a high elastic modulus.

Many admixtures hold potential in maintaining low permeability and high strength while reducing crack formation. Shrinkage may be minimized by using a shrinkage compensated cement, or chemical and mineral admixtures. Some additives have little effect on free shrinkage but can help reduce hydration temperatures and therefore also reduce the risk of early thermal contraction cracking.

Research Objectives

The objectives of this research project were to field test two geometrically identical bridge decks on Rt. 101 in Exeter NH, one made with Type k cement mix and one with a control mix, to laboratory evaluate these two field mixes and other potential mixes, and to evaluate a modified ring test apparatus for expansive concretes.

To accomplish these objectives the bridges were monitored for temperature, reinforcement stresses, and cracking. Environmental conditions such as ambient temperature, solar intensity, wind speed and relative humidity were measured with a weather station. Laboratory testing to determine shrinkage characteristics included free shrinkage beams and restrained rings. The restrained ring test was modified specifically to provide information about expansion potential, shrinkage characteristics, and cracking tendency of potential bridge deck mixes.

CHAPTER II

PRINCIPLES OF CRACKING

Types of Cracking

Cracks can be divided into 2 major categories, before hardening and after hardening. Plastic and construction movement cracking can occur before hardening. Cracking can be further broken down into structural and non-structural cracking, with the non-structural type being further subdivided into plastic, physical, thermal, and chemical cracking. (6) Physical cracking was considered in this study.

Plastic Shrinkage Cracking

Plastic shrinkage cracking occurs when the rate of surface evaporation exceeds the rate of bleeding. The likelihood of plastic cracking depends mainly on environmental factors that determine the evaporation rate, curing conditions, and water cement ratio. Mix designs that limit bleed water are more prone to plastic cracking than others. For instance, a typical high performance concrete (HCP) is extremely likely to undergo plastic shrinkage cracking due to high cement contents, low water to cementious ratios and the use of water reducing and air entraining admixtures. Admixtures, which limit bleeding, are prone to plastic shrinkage cracking. In order to limit plastic shrinkage, cracking the rate of bleeding must be greater than or equal to the evaporation rate.

Factors that reduce bleeding are air entrainment, high quantities of ultra-fine components, and low water content. (6) All of these factors are commonly present in bridge deck concrete. On days where the evaporation exceeds bleeding, appropriate procedures must be used to lower the evaporation rate over the deck. Rapid curing also helps reduce the likelihood of plastic cracking; however plastic cracks frequently form before any curing methods can be adequately implemented. (4)

Admixtures that additionally limit bleeding must be carefully evaluated. For instance, a silica fume admixture reduces bleeding, doesn't significantly reduce heat of hydration, increases modulus of elasticity, and increases cohesion. The relatively high hydration temperatures and the increased cohesion of silica fume concretes can be attributed to a particle size distribution 100 times finer than that of conventional portland cements and fly ashes. (2) All of these factors contribute to an increased likelihood of shrinkage cracking. However air entrainment also reduces bleeding but may help reduce cracking due to a lowering in the early modulus of elasticity. It has also been suggested that the surfactant properties of air entrainers reduce surface tension and compressive stresses upon pore drying, similar to a SRA, which reduces plastic shrinkage cracking and long term shrinkage. (6)

Physical and Thermal Cracking

Shrinkable and/or low modulus aggregates, finishing methods, improper curing procedures, and drying shrinkage cause physical cracking. Drying shrinkage is the most influential of these factors for thin members such as bridge decks. Early thermal contraction, temperature gradients, temperature cycles, and seasonal variations also cause thermal cracking. Restraint combined with internal temperature gradients or high hydration temperatures are causes of thermal cracking. Due to the thin nature of bridge decks, diurnal temperature gradients tend to be relatively small and do not significantly affect cracking, however a bridge deck's low volume to surface area increases the rate of water loss. This increased rate of water loss expedites drying shrinkage and the likelihood of moisture gradient cracking. (7) Early thermal contraction is the most important of the thermal factors in bridge deck construction. Mixes that limit heat of hydration and insulation techniques have proven successful in minimizing thermal contraction cracking. Low heat of hydration mixes limit the overall change in temperature that the structure will see in service. Insulation techniques have proven to be successful in allowing more tensile capacity to develop before thermal contraction occurs.

Chemical Cracking

Chemical cracking is greatly accelerated by the presence of cracks from other causes. Chemical deterioration due to cement carbonation and corrosion of reinforcement are the most common problems accelerated by intrinsic cracking. Other chemical forms of deterioration are Alkali-Silicate-Reaction (ASR) and Alkali-Carbonate-Reaction (ACR). Using non-reactive aggregates, limiting the alkali content, using pozolanic materials, and lithium nitrate can generally avoid ASR and ACR and therefore chemical cracking in new concrete.

Physical and thermal cracking provide a means of entry for air, water and de-icing chemicals to the reinforcement and any chemical reactions that may be occurring within the concrete. Intrinsic cracks such as thermal contraction, plastic shrinkage, and drying shrinkage need to be carefully evaluated to optimize bridge deck concrete. The reduction of intrinsic cracking will greatly reduce the rate of chemical deterioration and increase the service life.

CHAPTER III

EXPERIMENTAL EQUIPMENT DESIGN

Cracking Tendency Rings

In an effort to quantify shrinkage characteristics and cracking tendency a single inner ring test, as shown in figure 1, was developed and used by several researchers. (4,7,8,9) The NCHRP report 380 recommends the adoption of the single ring test by AASHTO to determine cracking tendency of concrete mixes. (4)

The single ring test procedure consists of casting a concrete specimen with a removable PVC outer ring. This outer ring acts only as a form and is removed after the first day of curing. The inner ring provides restraint against shrinkage. The test procedure is to monitor the inner ring until a sudden change in strain indicates that cracking has occurred.

Since the single ring test provides little to no restraint against expansion its use to evaluate expansive cements is limited. For instance when expansive concrete expands during hydration it must be effectively restrained so that a compressive stress develops, otherwise it is very likely to shrink more than a conventional concrete. The induced compressive stresses counteract tensile stresses generated by later drying shrinkage and eliminate cracking. A single ring mold can therefore not be used to evaluate an expansive concrete mix.

The dual ring test as shown by figure 2 measures strain in the inner and outer rings yielding information about expansive potential, shrinkage characteristics, and cracking tendency. This modified ring test was developed to evaluate expansive concretes. The possibility of quantifying thermal contributions with the double ring test also exists. The single inner ring test assembly of figure 1 forces all water to migrate radially outward through the surface of the outer perimeter of the ring because the sides are sealed.

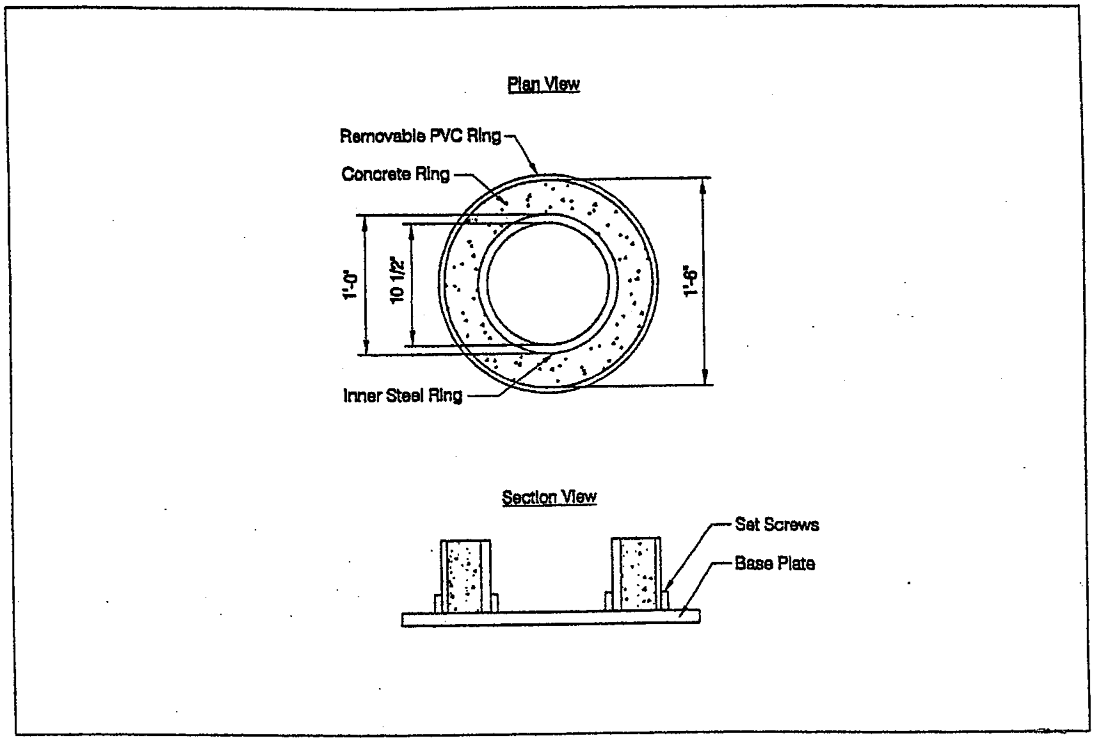

Figure 1. Existing single ring test.

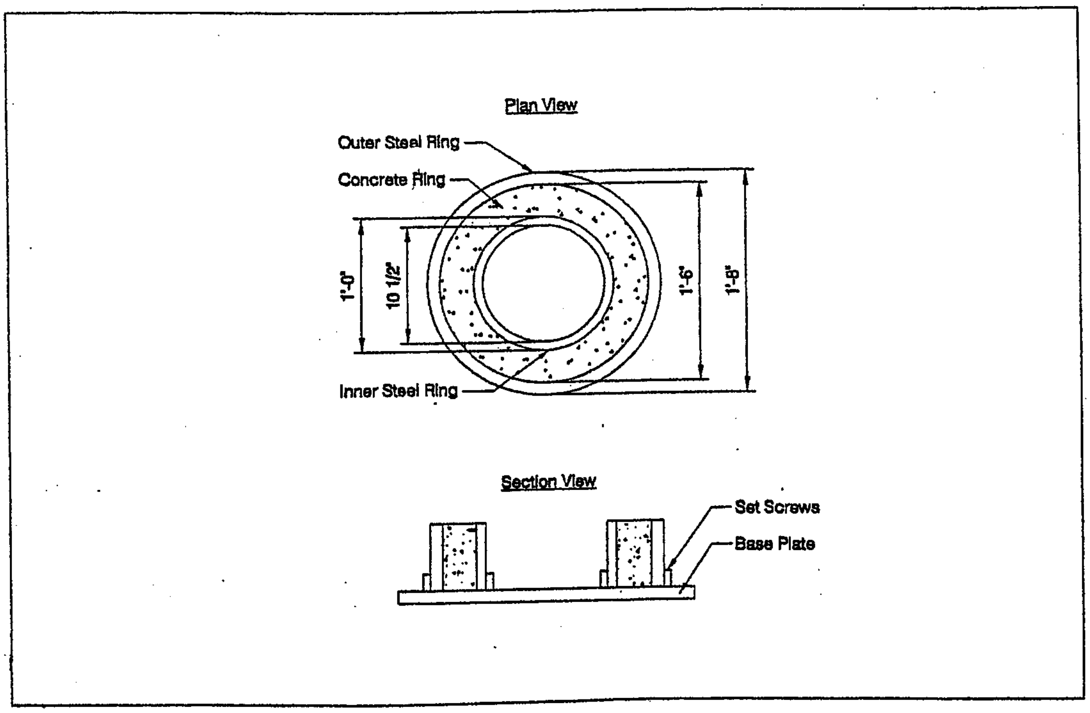

Figure 2. Modified double ring test.

This simulates the use of stay in place forms, which increase the chance of cracking by allowing drying from only one surface. The double ring test allows moisture to escape from only the sides because the inner and outer rings block water migration.

Shrinkage is effectively controlled when Type k cement is used if it is restrained to approximately 150 psi. This requires a strain of approximately 600 micro inches. The expansion required to stress the concrete bridge deck on Rt. 101 to150 psi, assuming that it was restrained only by internal longitudinal reinforcement, is approximately 1000 micro strain. Likewise the transverse reinforcement must be strained to over 600 micro strain to cause the concrete to be stressed to 150 psi. The laboratory rings are very confined as compared to a bridge deck and only require approximately 50 micro strain to cause a concrete restraint of 150 psi.

If a concrete is not properly restrained or if adequate expansion does not occur compressive stresses are not developed. When unrestrained, Type k cement concretes exhibit higher shrinkage than conventional mixes. Restraint in a bridge deck is obtained through the relatively high level of reinforcement as well as through shear connections to the supporting girders.

Route 101 Bridges

Two geometrically identical bridge decks were cast on Rt. 101 spanning Bloody Brook in Exeter NH. Both are 87 feet long with simple supported spans and compositely designed. Each deck consists of five AASHTO Type IV precast girders located eight feet nine inches on center for an overall deck width of approximately 54 feet including guardrails. Composite action was assured by shear steel protruding from the unfinished top flanges of the girders.

Gauges and Data Acquisition System

The project involved monitoring three concrete placements, a test slab at Morse Hall and the two bridge decks. A weather station was installed on the day of the deck placement next to the eastbound lane at approximately deck level. The locations of the weather station and strain gauges are shown in figure 3. The weather station was positioned in an unobstructed area at the same elevation as the bridge decks allowing for accurate measurement of weather conditions on the deck surface.

Data were collected with Campbell Scientific data loggers. The weather station was controlled with a CR-21X, and the decks were monitored using the CR-10 series data loggers. Deck data were recorded using CR-10 channel loggers with five-quarter bridge strain gauges and one thermocouple. Data from the systems were manually downloaded approximately every two weeks to a portable computer using Campbell PC208e software. During the first week data were sampled every five minutes, after the first week the interval was changed to15 minutes.

On both decks three strain gauges and a thermocouple were placed mid-span at location A as shown in figure 3. Two of the three gauges from location A were placed over the girder in the transverse and longitudinal directions. The third gage was placed transversely between the girders, and the thermocouple was located in the center of the slab over the girder. Instrumentation at location B consisted of 2 strain gauges in the longitudinal direction. One is over the middle girder and the other is between the center and adjacent girder. The placement of instrumentation was identical for both bridge decks. A section view of the bridge deck taken in the transverse direction as presented in figure 4 shows instrument placement, reinforcement spacing, locations A and B, and other relevant deck dimensions.

The Morse hall test slab had the strain gauges bonded to the reinforcement, which was identical to the steel in the actual bridge decks. A flat spot approximately ½" wide and 2 inches long was ground into the epoxy coated bars. Strain gauges were then bonded to the reinforcement. All strain gauges were functioning correctly when the placement began, however two were lost during placement of the concrete. The other two gauges short-circuited shortly after the placement.

