JOINT TRANSPORTATION RESEARCH PROGRAM

FHWA/IN/JTRP-2008/29-1

Final Report

**HIGH-PERFORMANCE CONCRETE BRIDGE
DECKS: A FAST-TRACK IMPLEMENTATION
STUDY**

Volume 1: Structural Behavior

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December 2008

INDOT Research Project Implementation Plan

Date: December 31, 2008

Research Project Number: SPR-2911

Project Title: High Performance Concrete Bridge Decks: A Fast-Track Implementation Study

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INDOT Research

TECHNICAL *Summary*

Technology Transfer and Project Implementation Information

TRB Subject Code: 25-1 Bridges

Publication No. FHWA/IN/JTRP-2008/29-1, SPR-2911

December 2008

Final Report

High-Performance Concrete Bridge Decks: A Fast-Track Implementation Study, Volume 1: Structural Behavior

Introduction

Transverse deck cracking has proven detrimental to the durability of bridge decks across the United States. These cracks occur at early ages, often before traffic is carried by the deck. The occurrence of transverse cracks is most commonly attributed to restrained shrinkage of the concrete. This restraint is provided by several different sources including composite action of the underlying girder, internal restraint by reinforcing steel, and other internal and external contributors such as differential shrinkage of the concrete and stay-in-place forms.

To reduce the tendency for transverse cracking, a potential solution is the development of a concrete for which shrinkage is minimized such that restrained shrinkage stresses developed in the concrete are lower than the tensile strength of the deck concrete. Therefore, high-performance concrete has the potential of reducing transverse cracking as it can exhibit certain characteristics such as reduced shrinkage. A ternary mixture containing cement, silica fume, and fly ash can create a concrete which experiences less shrinkage than that of conventional concrete. Besides a low shrinkage tendency, high-performance concrete including silica fume is less permeable than normal concrete. This reduction in permeability has the potential of providing a denser, more durable concrete bridge deck.

The elimination of all transverse cracks in every newly constructed bridge deck is futile. Precautions must be taken to control the cracks that do form. It has been observed that reinforcing steel

in bridge decks conforming to AASHTO requirements have yielded after transverse cracks form. Once the steel yields, undesirable crack widths can develop. Furthermore, crack width growth is proportional to bar spacing; the further the bar spacing, the wider the crack widths. Controlling both crack growth and crack widths, both the bar spacing and reinforcement percentage must be considered.

The objective of this research was to determine the performance, particularly in terms of transverse cracking and shrinkage, of a bridge incorporating design details which will likely reduce cracking as well as the use of a low shrinkage, high-performance concrete. A bridge deck containing a reduced bar spacing (#5 at 6-in. in both directions and both mats) and a high-performance concrete was instrumented to evaluate the behavior of a system designed to control transverse deck cracking. The bridge carrying SR-23 over US-20 in South Bend, Indiana was chosen for this study. Data collected from the instrumented structure as well as visual crack inspections were used to evaluate the behavior of the bridge deck. In addition, the gages were used to monitor transverse crack formation. To gain an understanding of the magnitude of restraint experienced by the deck concrete, two free shrinkage specimens were cast concurrently with the deck. A comparison of the strains from the deck and from the free shrinkage specimens provided an estimate of this restraint.

Findings

From the strains measured from this bridge, it was observed that the concrete and top layer of longitudinal reinforcing steel experienced

roughly the same amount of strain. The following observations were made when analyzing the recorded strains:

1. The strains measured above the girders averaged 10 $\mu\epsilon$ in tension during the 90 day monitoring period.
2. The strains measured between the girders resulted in an average of 18 $\mu\epsilon$ in compression.
3. Over the entire 90 day period, on average, the concrete between the girders was in compression, and the concrete directly above the girders was in tension.
4. Tensile strains measured in the concrete were not sufficient to initiate transverse cracking at the locations instrumented.

In addition to the gages installed in the deck, gages were installed in free shrinkage specimens. These specimens were constructed to determine the magnitude of unrestrained shrinkage provided by the high-performance mix. The following observations were made from the free shrinkage specimens as well as from comparison of their behavior with the deck:

1. Both specimens experienced increasing amounts of compressive strain for the first 12 days after casting.
2. After the primary shrinkage period (first 12 days), only minor variations in strain were experienced and were primarily caused by temperature changes.
3. The maximum amount of shrinkage experienced in the first 12 days was 53 $\mu\epsilon$. After the initial period, the absolute

maximum strain experienced was 110 $\mu\epsilon$ on December 24, 2004, during the coldest weather of the monitoring period.

4. By comparing the results of the free shrinkage specimens to that of the deck, it was observed that there was more restraint provided directly above the girders than at mid-bay between the girders. Furthermore, as installed in the deck, the concrete was restrained, on average, approximately 32 $\mu\epsilon$.

From a visual inspection of the deck, it was observed that no transverse cracks occurred during the first 132 days. This is in agreement with the strain measurements provided during the monitoring period. No full-depth transverse cracks were observed over the entire area inspected; however, eight partial depth transverse cracks were found. The cracks were observed at the construction joint located between the two phases of construction. The surface area at this location is increased which may lead to an increase in the drying shrinkage of the concrete. On average, the cracks were spaced at approximately 18 ft apart with a depth of approximately 4 in. originating from the bottom of the slab.

From consideration of both the measured strains and the visual inspections of this deck, it was found that the high-performance concrete, coupled with modified reinforcement details can provide for a deck with less cracking than a conventional bridge deck.

Implementation

Based on the results of this study, it is found that the combination of low-shrinkage concrete along with improved reinforcing details can significantly improved the cracking behavior of a bridge deck. This research study supports the recommendations previously outlined in the previous study "Investigation of Bridge Deck Cracking in Various Bridge Superstructure Systems" (FHWA/IN/JTRP-2002/25). Specifically, it is important that implementation of the following recommendations be continued:

1. The maximum reinforcing bar spacing should be 6 in.
2. The amount of reinforcing steel should conform to the following expression:

$$A_s = \frac{6\sqrt{f'_c}}{f_y} A_g$$

where: A_g = gross area of section, in.²
 A_s = area of reinforcement in cross-section, in.²
 f'_c = specified compressive strength of concrete, psi.
 f_y = specified yield strength of reinforcement, psi.

3. A minimum 7-day wet curing period should be provided.
4. The concrete should exhibit low-shrinkage.

Significant benefits can be realized through the continued implementation of these recommendations. Minimizing deck cracking can reduce maintenance and deck replacement costs while increasing the lifespan of the bridge.

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FINAL REPORT

FHWA/IN/JTRP-2008/29-1

High-Performance Concrete Bridge Decks: A Fast-Track Implementation Study

Volume 1 Structural Behavior

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Joint Transportation Research Program
Project Number: C-36-56UUU
File Number: 07-04-72
SPR-2911

Conducted in Cooperation with the
Indiana Department of Transportation and the
U. S. Department of Transportation
Federal Highway Administration

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views and policies of the Indiana Department of Transportation or Federal Highway Administration at the time of publication. The report does not constitute a standard, specification, or regulation.