Having only two days until the deck placement, another plan was devised to instrument the reinforcement to prevent strain gauge failure. Necessary development length was calculated and three-foot epoxy coated reinforcing bars of the appropriate diameter were cut and mounted with strain gauges in the laboratory. The deck gauges

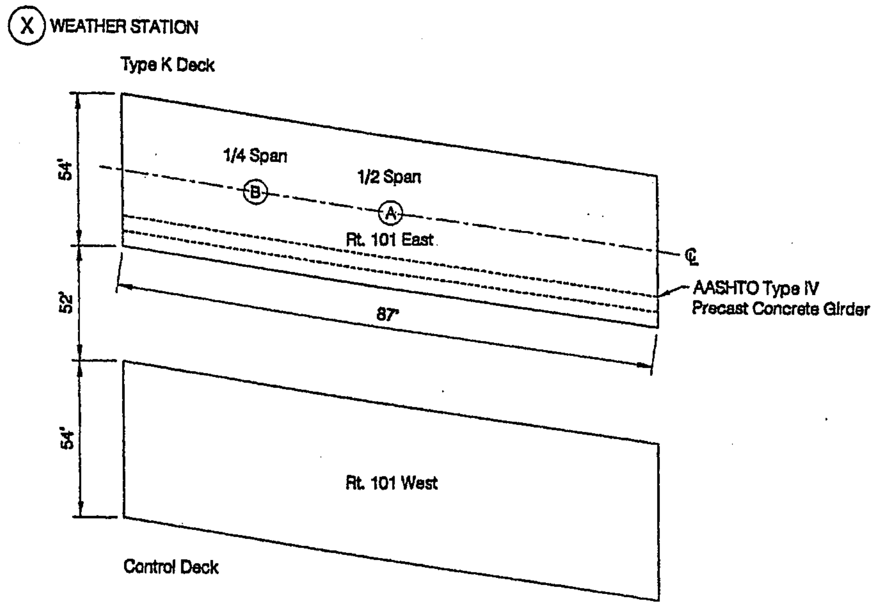

Figure 3. Plan view of route 101 test site.

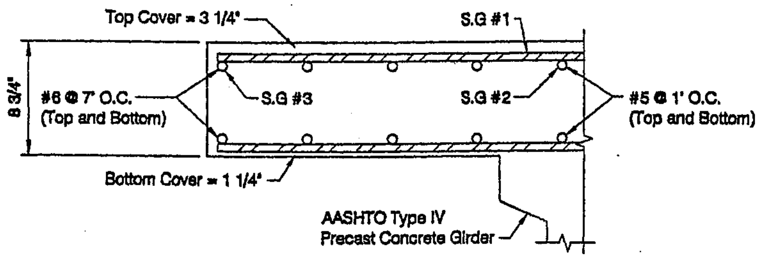

Figure 4. Transverse section view of deck reinforcement and sensor locations

were bonded to a flat spot on the three-foot sections. Gauges were also lost in the Type k cement bridge deck so an improved mounting procedure was developed resulting in a 100% success rate with the gauges in the control deck. Extensive testing resulted in a mounting that involved grinding a section all the way around the bar approximately six inches in length. Gauges were then epoxy bonded and allowed to cure for one day before removing the backing tape. When the wires were soldered to the gauges they were set up on rubber blocks to avoid the potential of short-circuiting on the reinforcement. Before soldering the wires they were separated and covered by M-COAT B nitrile rubber coating to allow bonding with the M-COAT A polyurethane coating and the M-COAT F strain gage protective system. Once the M-COAT B cured for twenty-four hours the assembly was covered with a thick coat of M-COAT A and allowed to cure for another twenty-four hours. The final step involved wrapping the mounting with the M-COAT F kit. The M-COAT F kit consists of a soft sticky pad, a rubber mat and a sticky aluminum tape. The soft sticky pad is tightly wrapped around the assembly, covered with the rubber mat and wrapped in the foil tape. The M-COAT F kit was chosen for its shock absorption abilities, ease of application and its uniformity. Initially M-COAT J was chosen rather than M-COAT F however application was messy, potentially toxic, more expensive and difficult to evenly spread. All M-COAT products are intended for strain gage waterproofing and are available from Micro-Measurements.

Chapter IV

LABORATORY TESTING PROCEDURES

Mix Design

All mixes made in the laboratory were intended to simulate field conditions so as to achieve a target slump required for adequate placement and consolidation. All aggregates used were obtained from City Concrete in Exeter NH and met NHDOT specifications. The ¾"crushed coarse aggregate and fine aggregate had absorptions of 1.1% and 1% respectively. Both aggregates were glacial in origin and consisted of rounded as well as angular particles.

The Type k mixes were identical to the actual field mix used on the Bloody Brook project as presented in table 1. All other mixes were based on the Control mix design used by the contractor on the Bloody Brook Project, without the 50% GGBFS substitution. Mixes prepared in the laboratory that did not contain Type k cement are as presented in table 2

Table 1. Laboratory type k mix design.

Component	1 yd^3
Type k	715 lb
City #67	1807 lb
City Sand	827 lb
Water	42.9 lb
Water to Cement ratio	0.5
Hycol®	3 oz/100
Darex AE®	0.5 oz/100
Slump initial	8 in
Slump 20 min	5 in
Air Design	6 %
Air Actual	5 %

Table 2. Laboratory concrete mix design

Component	1 yd^3
Type II	650 lb
City Coarse # 67	1175 lb
City Sand	1192 lb
Water	250 lb
Water to Cement ratio	0.39
Air	7+- 2%
Daracem 100®	30 oz/100
Darex II®	3 oz/100

Since the basic mix design included water reducers and air entrainment, mix water had to be added or removed to achieve the required slump depending on the ingredients in each mix design. Some mixes that had silica fume without a water reducer required more water to achieve the desired slump values. While others that had high doses of water reducers without any other additions required less water and the air entraining admixture had to be adjusted to achieve 5 to 9% air. These mixes were intended to simulate the concrete, as it would be placed in the field.

Mineral and Chemical Additives

The Silica Fume used was Force 10000™ manufactured by W.R. Grace. Addition rates of 0%, 5%, and 10% were evaluated in conjunction with other mix variables. The Silica Fume was added to the mixer after the aggregates and cement but before the mix water. Blue Circle Cement produced the GGBFS used in the mix designs. Substitution percentages of 0% and 25% were evaluated however the 50% substitution like the control mix used in the control deck over Bloody Brook were not evaluated in the laboratory. The GGBFS was combined with the cement before mixing with the aggregates. Two different air-entraining admixtures and two water reducing agents were used. Type k mixes used Darex AE at a dosage rate of 0.5 oz/100 lb cement and WRDA w/HYCOL at a dosage rate of 3 oz/100 lb cement. The other mixes used Darex II at dosage rates of 3 oz/100 lb cement and Daracem 100 at a dosage rate of 30 oz/100 lb cement. All air entrainers and water reducing agents were obtained from the storage tanks at City Concrete in Exeter NH.

Eclipse™, a W.R. Grace shrinkage reducing agent, was used at a medium dose of 1.5 gal/yd^3 of concrete. Typically dosage rates vary from 2% to 4% by weight of cement or 1.5-3 gal/yd^3. (10)

Mixing Procedure

A 1.5 cubic foot Lancaster counter current pan batch mixer type 30DF model number 192 was used for all mixing. Coarse and fine aggregates were blended together with a 10% solution of the air entrainment admixture and mixed for two minutes. Next the cement and mineral admixtures were blended in with the aggregates and mixed for two minutes. Approximately 50% of the mix water was then added and mixed for 2 minutes. The liquid admixtures were then added to the mix followed by the remaining mix water as necessary to achieve the required slump. If a visual inspection indicated proximity to the target slump all water addition was stopped and the mixer was left on for 3 minutes. If the target slump was not achieved additional mix water was added and the amount recorded.

Fresh Concrete Testing

All mixes were tested for slump and air content. The air contents were determined as per the specifications of ASTM C231-91b, Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method. The slump was measured per ASTM C143-90a, Standard Test Method for Slump of Hydraulic Cement Concrete. All test specimens were made per ASTM C192 M-95. The molds were compacted and then placed on a vibration table for approximately ten seconds. All mixes had a minimum of three 4" by

8" cylinders and two 3 1/2" by 10" shrinkage prisms cast. Nine mixes were placed into the double shrinkage rings as presented in table 3.

Shrinkage Testing and Curing

All free shrinkage beams were measured using a dilatometer accurate to 0.0001 inches. Measurement commenced upon demolding the beams at approximately 24 hours after mixing and continued on a daily basis for three to four weeks, and periodically thereafter. The strain in the inner and outer rings of the double ring molds was recorded every 15 minutes until strain values leveled out. Free shrinkage beams were three inches square with a gauge length of 10 inches. The beams were compared to a standard monel calibration rod.

All beams and rings except those with Type k cement were cured for a period of 3 days as specified by the NHDOT at the time of casting. These specifications have since been changed, to require a 7 day wet cure for all bridge deck concretes. At the time of casting the Type k mixes were the only mixes that had a seven-day wet cure specified. Some mixes were tested with different curing conditions. For these mixes two prisms were cured normally while a third was removed from the curing room at 24 hours.

Table 3. Mixes tested in double ring molds.

Mix	SRA[a], gal/yd^3		Silica fume, %			Cement Type and level of alkali			GGBFS, % substitution	
	0	1.5	0	5	10	k	Low	High	0	25
A	X		X			X			X	
C	X			X		X			X	
D	X				X	X			X	
L	X		X					X	X	
N	X		X					X		X
Extra	X		X					X		2X
O	X			X				X	X	
P	X			X				X	X	
S	X				X			X		X

[a] SRA = Shrinkage Reducing Admixture

The beams that correspond to the reduced curing all end with the number 3 and are presented in table 4.

Table 4. Mixes tested with a 24 hour cure.

Mix	SRA[a] gal/yd^3		Silica fume, % addition			Cement and level of alkali			GGBFS % substitution	
	0	1.5	0	5	10	K	Low	High	0	25
A3	X		X			X			X	
B3	X			X		X			X	
C3	X			X		X			X	
D3	X				X	X			X	
E3		X	X				X		X	
F3	X		X				X		X	
R3		X			X			X	X	
S3	X				X			X		X

[a] SRA = Shrinkage Reducing Admixture

CHAPTER V

RESULTS AND DISCUSSION

Expansive Concretes

Laboratory data

Four concrete mix designs were used for Type k evaluation. The laboratory mixes showing the different levels of evaluation are presented in table 5. Laboratory shrinkage testing indicated that Type k concretes offset drying shrinkage by an initial expansion reaction only when allowed to adequately cure as shown by figure 5. Samples A3, C3 and D3 were cured for only 24 hours as compared to 7 days for A1, C1 and D1.

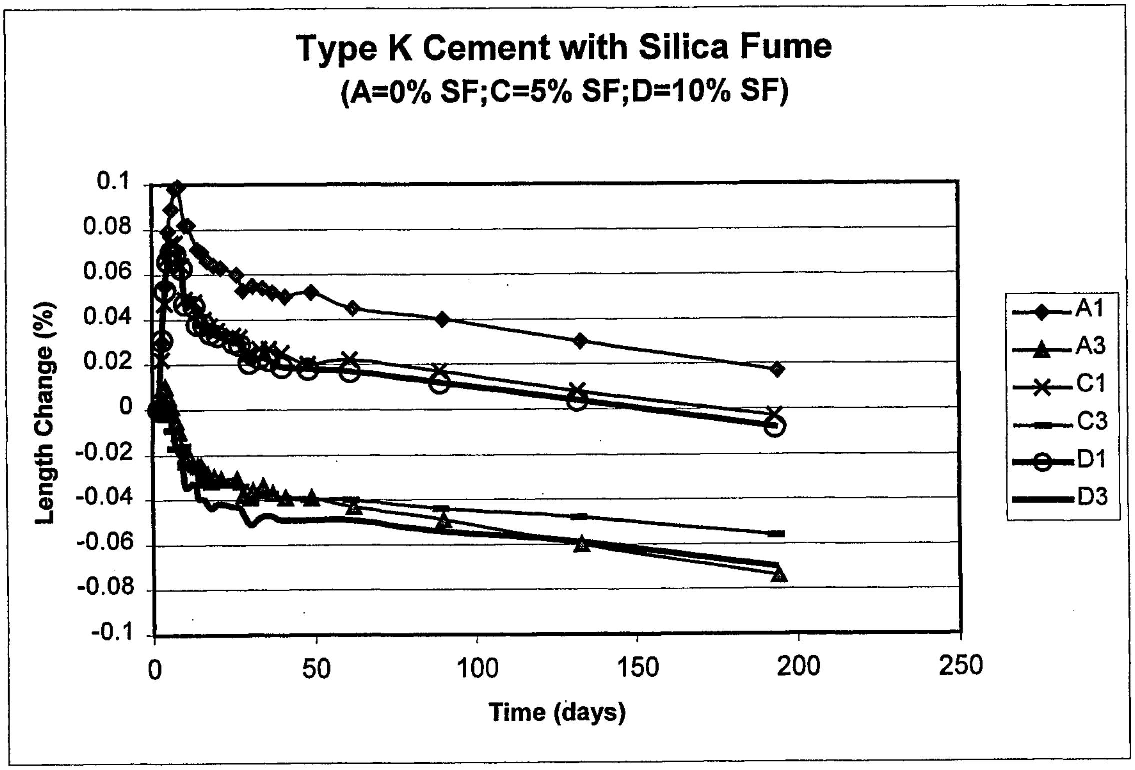

Figure 5. Length change versus time for Type k concretes with silica fume additions of 0%, 5% and 10%.

Table 5. Mixes evaluated.

Mix	20 day % shrinkage	180 day% shrinkage	SRA, gal/yd^3		Silica fume, % addition			Cement Type and Level of alkali			GGBFS % substitution		Extended range water reducer, oz/100 lb.		
								K	II						
			0	1.5	0	5	10		Low	High	0	25	0	15	30
R	-.013	.057		X			X			X	X		X		
E	.015	.046		X	X				X		X		X		
T	.012	.042		X	X					X	X		X		
U	.015	.047		X	X					X		X		X	
A	-.065	-.001	X		X			X			X		X		
C	-.035	.018	X			X		X			X		X		
B	-.014	.034	X			X		X			X			X	
D	-.031	.010	X				X	X			X		X		
F	.031	.067	X		X				X		X		X		
G	.043	.083	X		X				X		X				X
H	.035	.066	X		X				X			X			X
I	.037	.085	X			X			X		X		X		
J	.040	.070	X			X			X		X			X	
K	.046	.075	X				X		X			X		X	
L	.042	.095	X		X					X	X			X	
M	.023	.067	X		X					X	X		X		
N	.039	.089	X		X					X		X		X	
O	.018	.067	X			X				X	X		X		
P	.052	.110	X			X				X	X				X
Q	.043	.094	X				X			X		X		X	
S	.033	.082	X				X			X		X	X		

Notes on Mixes:

1) The Type K mixes as placed at Bloody Brook with 3 oz/100 lb of a WRDA with Hycol
2) All other mixes were based on the Bloody Brook control mix
3) SRA was Eclipse manufactured by WR Grace
4) Alkalinity: Low <0.6%, High > 1.0%
5) GGBFS NewCem manufactured by Blue Circle Cement
6) Extended Range Water Reducer was Daracem 100 manufactured by WR Grace.