Purdue University
West Lafayette, IN 47907
December 2008

1. Report No. FHWA/IN/JTRP-2008/29-1	2. Government Accession No.	3. Recipient's Catalog No.	
4. Title and Subtitle High-Performance Concrete Bridge Decks: A Fast-Track Implementation Study Volume 1: Structural Behavior		5. Report Date December 2008	
		6. Performing Organization Code	
7. Author(s) Robert J. Frosch and Timothy S. Aldridge		8. Performing Organization Report No. FHWA/IN/JTRP-2008/29-1	
9. Performing Organization Name and Address Joint Transportation Research Program 1284 Civil Engineering Building Purdue University West Lafayette, IN 47907-1284		10. Work Unit No.	
		11. Contract or Grant No. SPR-2911	
12. Sponsoring Agency Name and Address Indiana Department of Transportation State Office Building 100 North Senate Avenue Indianapolis, IN 46204		13. Type of Report and Period Covered Final Report	
		14. Sponsoring Agency Code	
15. Supplementary Notes Prepared in cooperation with the Indiana Department of Transportation and Federal Highway Administration.			
16. Abstract <p>Transverse cracking of concrete bridge decks is problematic in numerous states. Cracking has been identified in the negative and positive moment regions of bridges and can appear shortly after opening the structure to live loads. To improve the service life of the bridge deck as well as decrease maintenance costs, changes to current construction practices in Indiana are being considered. A typical bridge deck was instrumented which incorporated the following: increased reinforcement amounts, decreasing reinforcement spacing, and high-performance, low-shrinkage concrete. The low shrinkage concrete was achieved using a ternary concrete mix. The objective of this research was to determine the performance, particularly in terms of transverse cracking and shrinkage, of a bridge incorporating design details meant to reduce cracking. Based on measurements from the bridge, it was determined that maximum tensile strains experienced in the concrete were not sufficient to initiate cracking. An on-site inspection was performed to confirm that cracking had not initiated. The data was analyzed and compared with the behavior of a similarly constructed bridge built with nearly identical reinforcing details, but with a more conventional concrete to evaluate the effect of the HPC. Based on this study, it was observed that full-depth transverse cracks did not occur in the structure and that the use of HPC lowered the magnitude of restrained shrinkage strains and resulting tensile stresses.</p>			
17. Key Words bridge deck, concrete, cracking, transverse cracking, shrinkage		18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22161	
19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages 160	22. Price

ACKNOWLEDGEMENTS

This work was supported by the Joint Transportation Research Program (JTRP) administered by the Indiana Department of Transportation (INDOT) and Purdue University through contract SPR-2911. The support of the Indiana Department of Transportation (INDOT) and the Federal Highway Administration (FHWA) are gratefully acknowledged. The authors would like to extend thanks to Dr. Tommy Nantung from the INDOT Division of Research for his support throughout the project. In addition, thanks are extended to members of the Study Advisory Committee for their participation and thoughtful comments throughout the project. These members include Anthony Zander, John Jordan, Scott Newbolds, and Keith Hoernschemeyer.

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CHAPTER 1

INTRODUCTION

1.1 Bridge Deck Cracking

A 1996 study by the National Cooperative Highway Research Program including 52 state and other transportation authorities found that more than 100,000 bridges have experienced early-age transverse cracking (Krauss and Rogalla 1996). Bridge decks in Indiana are no exception to this cracking phenomenon. Cracking has been identified in both the negative and positive moment regions of the bridge deck. These cracks are consistently observed on the top of the deck and when visible, on the bottom side of the deck as well. They have been reported by a number of agencies in various locations encompassing a wide range of climates. Transverse cracking occurs regardless of superstructure type (steel or concrete). These cracks are typically transverse, full depth, and spaced 3 to 10 ft apart (McDonald, Krauss, and Rogalla 1995). These cracks are detrimental to durability of the bridge deck; therefore, their control is essential for the extension of bridge deck life.

1.1.1 Crack Types in Concrete Bridge Decks

Cracking found on bridge decks occurs in several different forms and is usually characterized according to the orientation with respect to the longitudinal axis of the bridge. There are five major crack types that are associated with bridge decks, including: transverse, longitudinal, diagonal, map or pattern, and random. In general, cracks of any type occur when the stress in the concrete exceeds the tensile strength of the material. These stresses may be induced by all or some of the following: thermal, bending, fatigue, shrinkage, corrosion of embedded steel, and chemical reactions.

1.1.1.1 Transverse Cracking

Of the five types of cracks listed, transverse cracking is the most common (Schmitt and Darwin 1995). These cracks usually appear soon after casting and often appear above the top layer of reinforcement. The reinforcement acts as an obstruction to settlement, and thus a crack will form above it as shown in Figure 1.1. Transverse cracks do not necessarily occur perpendicular to the longitudinal axis of the bridge. In locations where the top bars in skewed bridges are placed parallel to the skew, transverse cracking tends to occur above the bars parallel to the skew. Examples of transverse deck cracks are presented in Figures 1.2 and 1.3.

Figure 1.1: Obstructed Settlement Cracking (ACI 224 1998)

Figure 1.2: Transverse Deck Cracks Spaced Along Deck

Figure 1.3: Vertical Transverse Crack Through Plane of Reinforcement

1.1.1.2 Longitudinal Cracking

Longitudinal cracking occurs primarily in solid and hollow-slab bridges (Schmitt and Darwin 1995). These cracks usually occur above the longitudinal bar nearest to the top surface in solid slab bridges and above the void tube in hollow-slab decks. The most predominant factor leading to longitudinal cracking is thought to be the obstructed

settlement that the reinforcing bars and void tubes introduce. Another common location for longitudinal cracking is directly above the edge of the girder. This cracking is most probably caused by the construction practice of turning up the leg of angles that secure stay-in-place metal deck pans which in return cause a stress concentration at that location. An example of this detail can be seen in Figure 1.4. Figure 1.5 shows longitudinal cracking above the edges of the underlying girders.

Figure 1.4: Typical Detail of SIP Deck Pan Connection to Girder

Figure 1.5: Longitudinal Cracks Above Girder Edges

1.1.1.3 Diagonal Cracking

Diagonal cracking is usually associated with locations of the deck close to the ends of skewed bridges and also over single-column piers (PCA 1970). In these instances, the structure experiences resistance to deformation which is the likely cause for these diagonal cracks.

1.1.1.4 Map or Pattern Cracking

Map cracking is very common in bridge decks. The occurrence of these cracks are often credited to inadequate or improper curing which allows the surface moisture to evaporate too quickly; therefore, the bleed water cannot keep the top surface hydrated. These cracks are shallow in depth and very fine in width (Figure 1.6) . They are not thought to be detrimental to the long term durability of the deck.

Figure 1.6: Map Cracking

1.1.1.5 Random Cracking

Random cracking as the name suggests does not have a particular form. Irregular surface cracks form on the surface of the slab that do not fit into any other category listed above. The causes of these cracks are typically considered to be local imperfections or loadings.

1.1.2 Concerns with Bridge Deck Cracking

Cracking in concrete bridge decks is a problem throughout the United States. Cracks are frequently several times wider than the 0.007 in. limit suggested by ACI Committee 224 (ACI 224 2001) for concrete exposed to deicing chemicals. It is often observed that transverse cracks range in widths from 0.002 in. to 0.025 in. (Krauss and Rogalla 1996). These cracks can accelerate deterioration by allowing water and deicing chemicals to more easily penetrate the deck. As stated previously, cracks are often full

depth which quickens the degradation of the supporting girders and shortens the life of the overall structure.

Once deicing salts infiltrate the deck and reach the reinforcing steel, steel corrosion is likely to result. Corrosion of reinforcing steel can lead to cracking in the top surface of the deck or a delamination caused by the expansive pressure of the corrosion products. When two or more cracks intersect, there is also a risk of spalling at the crack intersections (Krauss and Rogalla 1996).

Methods that can reduce cracking and make the concrete less susceptible to infiltration of chlorides can increase the lifespan of the deck and reduce maintenance costs of the structure. For example, high performance concrete has many properties that may enhance performance, such as a reduced shrinkage tendency, and a denser microstructure to limit chloride permeability.