Curing for only 24 hours does not allow for the development of full expansion when Type k cement is used. This results in higher than average ultimate shrinkage values. Mix A, identical to the Type k bridge deck mixture placed on Rt. 101 over Bloody Brook, exhibited the highest initial expansion of slightly more than 0.10%. Mix A also exhibited the highest shrinkage, approximately 0.08%, of the Type k mixes tested. Three levels of silica fume addition were used, 0%, 5%, and 10%. Curves C and D show that doubling the silica fume from 5% to 10% did not significantly change the long-term shrinkage characteristics but decreased the expansion by 0.01%. The silica fume additions contributed to higher short term drying shrinkage, particularly in the 1 day cured samples as evidenced by curves C3 and D3.

Figure 6 shows the 5% silica fume mix compared to the same mix with a water reducer. The Type k cement mix with silica fume and a water reducer decreased expansion by 0.015% and increased subsequent shrinkage by 0.01%. Mixes B and C had the same water cement ratio and air content. Comparing curves B and C it becomes clear that mix B has a higher rate of shrinkage in the early stages as well as the long term, apparently due to the water reducer.

The expansion and shrinkage of Type k concretes are affected by both silica fume and water reducing agents. With a similar slump, air content, paste volume and water cement ratio both silica fume and the water reducer decrease expansion and potentially increase shrinkage. The expansion and contraction data from the laboratory mixes matches well with the published values from Blue Circle Cement and Williams Brothers. (11)

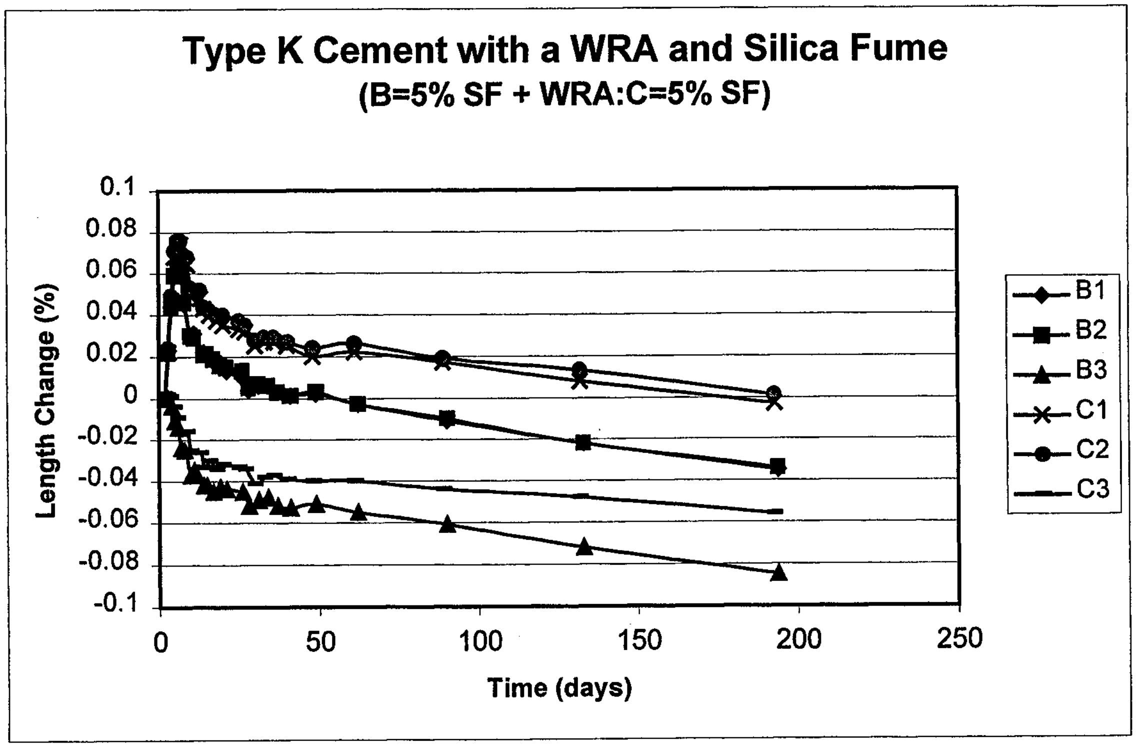

Figure 6. Length change versus time for a Type k concrete and 5% silica fume mix with and without a water reducer.

Type k cement and silica fume increase cohesion and limit bleeding, making Type k concretes containing silica fume particularly susceptible to plastic shrinkage cracking. (12) Silica fume also increases the rate of autogeneous shrinkage, or self-desiccation. This increased risk of plastic shrinkage cracking is partially compensated for by relatively high water contents and the strict placement and curing guidelines generally specified for Type k concretes.

In this work and in the literature, silica fume has been shown to reduce expansion time and decrease permeability, which makes a Type k mix more effective in reducing cracking. Rapid chloride tests for silica fume mortars are much lower than those without silica fume both for Type k cements and conventional portland cements. (13) Silica fume benefits both conventional and expansive concretes. However the effects of using silica fume are more beneficial for Type k concretes since expansion time is reduced.

Field data

The first placement occurred at the University of New Hampshire on the afternoon of July 30th 1997 in the back parking lot of Morse Hall, Durham N.H. This preliminary placement was intended to evaluate the contractors proposed Type k concrete mix design and the data collection equipment. The test slab at Morse Hall was a 10' by 10' by 8 3/4" panel placed on grade adjacent to other existing 10 by 10 panels on all four sides. The test slab reinforcement, concrete cover and curing conditions were identical to those of the Bloody Brook bridge decks. The test slab was instrumented with four strain gauges bonded in place to the reinforcing bars and a thermocouple mounted in the middle of the slab. During the placement two strain gauges were broken and soon afterwards the remaining two strain gauges short-circuited. Although potential problems with the strain gauges were identified as presented in table 6, the thermocouple wiring was shown to be adequate. Unfortunately the data acquisition system was not completely functional before the Type k deck placement, two days later on August 1st 1997. These data as with the Morse Hall test mix also showed strain gauge problems as presented in table 7. The control deck was placed on September 4th 1997 and all gauges performed flawlessly as shown in table 8.

The strain gauges from the Type k deck yielded erratic data most likely due to short-circuiting. Strain gauges two and three took opposite courses as shown on figure 7. Gauge #2 rose from –6000 to approximately –500 while gauge #3 went from –500 to –6000. These inconsistent results suggest that a short circuit took place or that an error was present in the programming of the data logger. Strain change due to diurnal cycles was approximately 800 microstrain. This value is not consistent with strains expected from temperatures recorded by the thermocouple.

The deck experienced a temperature rise of approximately 30 degrees Fahrenheit during hydration as shown by figure 8. This temperature would be expected to cause an expansion of 195 microstrain in the steel. Strain was calculated assuming that the steel was unrestrained and expanded freely with the temperature rise according to $\varepsilon = \Delta T * \alpha$, where strain equals temperature change multiplied by the coefficient of thermal expansion.

Table 6. Morse Hall test slab sensors.

Sensor	Location	Status
SG#1	Mid-span, centerline, trans, top mat	Broken
SG#2	Mid-span, centerline, long, top mat	Short-circuit
SG#3	Quarter point, trans, bottom mat	Short-circuit
SG#4	Quarter point, long, bottom mat	Broken
Thermocouple	Mid-span, centerline, center slab	Working

Table 7. Type k bridge deck sensors.

Sensor	Location	Status
SG#1	Mid-Span, Centerline, Over Girder, Trans	Working
SG#2	Mid-Span, Centerline, Over Girder, Long	Short-circuit
SG#3	Mid-Span, Centerline, Between Girders, Long	Short-circuit
SG#4	Quarter Point, Long, Over Girder	Broken
SG#5	Quarter Point, Long, Between Girders	Working
Thermocouple	Mid-Span, Centerline, Center Slab	Working

Table 8. Control deck sensors.

Sensor	Location	Status
SG#1	Mid-Span, Centerline, Over Girder, Trans	Working
SG#2	Mid-Span, Centerline, Over Girder, Long	Working
SG#3	Mid-Span, Centerline, Between Girders, Long	Working
SG#4	Quarter Point, Long, Over Girder	Working
SG#5	Quarter Point, Long, Between Girders	Working
SG#6	Backup to SG#3	Working
SG#7	Backup to SG#5	Working
Thermocouple	Mid-Span, Centerline, Center Slab	Working

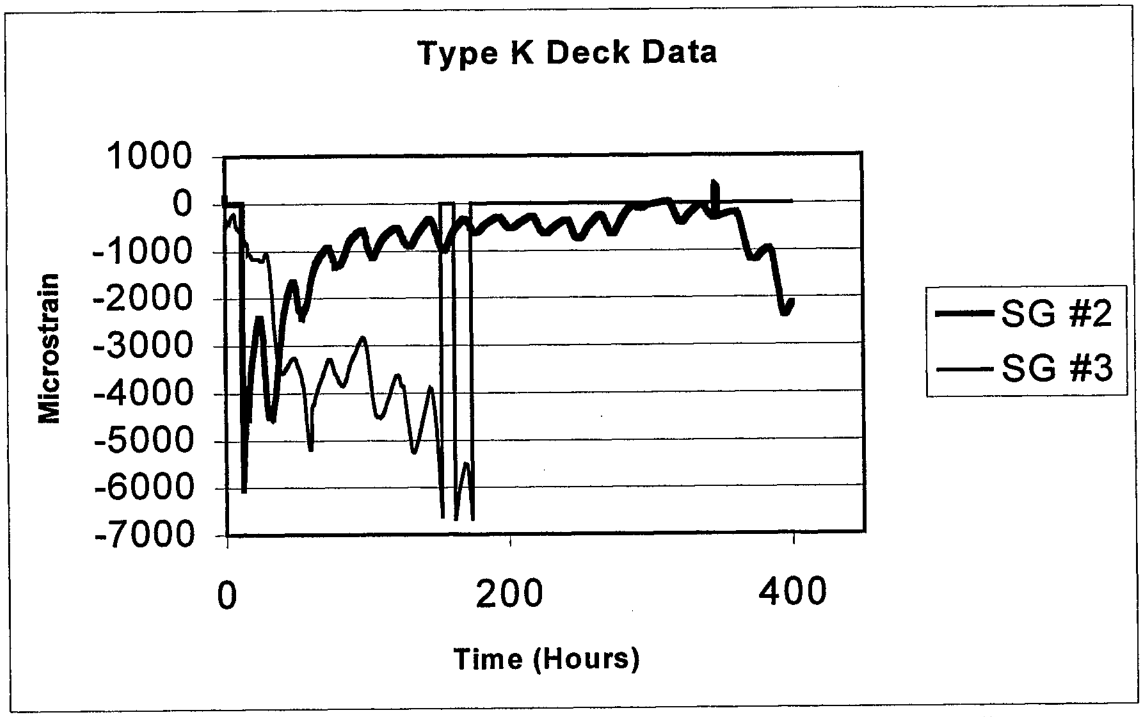

Figure 7. Micro strain versus time for Type k deck gauges #2 and #3.

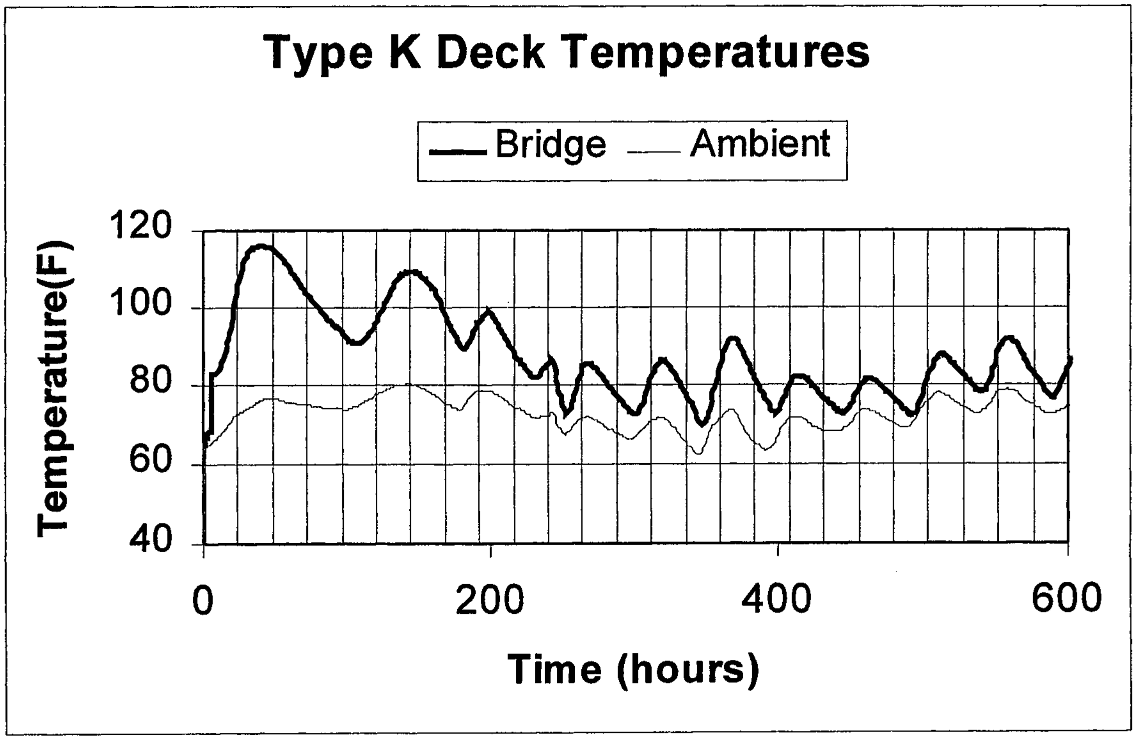

Figure 8. Temperature versus time for Type k bridge deck internal and ambient temperatures.

The darker line of figure 8 represents the thermocouple in the bridge deck beginning on the evening before the concrete placement. The sudden jump from 70 to 82 degrees corresponds to the time when the fresh concrete hit the thermocouple. The ambient temperature was measured underneath the bridge deck at the data logger, effectively reducing the affect of solar intensity.

The control deck temperature rose 15 degrees Fahrenheit, as shown by figure 9, exhibiting approximately 60 microstrain of expansion in the longitudinal reinforcement. This was significantly less than the thermal strain exhibited by the longitudinal steel in the Type k deck. The steel strain in the control deck caused by a 15 degree rise in temperature was calculated to be 97 microstrain. Strain values of 60 microstrain for the control deck seem reasonable since the girders and reinforcing steel provide restraint. By the time hydration temperatures reached their maximum value the concrete had set up. This forced the prestressed girders to expand with the deck. These girders partially stopped the deck from expanding and created stresses, lowering the strain that would have been seen from a direct thermal expansion.