1.2 High Performance Concrete

In 1987, the United States Congress initiated the five-year, \$150 million dollar, Strategic Highway Research Program (SHRP) to investigate various products to improve the constructability and reduce maintenance of the nation's highways and bridges (Moore 1999). From this research program, high performance concrete (HPC) for bridge decks evolved.

High performance concrete offers compelling advantages for transportation structures such as bridge decks and substructures. HPC can exhibit high strength to resist loads, low permeability to protect reinforcing steel from corrosion, improved durability to extend the service life of the deck and structure, and many other advantages. It is important to point out that high performance concrete as discussed here does not necessarily mean high strength, but it refers to concrete that provides specific properties that cannot always be achieved with conventional concrete.

One of the most common reasons for engineers to specify HPC is to improve the durability of their design. Many of the factors affecting durability involve the ingress of water which contains aggressive ions such as deicing salts. Therefore, a concrete that is less permeable will improve durability and increase the service life.

The permeability of concrete can be reduced with a decrease in the water-to-cement ratio or through the addition of chemical and mineral admixtures that improve consolidation or create a denser microstructure. The dense microstructure provided by some high performance concretes can reduce the permeability, but cracking must also be considered in determining the chloride ingress in the deck. Cracking may counteract the reduction in permeability provided by the HPC and eventually lead to a higher overall permeability and reduction in the durability of a structure. Cracks create interconnected channels that freely allow penetration of deicing salts or other harmful substances.

Another characteristic of a concrete that can improve the durability of a bridge deck is low shrinkage. During the early ages of the concrete, there is an increased tendency for the concrete to shrink. Through restraint provided by the girders and the reinforcement, shrinkage produces stresses in the young concrete that can initiate cracking. These early age cracks provide a route for the intrusion of moisture and chlorides to reach the reinforcing steel. By using a low shrinkage concrete, the amount of stress developed from external or internal restraint can be reduced. In addition, a lower

initial rate of shrinkage allows more time for the development of higher tensile strength and cracking resistance.

1.2.1 HPC Defined

High performance concrete as defined by the American Concrete Institute (ACI) is “concrete meeting special combinations of performance and uniformity requirements than cannot always be achieved routinely using conventional constituents and normal mixing, placing, and curing practices (Russell 1999).” ACI further states “a high performance concrete is a concrete in which certain characteristics are developed for a particular application and environment.” Examples of characteristics that may be considered critical for an application are:

- Ease of placement
- Compaction without segregation
- Early age strength
- Long-term mechanical properties
- Permeability
- Density
- Heat of hydration
- Toughness
- Volume stability
- Long life in severe environments.

It should be noted, however, that a uniform definition for HPC does not exist since there are differences in opinions to what constitutes a high-performance concrete. The Strategic Highway Research Program (SHRP) provides another definition for high-performance concrete using the following criteria:

- A maximum water-cementitious ratio (w/c) of 0.35
- A minimum durability factor of 80%, as determined by ASTM C 666, Procedure A
- A minimum strength criteria of either
 1. 21 MPa (3,000 psi) within 4 hours after placement (Very Early Strength, VES)
 2. 34 MPa (5,000 psi) within 24 hours (High Early Strength, HES)
 3. 69 MPa (10,000 psi) within 28 days (Very High Strength, VHS)

Goodspeed et al. (Goodspeed, Vanikar, and Cook 1996) proposed a definition of HPC for highway bridge applications in 1996 for the Federal Highway Administration (FHWA). The proposed definition is flexible and has many parameters which allow designers the ability to choose different categories of performance. The definition consists of four strength parameters and four durability parameters as listed in Table 1.1. Goodspeed went on to develop a table that divides different HPC mixtures into categories depending on their characteristics (Table 1.2).

Table 1.1: Definition of HPC for Highway Bridges

Strength Criteria	Durability Criteria
Compressive Strength	Freeze-Thaw
Modulus of Elasticity	Scaling
Shrinkage	Abrasion
Creep	Chloride Permeability

Table 1.2: FHWA HPC Performance Grades

Performance Characteristic	FHWA HPC Performance Grade			
	1	2	3	4
Freeze/Thaw Durability ¹ (x = relative dynamic modulus of elasticity after 300 cycles)	$60\% \leq x < 80\%$	$80\% \leq x$	-	-
Scaling Resistance ² (x = visual rating of the surface after 50 cycles)	x = 4,5	x = 2,3	x = 0,1	-
Abrasion Resistance ³ (x = avg. depth of wear in mm)	$2.0 > x \geq 1.0$	$1.0 > x \geq 0.5$	$0.5 > x$	-
Chloride Permeability ⁴ (x = coulombs)	$3000 \geq x > 2000$	$2000 \geq x > 800$	$800 \geq x$	-
Strength ⁵ (x = compressive strength)	$6 \leq x \leq 8$ ksi	$8 \leq x < 10$ ksi	$10 \leq x < 14$ ksi	$x \geq 14$ ksi
Elasticity ⁶ (x = modulus of elasticity)	$4 \leq x < 6 \times 10^6$ ksi	$6 \leq x < 7.5 \times 10^6$ ksi	$x \geq 7.5 \times 10^6$ ksi	-
Shrinkage ⁷ (x = microstrain)	$800 > x \geq 600$	$600 > x \geq 400$	$400 > x$	-
Creep ⁸ (per psi) (x = $\mu\epsilon$ /pressure unit)	$0.52 \geq x > 0.41$	$0.41 \geq x > 0.31$	$0.31 \geq x > 0.21$	$0.21 \geq x$

¹ Test in accordance to AASHTO T 161 (ASTM C 666 Procedure A)² Test in accordance to ASTM C 672³ Test in accordance to ASTM C 944⁴ Test in accordance to AASTHO T 277 (ASTM C 1202)⁵ Test in accordance to AASTHO T 22 (ASTM C 39)⁶ Test in accordance to ASTM C 469⁷ Test in accordance to ASTM C 157⁸ Test in accordance to ASTM C 512

1.2.2 Binder Materials

Conventional concrete is normally produced using cement, aggregates, and water. To achieve the properties desired of high performance concrete, other supplementary materials such as silica fume, fly ash, or ground granulated blast furnace slag are also often included in the mix. Each of these supplementary materials changes the properties of the concrete in different ways.

1.2.2.1 Silica Fume

Silica fume is the byproduct of the fabrication of silicon metal, ferrosilicon alloys, or other silica alloys. Many designers consider silica fume as though it was just recently introduced into the concrete world, but it was been around since the 1950s in Norway and since the 1970s in the United States (Holland 2001).

Silica fume is a highly reactive material that is used in small amounts to improve the properties of both the fresh and the hardened concrete. These improvements are achieved in two ways: physically and chemically.

The physical contribution is achieved during mixing of silica fume into concrete. A vast number of very small particles are introduced into the system. Similar to the way sand fills the voids between the coarse aggregate, silica fume fills the voids between the cement particles which is often called particle packing or micro filling. Even without a chemical contribution, considerable improvement to the concrete is achieved from the physical contribution alone.

The chemical contribution comes about due to the fact that silica fume has a very high silicon dioxide content which makes it very reactive in concrete. As the portland cement reacts chemically, it releases calcium hydroxide which then reacts with the silicon dioxide from the silica fume to form additional binder material.

It is important to note that concrete with silica fume, in general does not bleed. This means that it can be finished faster than conventional concrete, but the lack of bleeding can cause plastic shrinkage cracking unless necessary precautions are taken. Even with the possibility of increased shrinkage cracking, silica fume is commonly included in HPC to decrease permeability which can decrease the amount of chlorides that can infiltrate the concrete.

1.2.2.2 Fly Ash

Fly ash is a pozzolan which is derived from the noncombustible residue of pulverized coal after burning in power plants. The particles are swept out of the furnace with the gases and collected by electrostatic precipitators after cooling. Fly ash is the most extensively used supplementary byproduct material (Mindess, Young, and Darwin 2003). Fly ash is used in a variety of situations with much of the motivation for use being economic; the cost of fly ash is less than one-half the price of cement. Although economics may encourage the use of fly ash, it is important to note that its use can also improve the properties of the concrete.

The spherical shape of fly ash particles contribute to increased workability and improved packing. This increase in workability allows the use of less water for a given slump. The reduction in water-to-binder ratio promotes a more durable and stronger

concrete. The use of fly ash also introduces more fines into the system which improves cohesiveness, pumpability, and finishability while also reducing segregation.

Fly ashes are subdivided into two classes, F and C, which reflects the composition of the inorganic fractions. Class F is found east of the Mississippi River and comes from anthracite or bituminous coals. Class C fly ash comes from lignite or sub-bituminous coals.

In general, fly ash will react more slowly than its cement counterpart. However, this delayed reaction can be offset through the use of accelerators or a ternary mix including silica fume.

1.2.2.3 Ground Granulated Blast Furnace Slag

Ground granulated blast furnace slag (GGBFS), or slag, is a waste product in the manufacture of pig iron. Chemically, slag is a mixture of silica, lime, and alumina. It consists of the same oxides as portland cement, only in a different proportion. The composition of the slag is highly dependant on the rate at which it is cooled. Slag must be rapidly cooled, or quenched, to form a hydraulically active calcium aluminosilicate glass in order to be useful in concrete.

1.2.2.4 Binder Combinations

To achieve desired concrete properties, it is often necessary to produce a mixture of the various binder materials. There are definite advantages of using more than one supplementary material. The majority of concrete placed in the United States today has at least one supplementary material (Tennis 2002). If two supplementary admixtures are used in conjunction with cement, then the term ternary mixture is used to describe the concrete.

Supplementary cementitious materials such as fly ash or slag decrease the early strength of the concrete because they react slower than portland cement. One method for overcoming this slower strength gain is to add a second, more rapidly reacting supplementary material such as silica fume.

In a recent article (Bousoubaâ 2004), effects of a ternary mix design incorporating the use of portland cement, fly ash, and silica fume were investigated. It was found that for a mortar mixture with a water-to-cement ratio (w/cm) of 0.40 and low fly ash and silica fume contents (20% and 4%, respectively), the synergetic effect of the two materials led to a mortar with less cracking tendency during plastic shrinkage than those using fly ash or silica fume by themselves. The resistance of the concrete to chloride-ion penetrability was improved using a ternary mixture regardless of the type of fly ash, w/cm ratio, and the fly ash and silica fume contents when compared to a portland cement mixture alone.

1.2.3 Problems with High Performance Concrete

Many states that have used high performance concrete for bridge deck applications have observed an increased cracking tendency. HPC bridge decks are especially sensitive to water loss and poor curing practices. This sensitivity comes from the fact that the dense microstructure of the mix prohibits bleed water channels from forming, thus not providing a path for the bleed water to rise to the surface. Also, with

the use of higher cement contents and silica fume, hydration occurs faster than for conventional concrete. The increase in the rate of hydration combined with the lack of bleed water at the surface leads to increased plastic shrinkage shortly after casting. Furthermore, at water-to-cement ratios below 0.42, at which HPC mixes are often designed, autogenous shrinkage plays a more critical role. Therefore, HPC requires better curing than typically applied on conventional concrete bridge decks.

An AASHTO Subcommittee recently proposed changes to the AASHTO LRFD Bridge Construction Specifications (AASHTO 2004). The suggested changes involve modifications to the curing requirements to decrease the cracking tendency of bridge decks. The proposed specifications require that curing commence immediately after the free water has evaporated from the surface and finishing operations are completed. Since HPC usually does not experience free water on the surface, this translates into curing beginning immediately following finishing operations. AASHTO currently requires curing for seven days unless pozzolans in excess of 10 percent of the portland cement are used in the mix. When pozzolans are in excess of 10 percent, the curing period must be extended to 10 days.

1.2.4 Experiences with High Performance Concrete in Bridge Decks

In recent years, many states have started using high performance concrete to improve bridge life expectancy. The following are several experiences with high performance concrete.

1.2.4.1 New Hampshire

New Hampshire began using high performance concrete in 1996 to produce a highly impermeable, crack-free, freeze-thaw resistant concrete. To make certain that they achieved their objectives, they required a number of trial batches to refine any problems. Once the mix proportions were approved by NHDOT, a 5 cyd trial pour was conducted simulating the actual placing, finishing, and curing conditions. This trial pour was considered very important for the purpose of allowing the contractor the ability to fine-tune the admixture dosage to ensure a workable mix. The specifications for the deck concrete mix are shown in Table 1.3 while the approved mix design is listed in Table 1.4.

Table 1.3: Deck Concrete Specifications

Item	Specification
Cement	Type II
Silica Fume	7.50%
w/cm max.	0.38
Air Content	6 to 9%
28-day Cylinder	7200 psi
Chloride Ion	1000 coulombs
Corrosion Inhibitor	4 gal/yd ³
Curing Procedure	4-day wet cure with cotton mats

Table 1.4: Deck Concrete Mix Design

Material	Quantity (per cyd)
Type II Cement w 8% Silica Fume	660 lb
Fine Aggregate	1190 lb
Course Aggregate	1815 lb
Water	253 lb
Air Entrainment	5 oz.
Water Reducer	20 oz.
High-Range Water Reducer*	158 oz.
Corrosion Inhibitor	4 gal
Water/Cementitious Material Ratio	0.38

* Added at the Site

The concrete was poured using conventional techniques and covered with cotton mats within 15 minutes after a section was placed. Curing consisted of maintaining the wet cotton mats in direct contact with the concrete for a period of four days. The quick placement of the mats prevented surface drying and eliminated the initiation of shrinkage cracks (Waszczuk 1999). Once adequate concrete strength was achieved, the surface was transversely saw-cut 1.5-in. on center with grooves approximately 0.125-in. wide by 0.25-in. deep. Tining or raking the surface was not used due to the fear of tearing and exposing the surface to drying elements.

During the deck pour, some difficulty was encountered maintaining the required air content and a consistent slump. A higher dosage of superplasticizer than that used in the trial pour was needed at the site to achieve the desired workability. This difference is largely unexplained, but NHDOT believes that the interaction between the corrosion inhibitor and the other admixtures during travel time may have contributed to the inconsistent air content and slump results.

The final results of the bridge exceeded NHDOT's expectations. During several post-construction inspections, no visible cracks were observed. The University of New Hampshire conducted an extensive study of the deck surface and concluded that only microscopic longitudinal flexural cracks were present over the girder lines.

1.2.4.2 Illinois

The city of Chicago has recently reconstructed Wacker Drive which is a major two-level viaduct bordering the north and west sides of downtown. The existing 75-year old structure is being replaced because of severe corrosion of the embedded reinforcing steel and spalling of the concrete cover. Because a construction project of this size in downtown Chicago brings many complications, it was desired to incorporate a concrete that would provide a service life of 75 years.

Like most states using HPC, the requirements for the concrete are focused on durability, not strength. A lengthy process for prequalification of concrete materials began in 1999 when a plan was developed requiring evidence that the raw materials will

provide the desired characteristics. The desired performance criteria are provided in Table 1.5. For the raw materials to be satisfactory, the mix must conform to the requirements of Table 1.6.