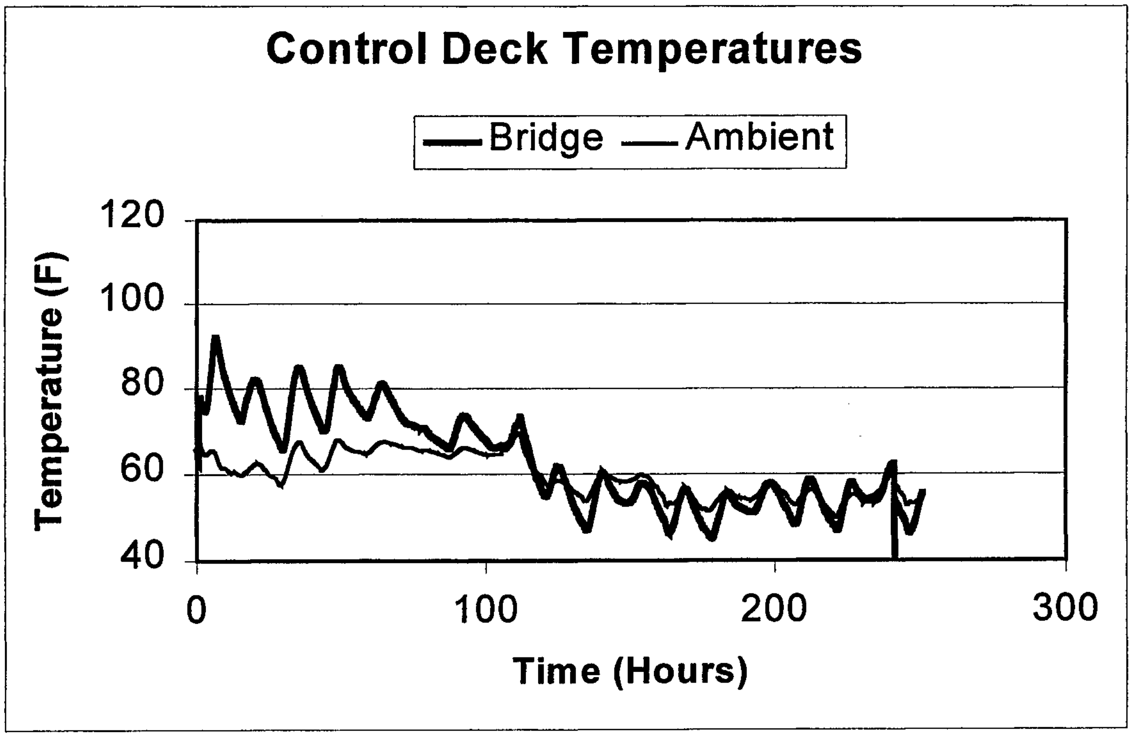

Figure 9. Temperature versus time for control deck internal and ambient temperatures.

Gauge number five yielded the most consistent readings as shown by figure 10. The measured strains were lower than expected and the diurnal cycles caused only 8 microstrain oscillations in the longitudinal reinforcement. The strains caused by the diurnal cycles of gauge number 5 were approximately ten times smaller than expected and the oscillations of gauges 2 and 3 were ten times larger than expected. The fact that these values are off by an order of magnitude for several gauges is indicative of a possible flaw in the data collection programming. The program as well as the gauge mounting was changed for the control deck placement.

Figures 11 and 12 show the effect of restraint on thermal expansion and shrinkage. Transverse shrinkage was not significantly different between the girders compared to shrinkage over the girders as shown by figure 11. However on day 14 the over girder gauge had a sudden drop in strain to the level of the mid-span gauge, indicating that differential shrinkage in the transverse direction was present due to the restraint caused by the girder and diaphragm system.

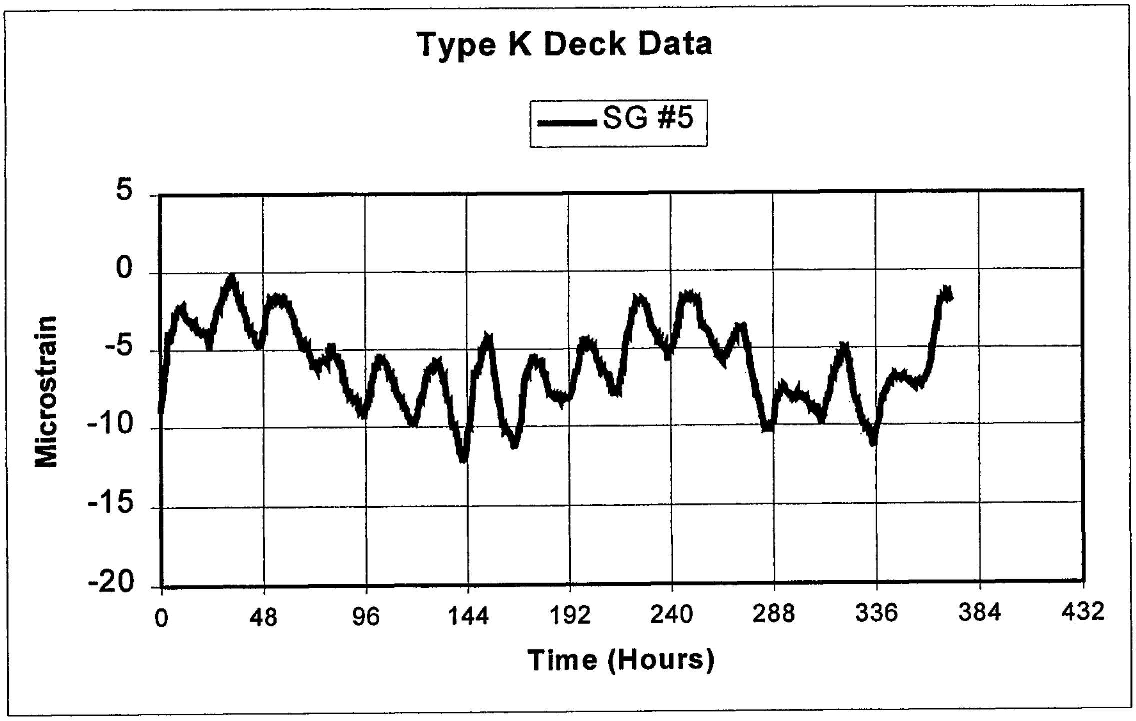

Figure 10. Micro strain versus time for Type k bridge deck strain gauge #5, longitudinal strain between girders.

Comparison of longitudinal shrinkage between and over girders is illustrated in figure 12 where the poly lines represent best-fit lines to the between and over the girder data. The over the girder gauge was more restrained from expansion during hydration and subject to slightly larger shrinkage and creep strains. The areas above the girders had compressive stresses induced during hydration as well as additional compressive stresses from creep in the pre-stressed girders. This increase in compressive stresses over the girders resulted in shrinkage strains over the girder that were 150 micro strain compared to the between the girder strains that were only 100 micro strain including the initial expansion of 25 micro strain. It is apparent from figures 11 and 12 that longitudinal strain changes are greater than transverse changes. The longitudinal direction contains less reinforcement and more girder restraint than the transverse direction. The initial expansion and the daily thermal cycles were similar for both directions with the exception of over the girders in the longitudinal direction where there was significant external restraint.

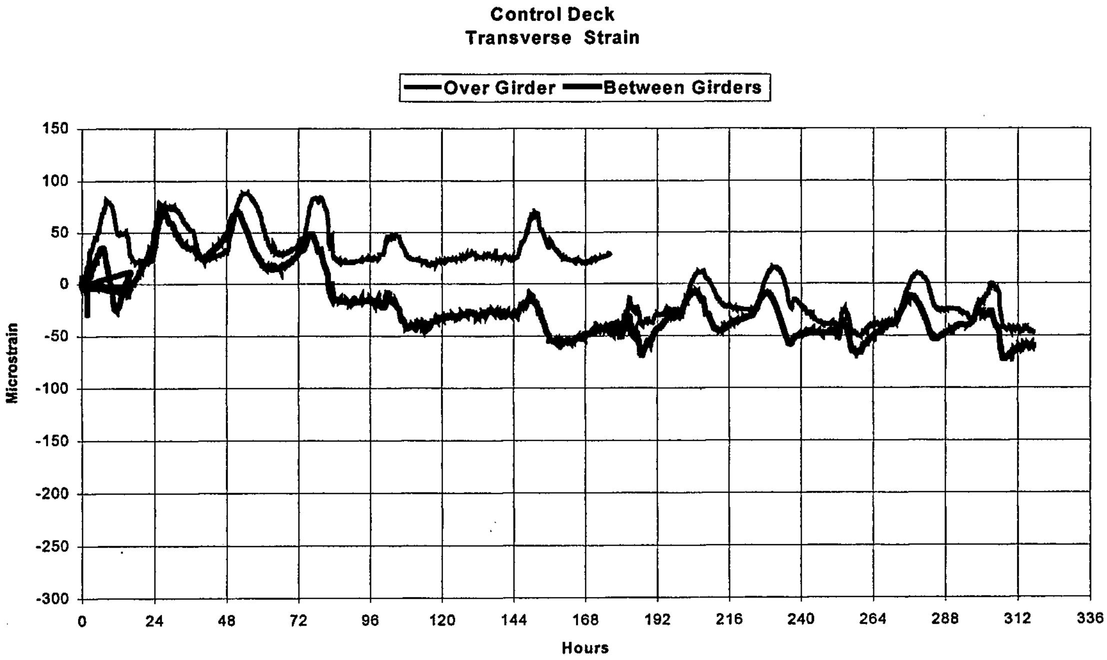

Figure 11. Micro strain versus time control deck transverse direction over the girders and between the girders.

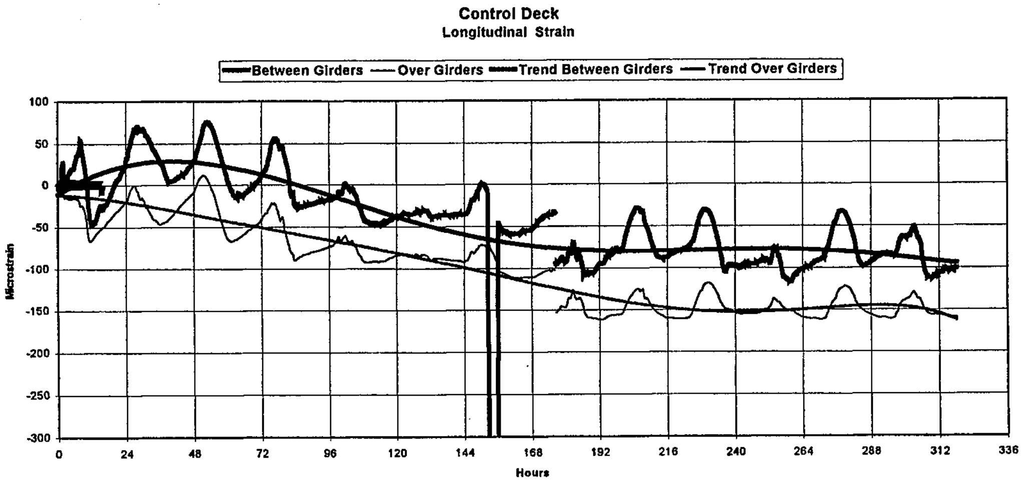

Figure 12. Micro strain versus time control deck longitudinal direction over the girders and between the girders.

In addition to monitoring reinforcing strains and environmental conditions regular inspections of the deck surfaces were conducted for cracks. Visual inspections were conducted immediately after wet curing, at one month, three months, six months and one year. Video footage of the Type k deck was also taken at one month. Soon after one

In addition to monitoring reinforcing strains and environmental conditions regular inspections of the deck surfaces were conducted for cracks. Visual inspections were conducted immediately after wet curing, at one month, three months, six months and one year. Video footage of the Type k deck was also taken at one month. Soon after one month the decks were diamond grooved in the transverse direction making crack detection more difficult. Whenever possible the decks were inspected while the surfaces were drying from a rain shower so as to accent the presence of cracking. No transverse cracks were noticed at any of the inspection intervals. Very minor random cracking was found in a small area of the Type k deck close to the guardrail where water failed to adequately drain. This was the only visible cracking. This cracking was noticed at the one-year inspection interval. No cracks were found in the control deck at any of the inspection intervals.

Mineral Admixtures

The use of GGBFS and silica fume were tested on several mixes, both by themselves and in conjunction with other admixtures.

Ground granulated blast furnace slag

GGBFS reduces the heat of hydration because less portland cement is used. GGBFS is commonly used as an additive in bridge deck concretes to reduce the risk of early thermal contraction cracking, to control ASR, to decrease permeability and in general to produce a higher quality concrete. Figures 13 and 14 show shrinkage values for mixes with GGBFS as the only test variable. Figure 12 compares two mixes with low alkali cement and Daracem 100 with and without GGBFS. The GGBFS reduced 20 day shrinkage by 0.008% and 180 day shrinkage by 0.017%. This is equivalent to 10% and

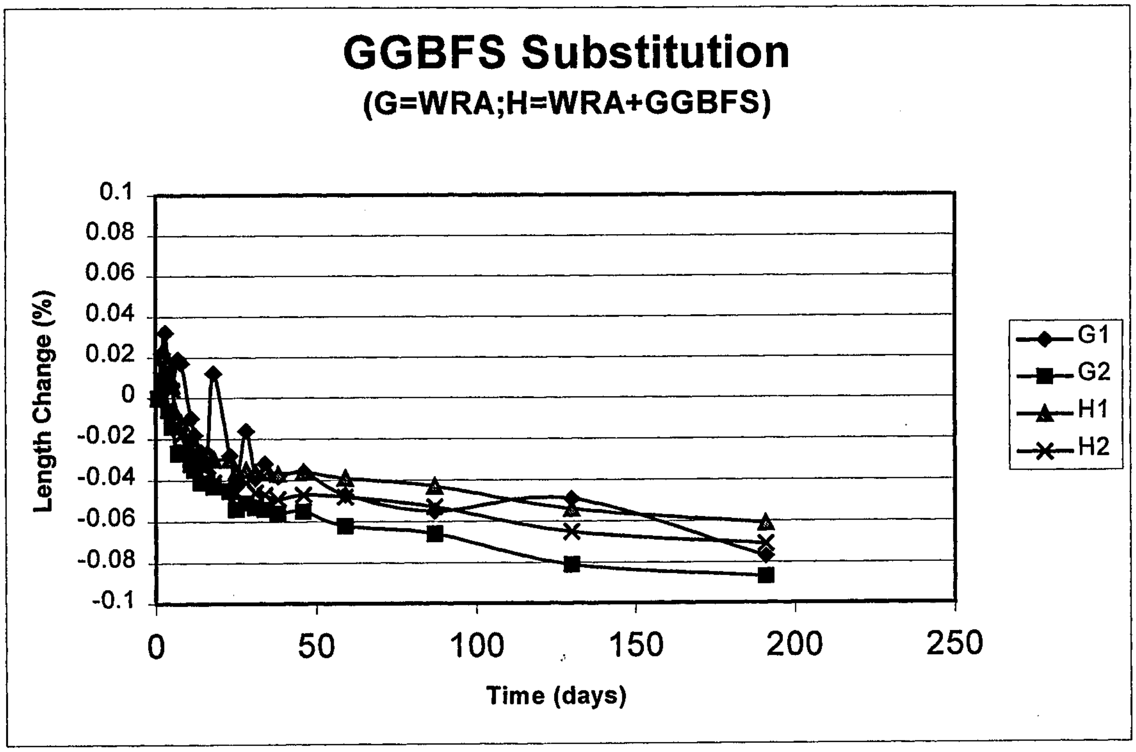

Figure 13. Length change versus time for a 25% GGBFS low alkali cement mix containing Daracem 100.

20% of the ultimate shrinkage respectively. Beam G1 had a loose measuring pin resulting in inconsistent data points, however when the pin was properly adjusted the results closely matched those of beam G2. Figure 13 compares essentially the same mixes as figure 12 however with a high alkali cement rather than a low alkali cement. The GGBFS reduced the shrinkage of the high alkali cement concrete by 0.003% at 20 days and 0.006% at 180 days. These values differ little from the control but do demonstrate the decreased effect of GGBFS in high alkali mixes as compared to low alkali mixes. In both cases the GGBFS mix showed lower shrinkage values than the control. The lower alkali cement benefited more than twice as much as the high alkali cement did from the GGBFS substitution. Comparing figures 13 and 14 with mix data it becomes evident that high alkali cement produced greater shrinkage with a lower dose of water reducer.

Figure 15 shows that the lower alkali cement reduced shrinkage by 0.019% as

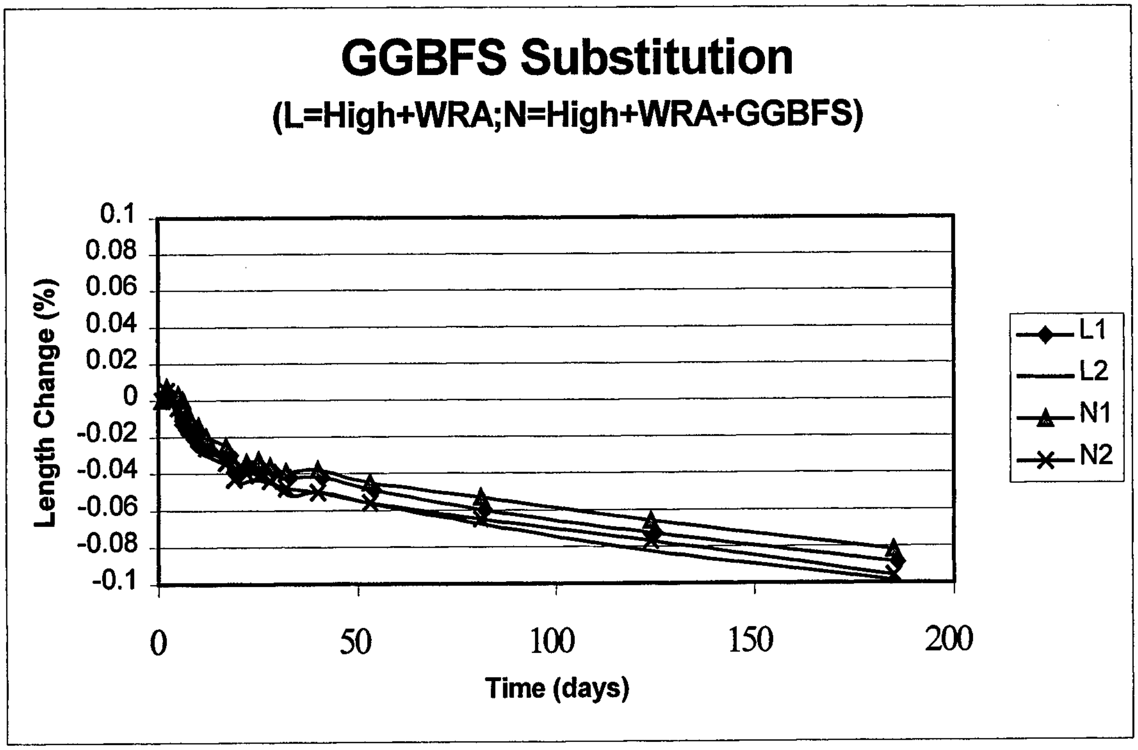

Figure 14. Length change versus time for high alkali cement mix containing Daracem 100 with and without 25% GGBFS.

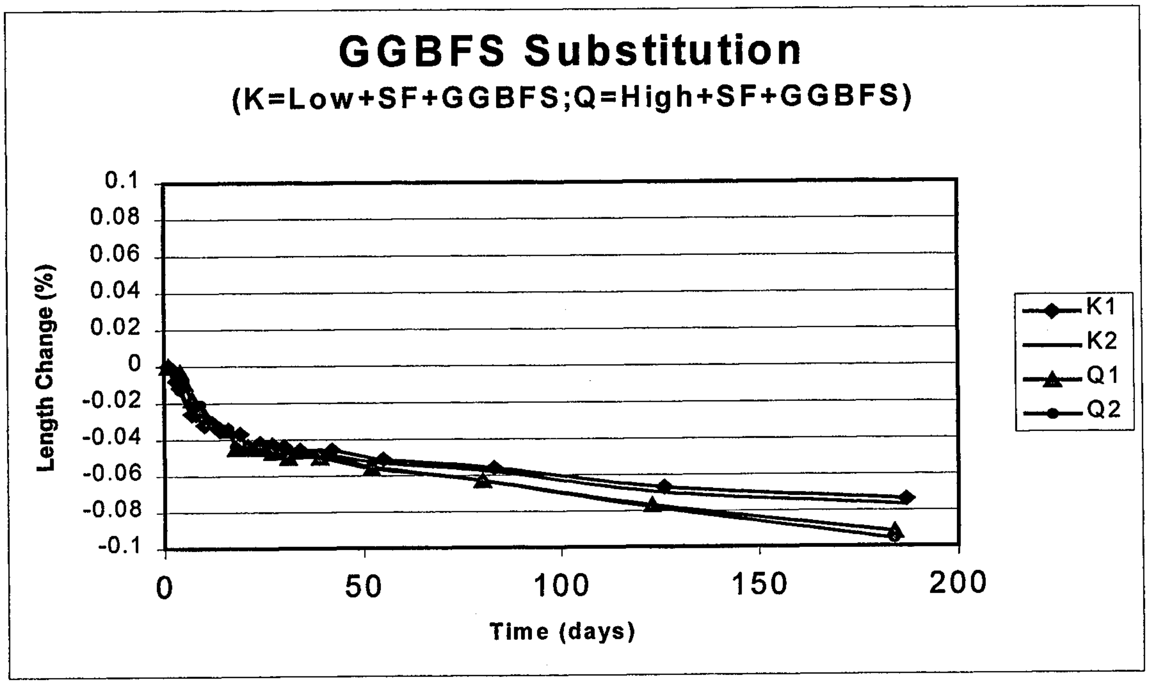

Figure 15. Length change versus time for a 5% silica fume 25% GGBFS mixture with high and low alkali cements.

compared to the high alkali cement for mixes containing GGBFS and SF.
Mix Q had a higher water cement ratio and paste volume making it mores susceptible to drying shrinkage. The previous two factors combined with a high alkali content resulted in considerably more shrinkage than the low alkali mix K. The difference in shrinkage between low and high alkali cements increases at later stages, as evidenced by the more downward slopes of the high alkali mixes.

The mixes containing GGBFS show the largest differences in drying shrinkage at later stages. The GGBFS continues to slowly produce hydration products at late stages partially offsetting the shrinkage due to loss of adsorbed water from the hcp. The early affects of GGBFS are beneficial in reducing thermal cracking however their affect on drying shrinkage is not evident until much later. This research indicates that high alkali cements reduce the effectiveness of GGBFS in reducing drying shrinkage.

Silica Fume

Silica fume in general decreases bleeding, increases the risk of plastic shrinkage cracking, reduces permeability and increases strength in most concrete. Type k concretes benefit most from silica fume, which reduces micro cracking from ettringite formation. (13) The overall expansion is reduced, however the rate at which expansion occurs is increased. Figures 16 and 17 show the effect of the addition of 5% silica fume in both high and low alkali mixes.

Figure 16 shows little or no effect on long term shrinkage however the short term shrinkage was increased by 0.005%. Mix O had silica fume and a 10% lower water cement ratio and a lower paste content. These factors contribute to the comparable shrinkage of Mix O and Mix M. Figure 17 shows that silica fume combined with low alkali cement increases 20 day shrinkage by 0.006% and 180 day shrinkage by 0.018%.

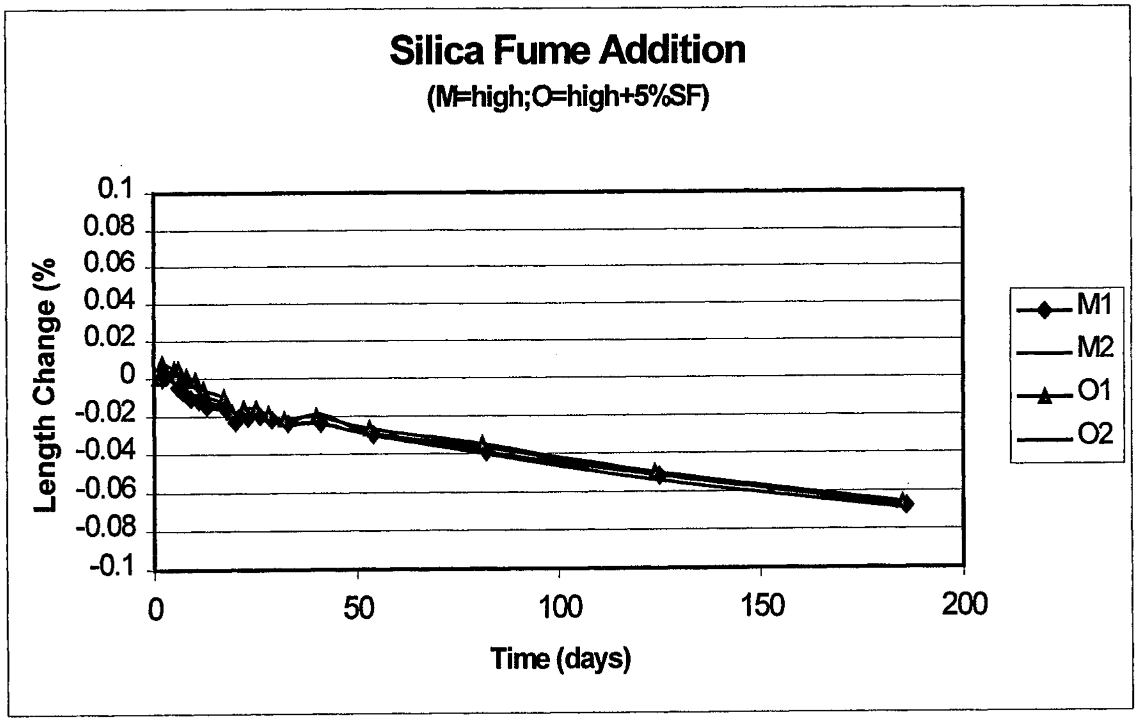

Figure 16. Length change versus time for a high alkali cement mixture with and without 5% silica fume addition.

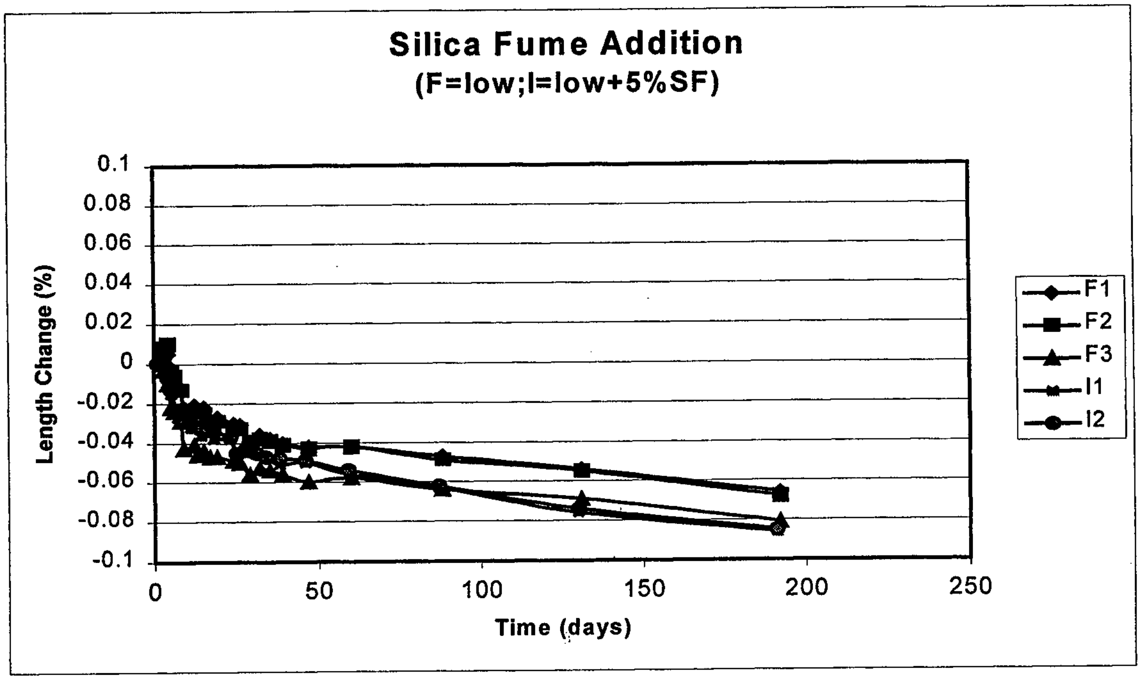

Figure 17. Length change versus time for a low alkali cement mixture with and without 5% silica fume.

The water cement ratios and paste contents of mixes F and I are identical to those of mixes M and O of Figure 16. Both figure 6 and 17 show that the silica fume mixes exhibit higher early shrinkage values. The uncured beam F3 showed approximately 0.02% of additional shrinkage at 180 days. This value is typical of the uncured specimens.

Figure 18 shows that a 10% SF addition in a GGBFS substitution mix increased the 20 day shrinkage by 0.004% and the 180 day shrinkage by 0.006%. Approximately 50% of the shrinkage occurs in the first 20 days. All mixes with SF have higher early shrinkage values therefore increasing the risk of early transverse cracking.