Table 1.5: HPC Mix Performance Criteria for Wacker Drive

Property	Required Value	AASHTO or ASTM Test Method
<i>Plastic Concrete Properties</i>		
Total Air Content of Plastic Concrete	$7 \pm 1.5\%$ *	T 152
Maximum Slump after HRWR Addition	8 in.	T 119
Slump, minimum after 45 minutes	4 in.	T 119
Initial Set Time, minimum	3 hours	T 197
<i>Hardened Concrete Properties</i>		
Post-Tensioning Strength, min.	4,200 psi	T 22
28-day Compressive Strength, min.	6,000 psi	T 22
28-day Compressive Strength, max.	9,500 psi	T 22
Total Air of Hardened Concrete	$7 \pm 1.5\%$	C 457
Air Void Spacing Factor, max.	0.010 in.	C 457
Air Void Specific Surface, min.	$500 \text{ in.}^2/\text{in.}^3$	C 457

* or as required to meet the total air content in the hardened concrete

Table 1.6: Testing of Durability and Material Properties of HPC for Wacker Drive

Property	Required Value	AASHTO or ASTM Test
Freezing and Thawing Resistance	DF > 90% at 300 cycles DF > 85% at 500 cycles	T 161
Chloride Ion Permeability Resistance	< 2000 coulombs at 28 days	T 277
Chloride Ion Penetration Resistance	1/2 to 1 in., < 0.03% Cl ⁻ by weight of concrete at 90 days 1/2 to 1 in., < 0.07% Cl ⁻ by weight of concrete at 6 months	T 259 and T 260
Deicer Scaling Resistance	Rating of 0-1 at 50 cycles	C 672
Shrinkage	< 600×10^{-6} at 90 days	T 160

Concrete suppliers submitted their materials for testing. In total, ten cements, five course aggregates, six fine aggregates, five fly ashes, one GGBFS, and three silica fumes were tested (Kaderbek 2002). Once the city of Chicago received the materials, a total of 14 HPC mixes were batched, and samples were cast at the plants for durability testing. Many potential problems were evident during these initial tests. For example, several of

the fine aggregates were found to be potentially alkali-silica reacting. Furthermore, several local cements and some fly ashes were found to have high alkali contents. Some suppliers' concrete exhibited poor performance in chloride ponding and chloride permeability, had high shrinkage, or did not meet strength requirements.

By looking at these raw material failures, it is clear that specifying high performance concrete alone was not adequate to achieve the desired performance characteristics. A list of acceptable raw materials and suppliers was created from the prequalification testing. After the contract was awarded, a large trial slab using concrete pumps and other bridge finishing equipment was required to gain experience using the approved concrete mix design. The concrete mix design used in the deck of Wacker Drive is provided in Table 1.7. Finally, a quality control and quality assurance plan specific to the high performance concrete was developed.

Table 1.7: Concrete Mix Design for Wacker Drive

Material	Quantity (per cyd)
Portland Cement ⁽¹⁾	525 lb
Fly Ash, Class F	53 lb
Silica Fume	27 lb
GGBFS	79 lb
Fine Aggregate ⁽²⁾	1140 lb
Course Aggregate ⁽³⁾	1800 lb
Water	254 lb
Water Reducer	41 fl. oz.
HRWR	55-110 fl. oz.
Air Entrainment	As needed
w/cm ratio	0.27

(1) Type I/II, (2) Natural siliceous sand, (3) 3/4-in. max. size limestone

1.2.4.3 Minnesota

The bridges in Minnesota experience harsh conditions from wide temperature extremes, long snow and ice seasons, and many applications of deicing chemicals. Since the 1970s, the state has used a deck protection system including epoxy coated reinforcement, a 7-in. thick conventional concrete slab, and a 2-in. thick low slump overlay (Kivisto 2003). Mn/DOT has had good experiences from the use of this system, but the state is also interested in a HPC deck as these could provide a cost savings of 5 percent or more.

Mn/DOT's HPC specifications include a minimum cementitious materials content of 611 lb/cyd and the use of 75 percent Type I cement with 20 percent Class C or F fly ash, and 5 percent silica fume. This ternary mixture is used to reduce permeability. The specified compressive strength is 4,300 psi at 28 days and water-to-cementitious material ratio is less than 0.4. During placement, the evaporation rate must be less than 0.1 lb/ft²/hr, and wet burlap or cotton mats must be placed within 15 minutes after casting. Mn/DOT requires that the concrete be wet-cured for seven days.

Fifteen HPC bridge decks were placed between 1997 and 2003. Most of the deck placements have gone well, but they have experienced a few problems. On two different decks placed in 1999, spalling occurred due to silica fume balling. The balling was not noticed during placement of the deck but became apparent after the first winter of service. In both of these cases, the concrete used a silica fume slurry that was added to the mix by an external tank at the plant. Mn/DOT is unsure why the problem originated. To keep this problem from reoccurring, Mn/DOT now requires the trucks to comply with ASTM C94 and limits the truck to 75 percent capacity. They also require the contractor to wet-sieve concrete samples on-site in order to detect balling before they are placed in the deck. To correct the problem, the contractor was allowed to core and patch the spalled areas on both bridges. The spalling reappeared the following winter on an additional bridge; therefore, Mn/DOT required the contractor to mill 2-in. from the surface and place a low-slump overlay. Mn/DOT is currently monitoring the patches on the second bridge.

Another problem that occurred on these HPC bridge decks is cracking on the surface due to improper curing practices. The first instance of cracking occurred in 1999 before there was a time limit to place the wet burlap on the finished concrete. The contractor used a fogging machine to spray a water mist over the deck. The wind speeds increased during the pour, and the misting could not maintain the rate of evaporation of the deck at a safe value. Several areas of map cracking developed and were evident after curing was completed. To prevent this problem from occurring again, Mn/DOT changed the specifications for HPC and requires contractors to place wet burlap within 15 minutes after finishing and maintain wet curing for seven days.

Another instance of early age cracking occurred in 2002 due to poor construction practices. In this case, the contractor did not have his work bridges set up behind the paving machine for immediate application of the wet burlap. Work bridges are platforms spanning the width of the bridge enabling the workers to access the deck. The contractor attempted to fog the deck from the ends and the sides; however, this procedure did not keep up with the rate of evaporation of water from the surface. Transverse cracks at 5 ft intervals occurred throughout the deck.

Overall, Mn/Dot is pleased with the performance of their 15 HPC decks. Foreseeable problems are explained to contractors during deck pre-placement meetings in an attempt to keep them from occurring in future projects.

1.3 Concrete Shrinkage

Concrete experiences volume changes due to stimuli such as applied stress, change of moisture content, and changes in temperature. Concrete is composed of four main components: cement, aggregate, water, and admixtures. If looked upon individually, each of these components do not shrink. When viewed as a system, the combination of the components undergoes shrinkage.

Loss of water from fresh concrete is one of the major contributors to concrete shrinkage. The most common situation of this type of shrinkage in a bridge deck application is the loss of water from the top surface of the concrete. In fresh concrete, the space between the particles are filled with water. With time, the water not used in hydration evaporates or travels to the surface which creates a complex series of menisci. These menisci create negative pore pressures which lead to the entire volume of concrete

contracting. This type of shrinkage in fresh concrete is known as plastic shrinkage, while it is known as drying shrinkage in hardened concrete. During drying shrinkage, the negative pore pressures draw on water that is present on the surface of the calcium silicate hydrates (C-S-H). This attraction creates forces between the individual C-S-H particles leading to shrinkage of the system. The water that is drawn from the C-S-H surfaces is known as absorbed water. If there was no tendency for the free water in the open pores to evaporate, shrinkage would not exist because the system would be in equilibrium. Therefore, the amount of plastic and drying shrinkage is often considered to be a function of the difference between internal and external relative humidity. If the external relative humidity was kept at 100%, this form of shrinkage theoretically would not exist.

There are a total of four main types of shrinkage that concrete undergoes. The major types are drying and plastic shrinkage which were previously discussed. The other less predominate forms of shrinkage are carbonation and autogenous. Carbonation shrinkage is caused by a chemical reaction between the cement paste and carbon dioxide. The CO_2 present in the atmosphere is only sufficient to cause a considerable reaction with cement over a long period of time. Since it is estimated that this form of shrinkage only affects concrete over extended periods of time, it is believed to have insignificant effects on early-age shrinkage (Mindess, Young and Darwin 2003).

Autogenous shrinkage can occur if no additional water other than that provided during mixing is present in the system. Due to the chemical reactions during the hydration process, concrete can dry out even if no moisture is lost to its surroundings. This form of shrinkage is normally not of concern unless the water-to-cement ratio is less than 0.3. This form of shrinkage, however, is increased with the addition of reactive pozzolans such as silica fume. This type of shrinkage is rare, but with the increasing usage of HPC, it may become more appreciable.

1.3.1 Restrained Shrinkage

If concrete were free to shrink, shrinkage and temperature changes would not be detrimental to the integrity of the concrete. However, in any bridge deck application, elements exist that cause external restraint to the system. The amount of restraint in the system determines how much the movement is restricted as well as the magnitude of the stresses developed in the section. As shrinkage is restrained, tensile stresses are developed in the concrete. Depending on the magnitude of the shrinkage, these tensile stresses can exceed the tensile strength of the concrete and cause cracking.

External restraint to bridge decks comes in many forms. One of the major contributors is the restraint provided by the girders. In current design, bridge decks have become part of the structural system. The deck is compositely attached to the girders which, in return, significantly reduces the overall depth requirements of the girders. Because these girders act compositely with the deck, they provide substantial restraint, in particular along the longitudinal direction of the structure. To reduce the tensile stresses generated from this restraint, a low shrinkage concrete is preferred. High performance concrete provides a means to achieve this property.

Besides external restraint, internal restraint is also possible. Internal restraint is produced in a concrete bridge deck due to the manner in which the concrete shrinks. As soon as the concrete is cast, it begins to shrink. This shrinkage is related to the relative

humidity of the exposed surface as discussed earlier. Because only the top surface of the concrete is exposed to the external environment, it will shrink more and at a more rapid rate than the center of the deck. This differential shrinkage creates internal restraint and thus internal stresses in the concrete. Since the internal restraint varies depending on the amount of the concrete exposed to the environment, it is common to compare concrete members by their surface-to-volume ratios. If the surface-to-volume ratio increases, the amount and rate of shrinkage also increase.

Another form of internal restraint is provided by the steel reinforcement located in the bridge deck. As the concrete tries to shrink, the reinforcing steel resists this shrinkage, and as a result, the concrete experiences shrinkage restraint.

1.4 High Performance Bridge Decks

The Indiana Department of Transportation (INDOT) will begin reconstruction of the west leg of I-465 during 2007. The project will stretch for 11 miles from the interchange at SR 67 (Kentucky Avenue) to just south of the 56th Street intersection. The location of the project is shown in Figure 1.7. INDOT has termed this project *Accelerate 465*. The preliminary right-of-way acquisition and construction cost for this project are estimated at \$400 million dollars. Preliminary design has already begun (2004), and the design is expected to be completed by December 2006. Construction is expected to begin in 2007 and to be completed in 2010. One of the criteria specified at the onset of *Accelerate 465* is that all of the bridges being replaced for this project should have a long service life and require minimal maintenance.

Figure 1.7: Location of *Accelerate 465*

Purdue University has recently completed two projects related to bridge deck cracking and durability. The first study, *SPR-2404 "Investigation of Bridge Deck Cracking in Various Bridge Superstructure Systems"* (Frosch 2003), resulted in a recommendation for the area of reinforcing steel in the deck's cross section and in a maximum spacing to limit the amount and width of cracks experienced.

In a typical bridge deck design, using #4 top bars and #5 bottom bars spaced at 12-in. on center, the reinforcing steel has been observed to yield once the deck developed transverse cracks. This reinforcement spacing and size conforms to the AASHTO specifications and in particular the AASHTO LRFD empirical design method which requires 0.21 in.²/ft for the top bars and 0.31 in.²/ft for the bottom bars. Once the steel in the deck yields, it allows for uncontrolled crack growth and undesirable transverse crack widths in the deck.

To prevent yielding of the reinforcement that can result in uncontrolled crack growth, Frosch et al (2003) recommended increasing the total amount of reinforcing steel in the deck cross-section. According to the recommendation, the following total area of reinforcement is required:

$$A_s = \frac{6\sqrt{f'_c}}{f_y} A_g$$

where:

A_g = gross area of cross-section, in.²

A_s = area of reinforcement in cross-section, in.²

f'_c = specified compressive strength of concrete, psi.

f_y = specified yield strength of reinforcement, psi.

Using this recommendation for 4,000-psi concrete and 60,000-psi reinforcement, the total required steel amount is 0.63% of the gross concrete area. To control early-age cracking, Frosch et al (2003) also recommended a maximum epoxy-coated bar spacing of 6-in. to produce maximum crack widths of approximately 0.016-in. using current bridge deck cover requirements.

The second study, *SPR-2325 "Performance Related Specifications for Concrete Bridge Superstructures"* (Olek et al 2002), investigated different mixes of portland cement, fly ash, micro-silica, and ground granulated blast furnace slag with respect to their strength and durability characteristics. The project confirmed that a ternary binder concrete provides enhanced concrete durability without yielding excessive strength values. Ten concrete samples were created incorporating a ternary concrete mixture. From these samples, mathematical models were constructed that allow for prediction of strength, rapid chloride permeability, and chloride diffusion coefficient values based on the binder composition of the mixture.

Based on the previous studies, bridge deck design and construction should consider three major issues to control early age shrinkage cracking as well as provide for long term durability. These issues include:

1. Mix Design
 - a. Low permeability
 - b. Compressive strength at 28 days greater than 4000 psi but (preferably) below 6000 psi.
 - c. Maximum water to cement ratio of 0.40
 - d. Air Content of 6.5%
 - e. Low shrinkage
2. Structural
 - a. Minimum area
 - b. Reduced reinforcement spacing
3. Curing
 - a. Minimum of 7 days wet curing

1.5 Objective and Scope

The objective of this research is to determine the performance, particularly in terms of transverse cracking and shrinkage, of a bridge designed incorporating the recommendations presented in Section 1.4. A bridge on State Road 23 crossing US 31 in St. Joseph County, Indiana was chosen for this investigation. The goal of this research was to specifically evaluate the short term (< 90 days) behavior of this structure and critically examine the design and construction recommendations.

CHAPTER 2

FIELD INSTRUMENTATION

2.1 Bridge Design

To evaluate the performance of the use of an HPC mix along with smaller reinforcement spacing, a bridge incorporating this design methodology was instrumented. It was desired to select a bridge that incorporated common design practices and was similar in size to those that will be constructed as part of the Accelerate 465 program.