Figure 19 compares a mix with Eclipse with and without SF. The early shrinkage with SF was slightly increased by 0.001% but the shrinkage at 180 days was increased by 0.015%. Prism R3 followed the same curve as prisms R1 and R2 demonstrating that Eclipse significantly reduces the shrinkage differences produced by inadequate curing

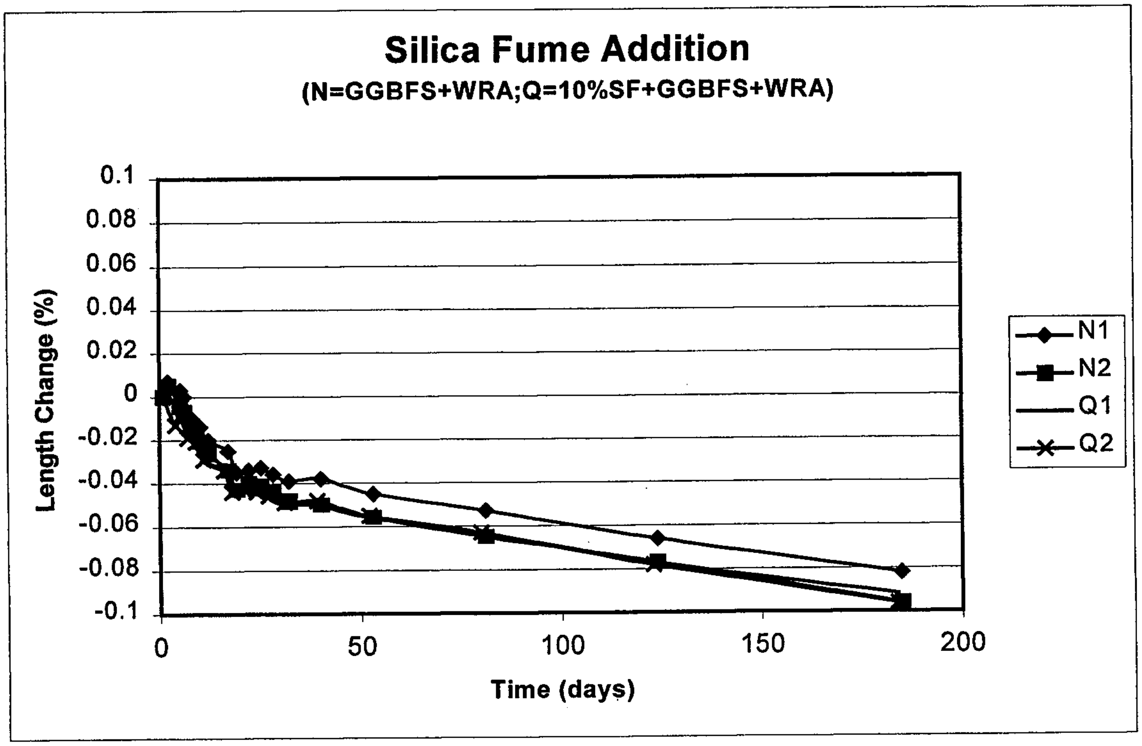

Figure 18. Length change versus time for a high alkali cement, GGBFS and Daracem 100 mixture with and without 10% silica fume.

Both figures 18 and 19 indicate higher long term shrinkage with silica fume. Drying shrinkage is most severe in the first three weeks, amounting to approximately 50% of the 180 day shrinkage.

Chemical Admixtures

The effect of water reducing agents and mid range dosages of a SRA were evaluated. The mid range dose used for all mixes was 1.5 gal/yd^3. Two water reducing agents, WRDA w/hycol and Daracem 100, both marketed by W.R. Grace were used in the study. Daracem 100 was used in a dosage of 15 or 30 oz/100 lb cement for the rest of the mixes

Water reducing admixtures

Figure 20 compares a high alkali cement mix with Daracem 100 to the same mix without a WR. The shrinkage difference between the two mixes was 0.019% at 20 days and 0.028% at 180 days. Mix L had a slightly lower water cement ratio and paste content but

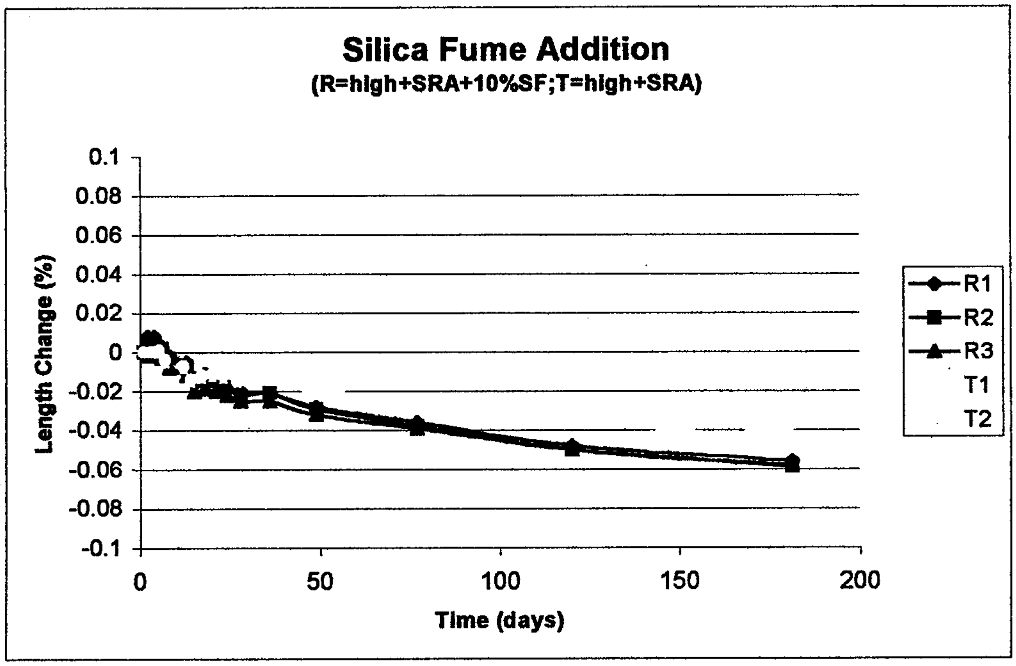

Figure 19. Length change versus time for a high alkali cement and Eclipse with and without 10% silica fume.

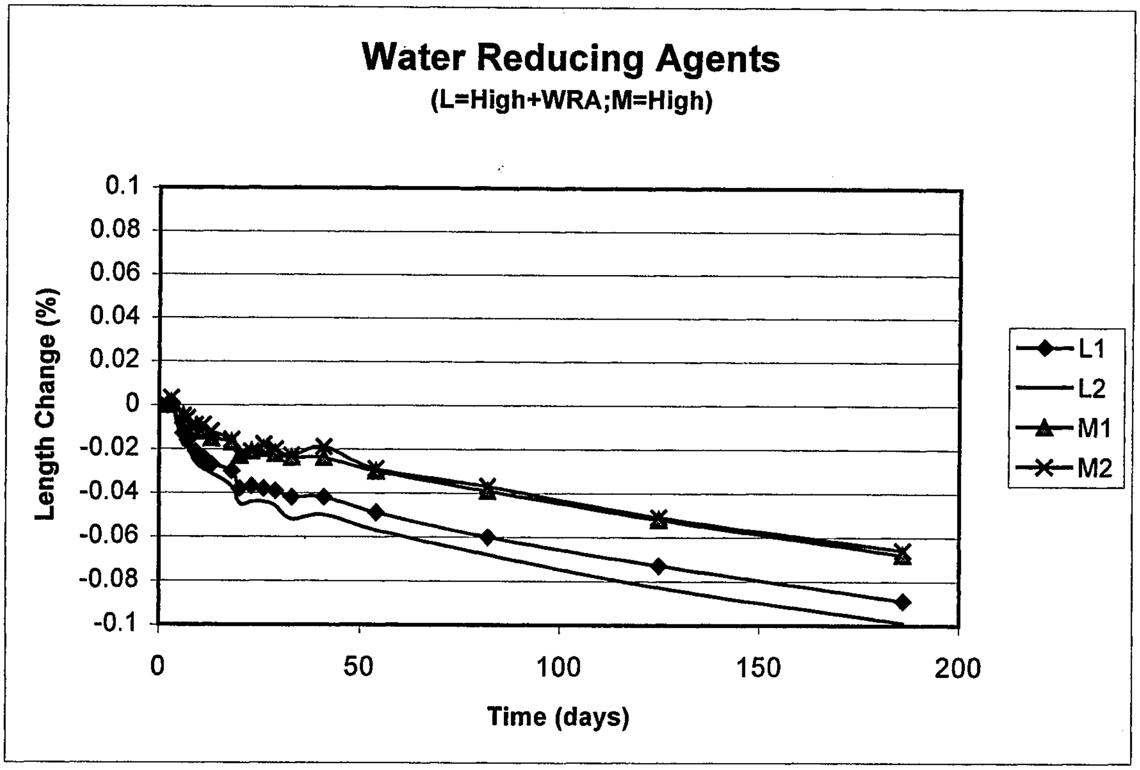

Figure 20. Length change versus time for with high alkali cement mixture with and without Daracem 100.

still exhibited more shrinkage. Figure 21 compares a high alkali cement, 10% silica fume and GGBFS mixture with and without Daracem 100. The shrinkage difference was 0.010% at 20 days and 0.012% at 180 days. Once again the mix containing Daracem 100 had a lower water cement ratio and paste content but exhibited higher shrinkage values.

Figure 22 compares a low alkali mix with a 5% silica fume addition to the same mix with Daracem 100. The shrinkage difference between the two mixes was 0.012% at 20 days and 0.016% at 180 days. Mix G contained Daracem 100 and exhibited higher shrinkage values. The measurements from beam G1 were erratic and had to be removed from consideration due to a loose stud.

Figures 23 and 24 compare mixes with doses of 30 oz/100 lb cement of Daracem 100. Figure 23 compares a low alkali mix containing a 5% silica fume addition with and

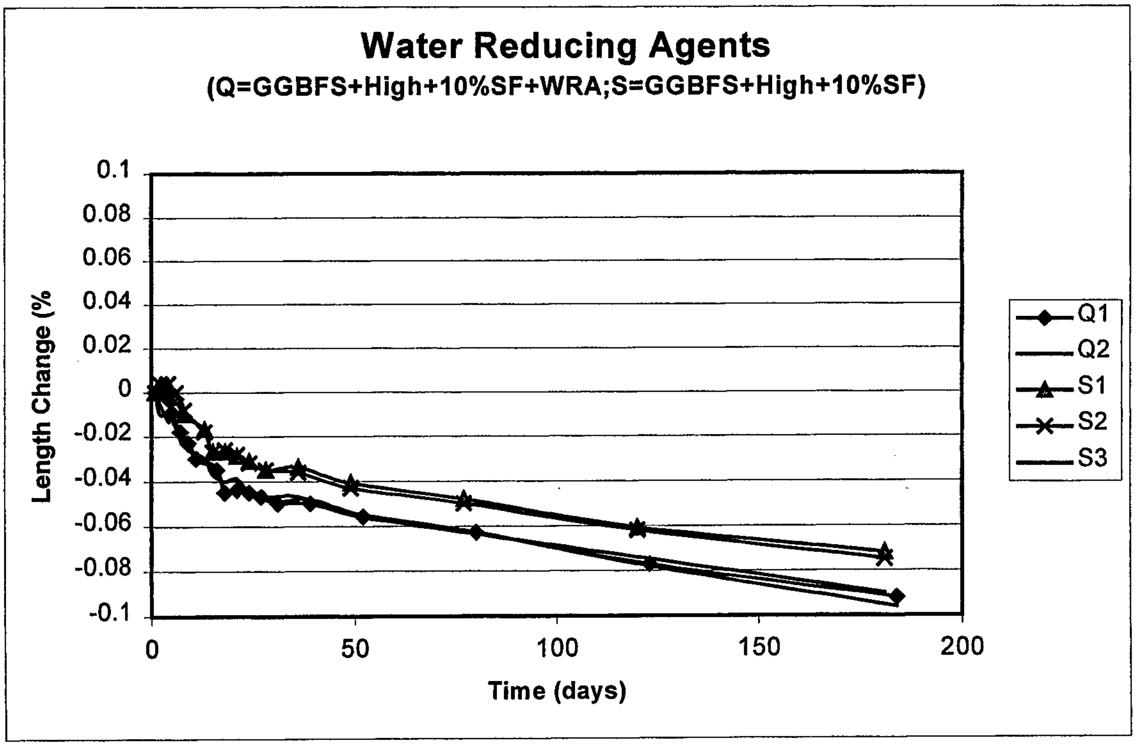

Figure 21. Length change versus time for a high alkali cement, 10% silica fume and 25% GGBFS with and without Daracem 100.

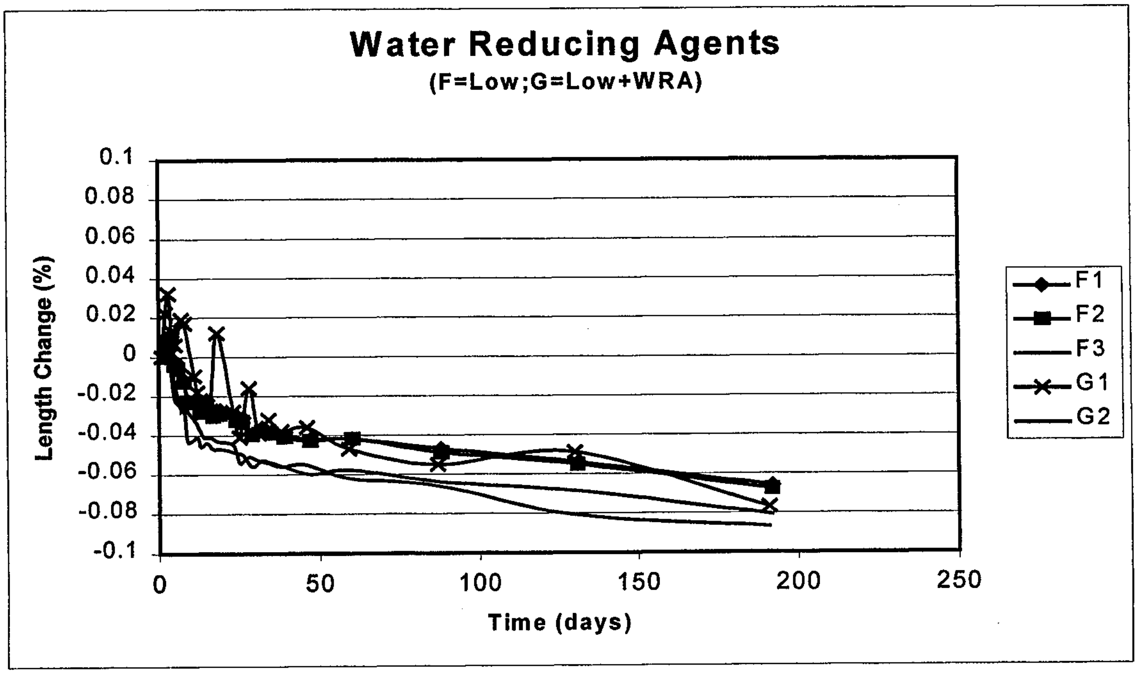

Figure 22. Length change versus time for a low alkali cement mixture with and without Daracem 100.