The bridge carrying State Road 23 (SR 23) over US Highway 20 (US 20) was chosen for this project (INDOT bridge No. 23-71-8282). The bridge is a 202 ft, two-span, continuous, composite, prestressed bulb-T beam structure. This particular bridge incorporates the use of integral end bents, which INDOT is using increasingly, and is located in St. Joseph County south of South Bend. Construction consisted of the total replacement of the current structure.

The original structure was built in 1964 and the most recent rehabilitation was performed in 1984. Due to the age and location of the structure, it displayed signs of considerable deterioration and required replacement as evident in Figure 2.1. The new bridge was designed and constructed to be consistent with the requirements of the 16th Edition of the Standard Specification for Highway Bridges (AASHTO 1996) along with the subsequent interim specifications through 2002, and the 1999 INDOT Standard Specifications and Supplemental Specifications effective March 1, 2004 (INDOT 1999). In addition, the integral end bents were designed in accordance with INDOT Bridge Design Memorandum #233 Revised (INDOT 1992). Finally, the deck reinforcement was required to meet special design requirements as discussed in Section 1.4. Structural plans for the SR 23 over US 20 bridge are provided in Appendix A, and the complete set of special provisions are provided in Appendix B.

Figure 2.1: Bridge before Construction

2.1.1 Bridge Deck Concrete

To provide the desired durability, a special provision was prepared to address the specific needs regarding the concrete mixture. All section numbers referenced in the following provision refer to the INDOT Standard Specifications. A portion of the special provision regarding concrete mix criteria is provided below:

CONCRETE MIX CRITERIA. The Concrete Mix Design (CMD) shall produce workable high-performance concrete mixtures having the following properties:

- a) The paste volume of total cementitious material and water shall not exceed 28.0% of the concrete volume design (e.g 7.56 ft³ maximum paste volume per cubic yard of concrete). Each cementitious material shall be batched within a tolerance not to exceed 1.0% in accordance to 702.06.
- b) The cement content in the ternary binder system shall be at least 231 kg (390 lbs) per cubic meter (cubic yard) of concrete. Air-entraining cements will not be permitted.
- c) Class F or C fly ash shall be used as part of the total cementitious content in the ternary binder system. Fly ash shall constitute 20.0 to 30.0 percent by mass (weight) of the total cementitious content in the mix design. Fly ash shall not be used in conjunction with Type IP cement.
- d) Silica fume shall constitute 5.0% - 7.0% of the total cementitious content in the mix design.
- e) The water-cementitious ratio shall be no less than 0.380 and not exceed a maximum of 0.420.
- f) The CMD target for air content shall be set at either 6.5% or 7.0%.
- g) The slump shall be within a range of 100 mm to 190 mm (4 in. to 7.5 in.).
- h) The target compressive strength at 28 days shall be a minimum of 33.78 MPa (4900 psi).

Because a mixture complying with all of these requirements is fairly complicated, a trial batch performed by the contractor was required to ensure that the desired characteristics were met and that the mix was still workable. The trial was held on September 16, 2004. The moisture content of the aggregate was obtained early that morning to determine the batch weights of the individual ingredients in the mix. The batching operator was provided the total moisture contents of the aggregates and mistook these values to be free moistures. Because no adjustment was made for aggregate absorption, the batch plant computer underbatched the target water by approximately 35%. As a result of this error, the moisture content of the mix was not correct given the amount of aggregate present. The concrete plant corrected this problem by adding additional amounts of aggregates and admixtures.

Once the concrete mix was adjusted to the correct moisture content, a number of specimens were collected including: compressive cylinders, cylinders for maturity measurements, restrained rings, free shrinkage prisms, and modified flexural beams to measure free shrinkage. Several of the collected specimens are presented in Figure 2.2. It should be noted that from initial batching to casting of the specimens, the concrete was in the mixing truck for over 90 minutes due to the time required for adjustments and testing. Subsequent testing indicated that the strength gains achieved from the collected

specimens did not meet INDOT's expectations. Because the concrete mix proportions were adjusted due to the error made at the plant and given that the strength gains were not as desired, a second trial batch was performed on October 5, 2004.

Figure 2.2: Collected Specimens at Trial Batch

During the second trial batch, the batching procedure was completed without the problems experienced during the first trial batch. The strength gain was satisfactory, and the mix design met the specifications in the special provisions and was considered acceptable for use in the bridge deck.

2.1.2 Bridge Deck Reinforcing Steel

Besides meeting the requirements of the AASHTO 16th Edition, it was required that the bridge deck reinforcement meet the following requirements:

1.
$$A_s = \frac{6\sqrt{f'_c}}{f_y} A_g$$
2. Maximum spacing of bars equal to 6 in.

Based on these requirements, an 8-in. deck was designed. The deck design consisted of, #5 bars spaced at 6-in. in both the top and bottom mats. Over the piers in the top mat, #7 bars were lapped with #5 bars to account for the negative moment in these regions.

2.2 Test Structure - Construction

Kankakee Valley Construction Company, Inc. was awarded the construction contract in June 2004. To facilitate traffic at all times, construction of the bridge progressed in two phases. The location of each phase of construction is presented in Figure 2.3.

Figure 2.3: Phased Construction Layout

All elements of the existing bridge associated with the first phase of the project were removed. Once demolition was completed, piles were driven for the new substructure. Numerous problems were experienced as proper bearing could not be achieved with the as-designed pile length. Additional pile length had to be added to provide adequate length to reach bearing. Following pile driving, the bents and center pier were cast. One of the end bents constructed during Phase 1 is shown in Figure 2.4.

Figure 2.4: End Bent Cast during Phase 1 Construction

Once the pier and end bent were cured, prestressed concrete Bulb-T beams were set into place. After placement of the girders, the second lift of the end bent was cast to provide an integral connection of the girder with the abutment. The integral abutment is shown in Figure 2.5

Figure 2.5: View of Integral End Bent

Diaphragms were cast at the joints between the girders to provide torsional stiffness. Stay-in-place (SIP) steel forms were used to form the bottom of the slab between the girders (Figure 2.6).

Figure 2.6: Stay-in-Place Deck Forms between Beam Flanges

After the SIP forms were connected to the concrete beams using metal clip angles, the outside coping was formed. The coping was formed by attaching standoffs to the girders as shown in Figure 2.7. Once the standoffs were attached, lumber was laid along the longitudinal direction of the bridge. Plywood was then placed in order to form the bottom of the slab. The purpose of the coping is mainly to support the barrier wall at the edge of the bridge. After the coping line forms were installed, the deck reinforcement was placed. As discussed earlier, the reinforcement layout on this deck was designed following the recommendations provided by Purdue University and presented in Section

1.4. To create a composite structure, shear reinforcement was extended through the top of the beam flange at the prestress plant. This reinforcement detail is shown in Figure 2.8.

Figure 2.7: Forming for Coping

Figure 2.8: Shear Reinforcement for Composite Action

The steel layout is shown in Figure 2.9, and the layout design as included in the plans is provided in Figure 2.10.

Figure 2.9: Layout of Reinforcement in Positive Moment Region on Bridge Deck

a) Top Positive Moment Reinforcement

b) Top Negative Moment Reinforcement

c) Bottom Reinforcement

Figure 2.10: Deck Reinforcing Steel Size and Location

Placement of the deck began at 8:00 AM on November 3, 2004 with an ambient temperature of 38°F at the west end of the bridge using a Putzmeister 422 pump truck. A mechanical screed was used to level and consolidate the concrete (Figure 2.11). Once the concrete was screeded, a fog mister located on the mechanical screed sprayed a fine mist over the previously finished concrete.