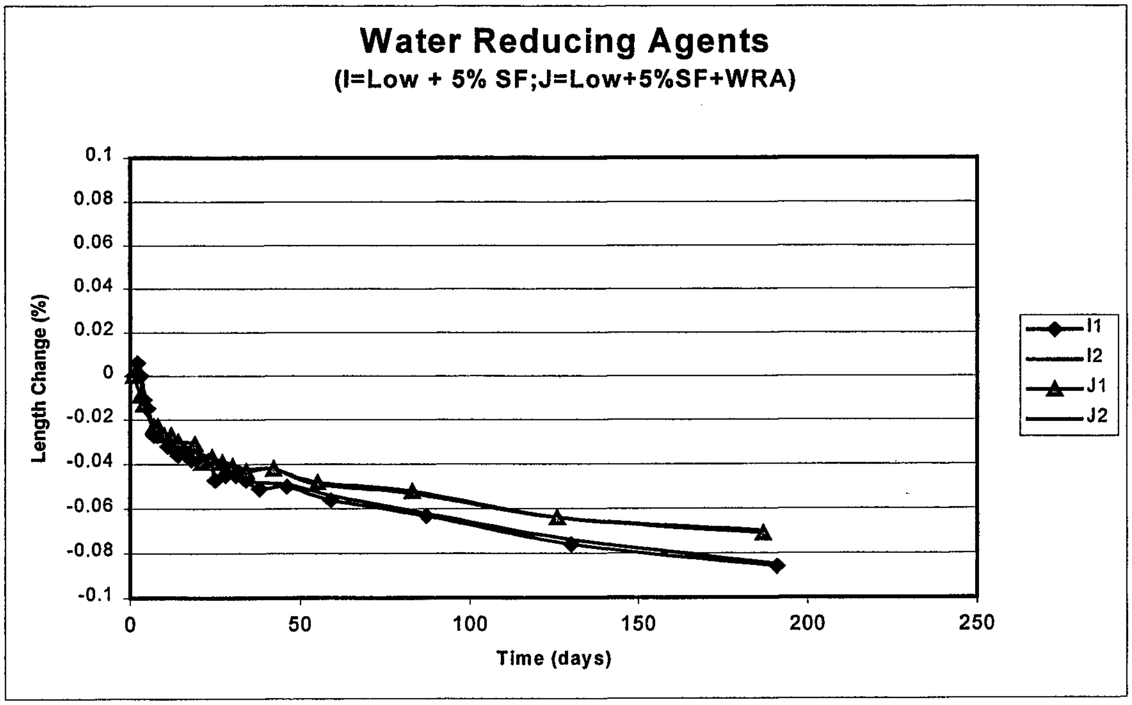

Figure 23. Length change versus time for a low alkali cement and 5% silica fume mixture with and without Daracem 100.

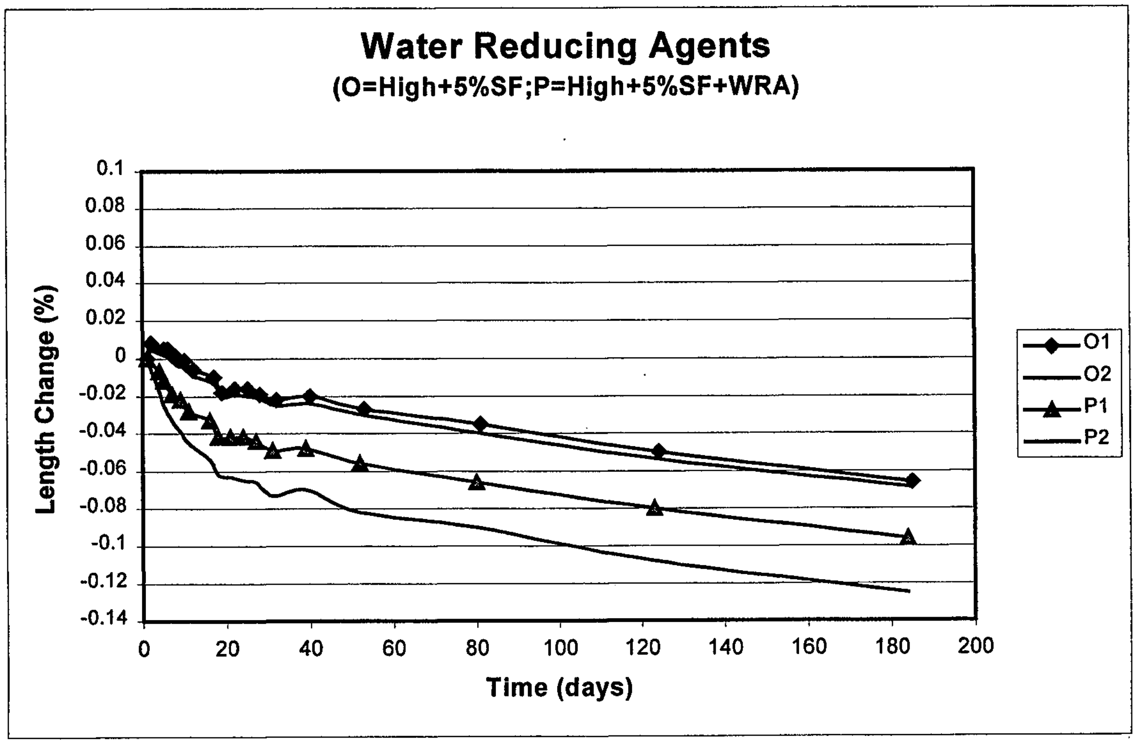

Figure 24. Length change versus time for a high alkali cement and 5% silica fume mixture with and without Daracem 100.

without a high dose of Daracem 100. The shrinkage difference between the two mixes was 0.012% at 20 days and 0.016% at 180 days. Mix J contained Daracem 100 and exhibited lower shrinkage values, differing from all other mixes tested. Figure 24 shows two mixes with high alkali cements and a 5% silica fume addition with and without a high dose of Daracem 100. The average shrinkage difference between the two mixes was 0.034% at 20 days and 0.043% at 180 days. Mix P contained Daracem 100 and exhibited the highest overall shrinkage of all mixes tested. The addition of WRAs increased shrinkage by an average of 0.020% while decreasing the water cement ratios by approximately 6%.

Shrinkage reducing admixtures

Four mixes were made using the manufacturers recommended medium dosage rate of 1.5 gal of Eclipse per cubic yard of concrete. Variables tested with Eclipse include cement alkali, silica fume, GGBFS and water reducing agents. Figure 25 compares a low alkali mix and a high alkali mix with Eclipse.

Little difference exists between these mixes suggesting that cement alkali has little affect on shrinkage when an SAR is used. Figure 26 shows a high alkali mix with Eclipse with and without 10% silica fume addition. The mix with silica fume shrank 0.015% more than the control. The SRA definitely reduced the early shrinkage of silica fume mixes; however, long term results are almost identical to mixes without the SRA. Line R3 of figure 21 follows almost exactly the R1 and R2 lines , indicating that curing has little effect on shrinkage when SRAs are used. SRAs improved workability and finishing as an added benefit. The shrinkage due to rapid self-desiccation of silica fume mixes is greatly reduced by lowering the mix water surface tension. This lowering of surface tension results in lower compressive stresses in the pore structure upon drying and therefore results in less early shrinkage.

The shrinkage response of concrete mixes with various combinations of admixtures and cement alkali content with and without the use of a SRA are shown on figures 27, 28, and 29. A medium dose of SRA, as recommended by the manufacture, was evaluated in this work. The SRA medium dose was found in general to reduce the 20 and 180 day shrinkage by approximately 40%. Data from other researchers indicates

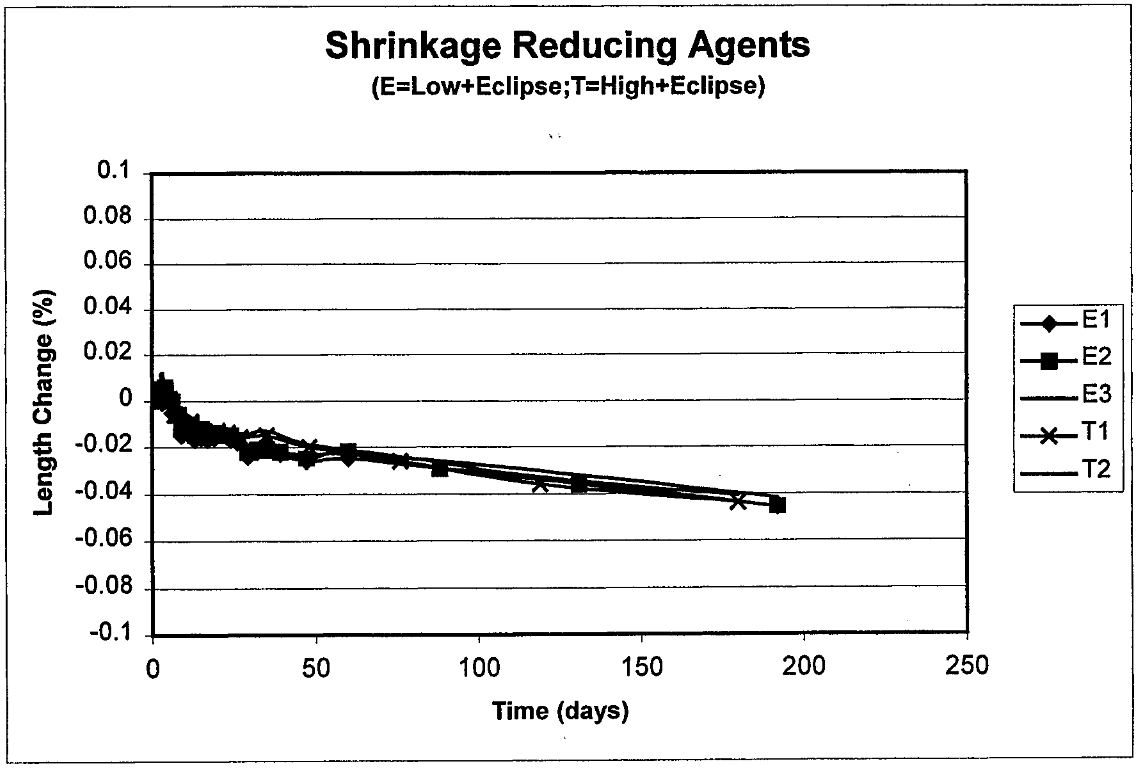

Figure 25. Length change versus time for high and low alkali cements with Eclipse.

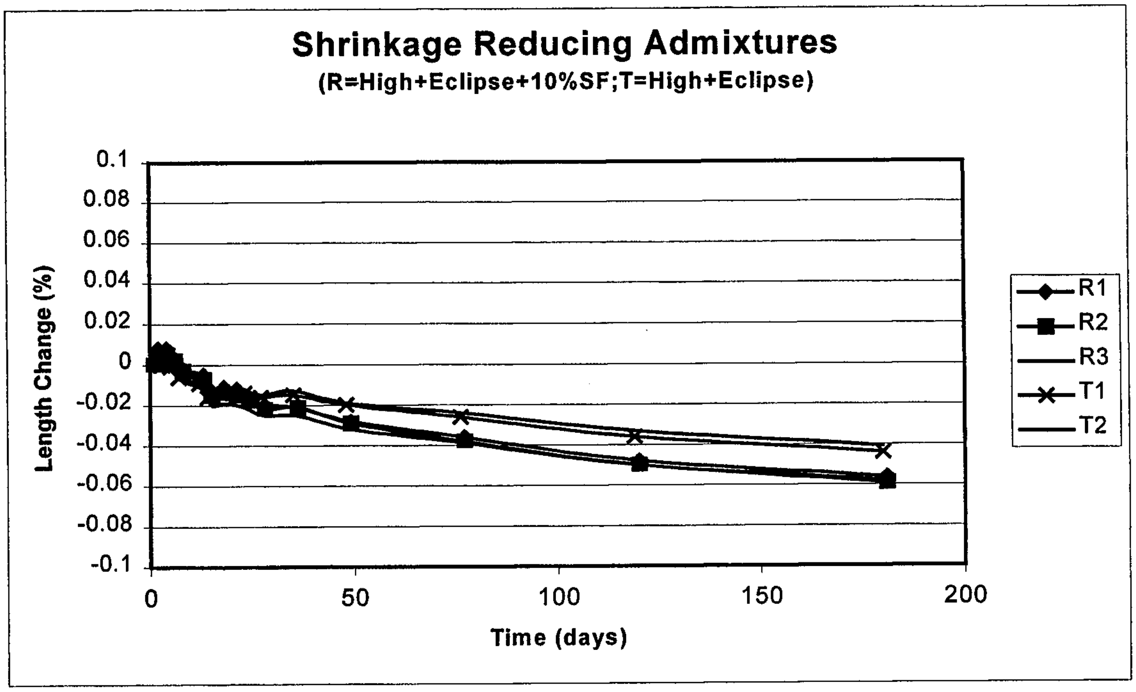

Figure 26. Length change versus time for a high alkali cement and Eclipse mixture with and without a 10% silica fume addition.

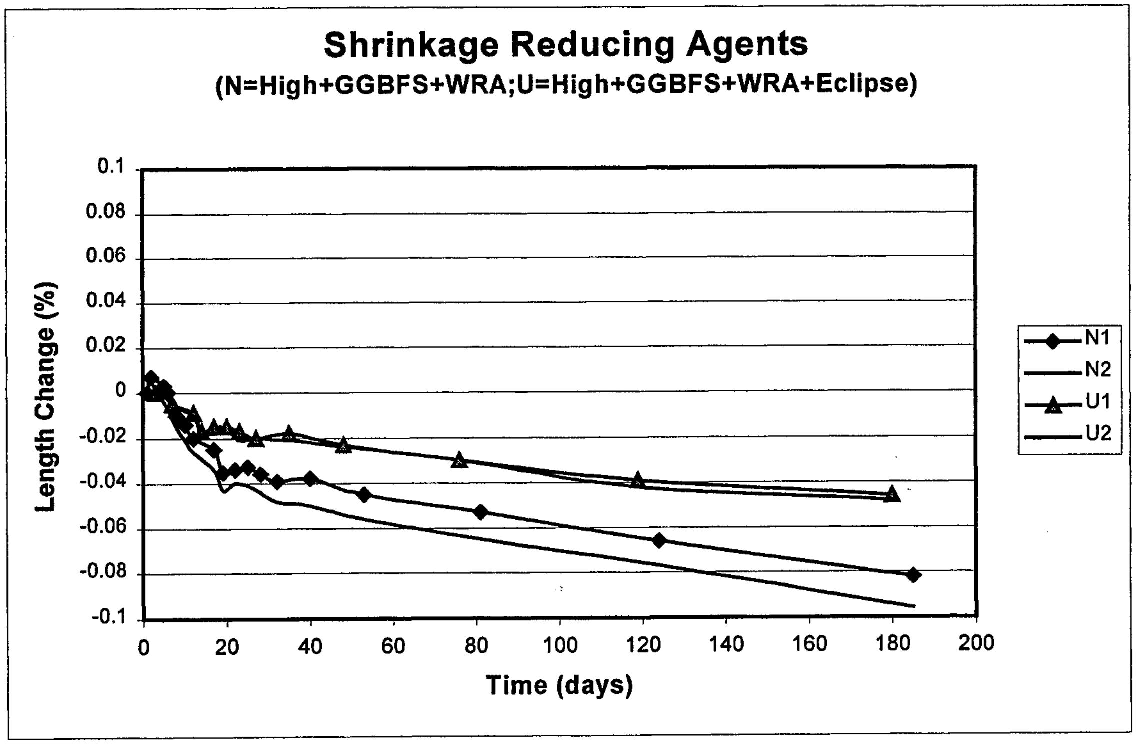

Figure 27. Length change versus time for a high alkali cement, GGBFS and Daracem 100 mixture with and without Eclipse.

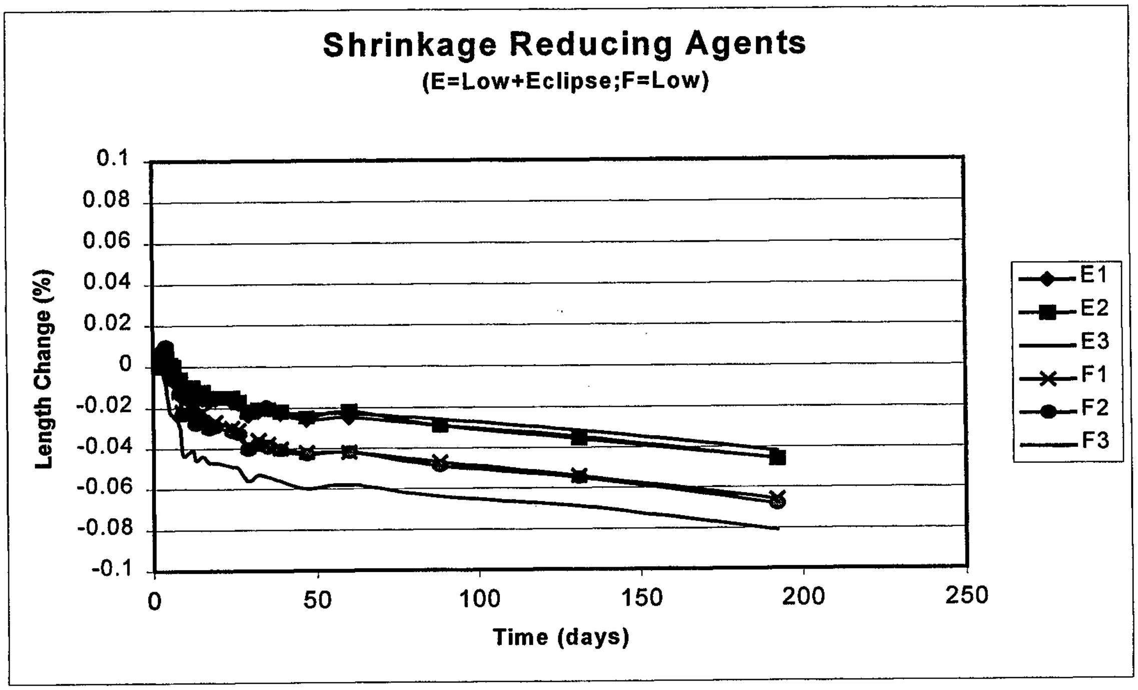

Figure 28. Length change versus time for Eclipse with low alkali cement.

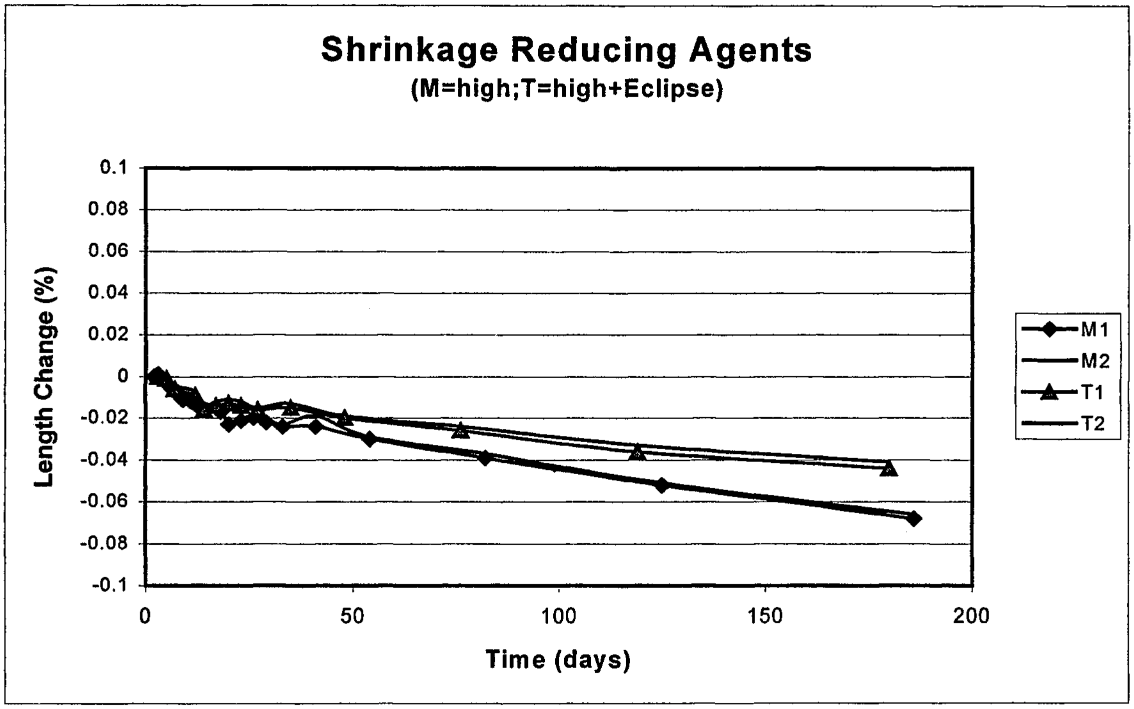

Figure 29. Length change versus time for Eclipse with high alkali cement.

that shrinkage reductions up to 60% can be obtained by increasing the dosage to 3.0 gal/yd^3. (10) The effect of alkali and the use of GGBFS on the shrinkage of SRA mixes is insignificant as shown by figures 28 and 29 and 27 respectively.

Shrinkage Rings

Twelve mixes were tested in the double ring molds. Type k mixes typically expanded initially then shrunk slightly as shown in figure 30. The conventional mix showed no change in the strain as measured on the surface of the inner and outer steel rings as shown by figure 31. Calculations estimated the Type k rings would only expand approximately 50 microstrain at 150 psi. The test rings showed approximately 100 microstrain indicating the Type k expansion is not limited to 150 psi of restraint. The outer ring expanded 130 microstrain while the inner ring compressed 105 microstrain. This difference between inner and outer ring strains is illustrated in figure 30, matching exactly the expected result generated by assuming a thin walled pressure vessel. Mixes L and N as shown in figure 31 were typical of the conventional mixes tested.

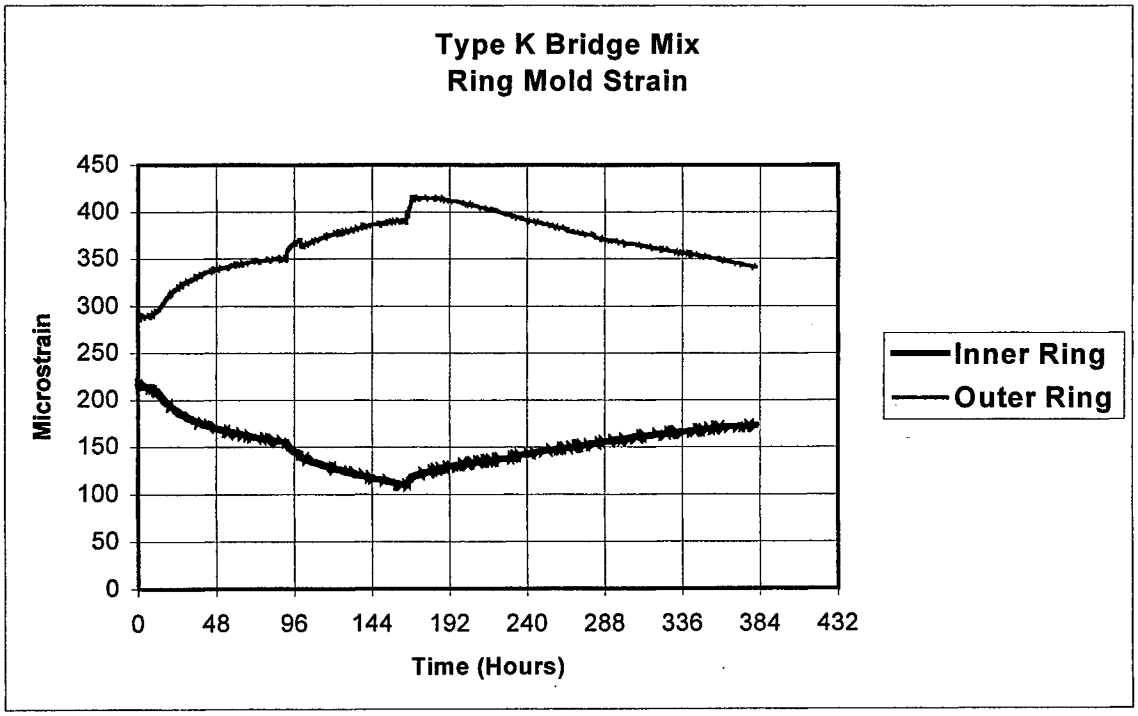

Figure 30. Microstrain versus time for Type k ring mold.

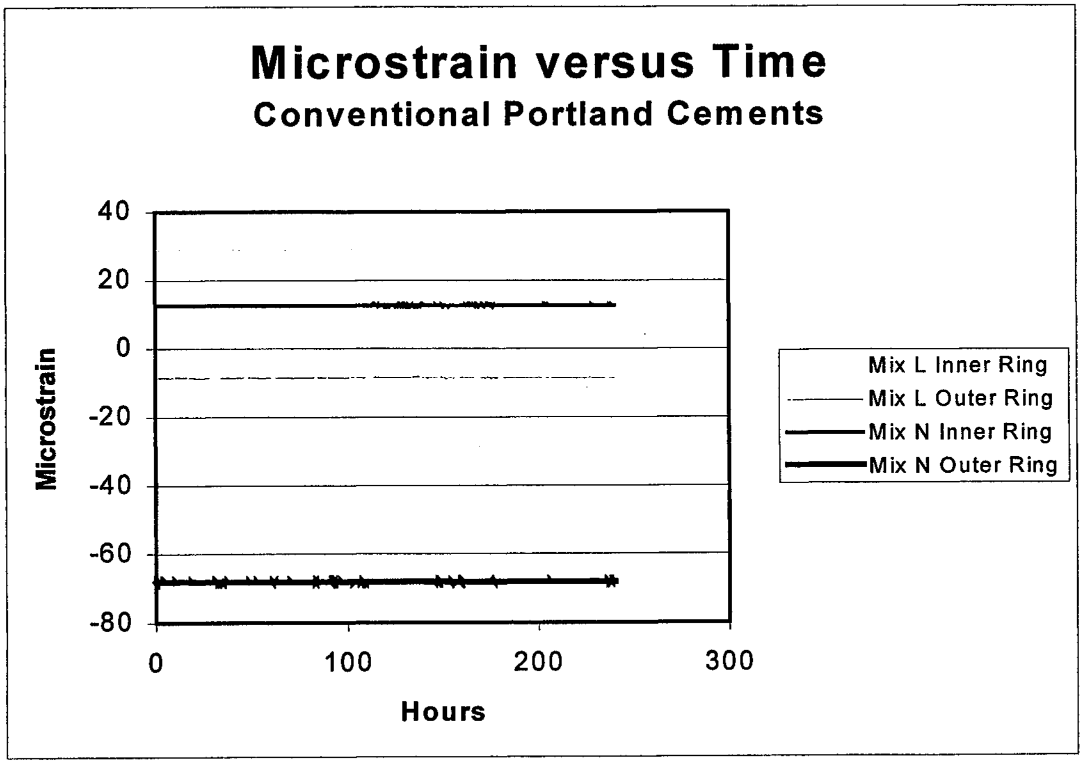

Figure 31. Microstrain versus time for conventional concrete ring mold.

Some mixes caused up to 5 microstrain in the rings from early hydration temperatures, however these strains disappeared before the rings were removed from the curing room implying the expansion was due to a temperature increase during hydration. When the rings were cut from the specimens there was a large strain change, some outer rings had approximately 500 microstrain built up. Most of this strain can be attributed to residual stresses from manufacture. Several inner ring strains were monitored when the outer ring was removed. In these cases an average of 10 microstrain of expansion was recorded, indicating that the concrete annulus was still under compression from two directions more than one year after casting.

The conventional cracking tendency test, as previously shown in figure 1, generates a radial moisture gradient through the sample by allowing drying only from the outer circumferential surface. This situation is physically accurate only for bridge decks cast with stay in place forms. The fact that most bridge decks do not dry from only one side is irrelevant when comparing different concretes in the same test apparatus for relative cracking tendency.

A more flexible and removable outer ring should be used to measure initial expansion and restraint but should be removed when drying shrinkage begins. This would allow for expansion data and thermal strain measurements during hydration and curing. The rings most likely would never have cracked due to the level of confinement provided by the double ring tests. The removable outer ring would provide the initial restraint against expansion and eliminate the effect of confinement from two directions. Allowing drying from the outer circumference as well as the top and bottom would be expected to make the ring test more reliable for Type k mixes. This will also minimize moisture gradients within the sample. Regardless of which test is adopted a relative cracking tendency test is recommended for use state agencies.

CHAPTER VI

CONCLUSIONS

Based on the data obtained during this study for the materials tested the following conclusions seem appropriate. These conclusions may or may not apply to materials not tested.

1) Field Work Observations:
   a) Strain gages bonded to reinforcement should be mounted in the laboratory and proof tested prior to installation.
   b) Concrete strains should be monitored with reinforcement strains.
   c) Cracking was not observed on either the control or Type k concrete bridges during the testing period.

2) Expansive Concretes made with Type k cement:
   a) Expansion is reduced by silica fume and water reducers.
   b) Free shrinkage prisms show that shrinkage is offset by initial expansion.
   c) Free shrinkage was greater than initial expansion for the mix containing 5% silica fume and a WRA. This was the only one of the four Type k mixes tested that showed an ultimate negative length change at 180 days.
3) Ground Granulated Blast Furnace Slag:
   a) Low alkalinity cement interacts more effectively with GGBFS than do high alkalinity cements.
   b) Long term drying shrinkage shows more benefit from GGBFS than short term drying shrinkage.
4) Silica Fume:
   a) Silica fume reduces the early expansion in Type k concretes.
   b) Silica fume increases short term drying shrinkage.

   c) Cement alkalinity is less of a drying shrinkage factor with the addition of silica fume.

5) Water Reducing Agents
   a) WRAs increase shrinkage:
   b) Daracem 100 increased shrinkage by an average of 0.020%.
   c) The WRDA with HYCOL used with the Type k mixes produced similar results.
6) Shrinkage Reducing Agents:
   a) Cement alkalinity is less of a drying shrinkage factor with the addition of a SRA.
   b) The effect of poor curing on shrinkage is greatly reduced by the addition of a SRA.
   c) SRAs reduce the early drying shrinkage caused by the addition of silica fume.
   d) A medium dose of 1.5 gal/yd^3 reduced 20 day and 180 day shrinkage by approximately 40%.

7) Shrinkage Rings:
   a) A removable outer ring is required for testing expansive concretes.
   b) Too much restraint is present with the rigid permanent double rings for the effective monitoring of strain.

8) Future Research:
   a) Comprehensive field data should be collected to relate strength and modulus gain, hydration temperatures, drying shrinkage, and environmental conditions to cracking tendency.
   b) Girders should also be monitored for deck interaction. Knowledge of strand strain as well as strain in both flanges would be beneficial.

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