Figure 2.11: Mechanical Screeding of Deck

Once the concrete began to set, wet burlap and plastic were placed on the deck starting at approximately 7:00 PM to wet cure the concrete. After the first night of curing, insulated blankets were placed on top of the burlap and plastic due to the cold weather. This curing regime was continued until seven days after casting. It should be noted that in the special provisions, the curing period should have been ten days since the concrete was cast in cold weather. Also, the provision required the use of soaker hoses or other means to keep the burlap wet. Due to rain that occurred during the curing period, the contractor felt that the burlap was adequately moist and did not need to be rewetted. The INDOT project supervisor agreed with the assessment and thus soaker hoses as required in the special provisions, were never utilized. The barrier walls on the bridge were formed and poured on November 16 and 17, 2004. Figure 2.12 illustrates the sequence of events between deck casting and opening the bridge to traffic. Phase 1 construction was completed and the bridge was opened to traffic on November 19, 2004.

Figure 2.12: Time Line Between Casting and Opening of Bridge

It is important to note that there was a persistent problem during casting of the bridge deck. Throughout the pour, the contractor could not maintain the air content according to the specifications. Since this project was a QC/QA project for INDOT, the bridge was subdivided into two lots, each of which was further subdivided into 6 sublots. On the west end of the bridge (Lot 1), two of the six sublots went to failed material review. Of these failed sublots, Sublot 1 had the lowest air content of 3.1%. The target value of air content was set at either 6.5% or 7.0%. The air content as it entered the pump was above 10%. The location of each sublot is presented in Figure 2.13. This drastic drop in air content lead the contractor to believe that the pumping operation was the cause of the problem. It was requested that the pump operator move the pump truck away from the deck to reduce the vertical drop that the concrete was experiencing. The pump operator refused; therefore, the problem was attempted to be corrected in other ways. For instance, a sling was used to reduce the drop as the concrete was delivered to the deck (Figure 2.14). Table 2.1 lists the air content measured by INDOT from the concrete placed in the deck.

Figure 2.13: Location of Each Sublot

Figure 2.14: Sling Attached to Pump Hose

Table 2.1: Air Content Values from Deck Cast

	Lot 1 (West End)						Lot 2 (East End)				
Sublot	1	2	3	4	5	6	1	2	3	4	5
Air Content (%)	3.1	4.4	4.6	4.3	5.5	3.9	5.0	7.0	6.5	4.8	7.5

2.3 Materials

2.3.1 Concrete

The concrete for this project was obtained from Kuert Concrete, Inc., who is a local ready mix supplier in the South Bend Area. The mixture proportions and specifications for this concrete are provided in Table 2.2.

Table 2.2: QC/QA Mix Design

Material	Quantity	Specifications/Suppliers
Cement	420 lb/yd ³	Type 1, Lafarge, Alpena MI
Fly Ash (Class C)	112 lb/yd ³	ISG, Shafer unit 15
Silica Fume	28 lb/yd ³	Condensed, Rheomac SF 100
Sand	1289 lb/yd ³	#23 NS from Moose Lake, SC#2472
Stone	1782 lb/yd ³	#8 CS from Material Service, SC#2472
Water	224 lb/yd ³	City of South Bend
Air	3.5 fl oz/cwt	Micro Air, AEA, Master Builders
Water Reducer	20.0 fl oz/ cwt	Rheobuild, HRWR, Master Builders
W/C	0.4	-

Twenty-one standard 6 x 12-in. compressive test cylinders (ASTM C31) were collected in the field at the time of the bridge deck casting. The samples were taken from Sublot 5 of Lot 1. It was desired to collect samples from Sublots 3 or 4 due to the instrument locations, but due to other personnel collecting samples at that time, it was not possible to obtain concrete until Sublot 5. To simulate the concrete in the deck, the cylinders were cured under conditions matching those of the bridge deck as much as possible. Once the test cylinders were wet cured for seven days, they were transported to the Bowen Laboratory at Purdue University for testing. Compressive tests were performed at 7, 14, 21, 28, and 56 days with three specimens being tested each day. The cylinders were tested in accordance with ASTM C39 using a 600 kip testing machine and loading at a rate of 60,000 lb per minute. On each test day, the average of the three cylinders tested was recorded. The resulting strength gain curve is provided in Figure 2.15. The results of each compressive test are presented in Appendix C. As can be seen, the average 28-day compressive strength of 6,930 psi exceeds the requirement of 4,900 psi as specified in the project special provisions. Splitting tensile strength was also tested according to ASTM C496-04. The specimens were loaded at 17,000 lb per minute using the same testing machine used for obtaining compressive strengths. Three specimens were tested at 7 and 28 days. The resulting strength gain curve is provided in Figure 2.16.

Figure 2.15: Compressive Strength vs. Time for Compressive Cylinders

Figure 2.16: Splitting Tensile Strength vs. Time for Concrete Cylinders

The modulus of elasticity at 28 days was also measured following ASTM C469. Two cylinders were tested for comparison. The average modulus of elasticity was determined as 5,200 ksi (Figure 2.17). The results were also compared to ACI 318-02,

Section 8.5.1 which states, for normal weight concrete, $E_c = 57,000\sqrt{f'_c}$ where f'_c is the compressive strength and E_c is the modulus of elasticity, both in psi. The ACI equation results in a modulus of 4,700 ksi which agrees reasonably with the measured value.

Figure 2.17: Modulus of Elasticity of Concrete Specimens at 28 Days

2.3.2 Reinforcing Steel

All reinforcing steel was epoxy coated and conformed to ASTM A615 Grade 60. Tensile strength of the steel was determined according to ASTM A370 using a 120-kip Baldwin universal testing machine. Two sections of a # 5 bar were collected from the bridge site. From these two bars, specimens were cut and tested. The results of the tests are provided in Table 2.3.

Table 2.3: Tensile Test Results

	Sample 1	Sample 2	Sample 3	Average
Yield Strength (ksi)	66.0	66.7	66.3	66.3
Ultimate Strength (ksi)	99.3	99.3	99.3	99.3

2.4 Instrumentation Design

The instrumentation was designed to provide measurements that would enable evaluation of the behavior of the deck, specifically in terms of transverse cracking and shrinkage. To develop an instrumentation scheme that would provide the desired data, two different types of gages were used. Foil strain gages were installed on the steel reinforcing and embedded concrete strain gages were cast into the deck to measure strains in the bridge.

The west end (Span B) of Phase 1 construction (Figure 2.18) was chosen as the location of the installed instrumentation. Phase 1 was selected due to the time frame of construction and the west end was chosen because of the ease of placement of the cabinet for the data acquisition system.

Figure 2.18: Location of Instrumentation

2.4.1 Bridge Deck Strain Gages

Transverse cracks are, in general, first noticed at the midspan section of a bridge deck. The typical spacing of these cracks is between 3 to 10 ft along the length of the structure. Strain gages were designed for installation along a 20 ft length of the bridge deck not only to position a gage near a crack, but to also obtain the longitudinal distribution of strain. Ten gages spaced at 22 in. apart were selected to be placed along the 20 ft length of reinforcing bar. The bar was located in the top mat in the longitudinal direction, with one end of the bar at midspan between Girders 2 and 3. Transverse cracks are often seen in locations such as the one selected for placement of the reinforcing bar in this study.

Shrinkage strains are related to the amount of restraint experienced in the deck as well as the concrete's tendency to shrink. Therefore, direct measurement of strains throughout the deck is essential. When a transverse crack initiates in the deck, the strain in the steel as well as the concrete will change abruptly. By locating an embedded concrete gage directly below a strain gage attached to the steel, an understanding of the

behavior of both the reinforcement and concrete at that location can be achieved. Furthermore, if a crack forms, it should be observed in both the strain from the steel reinforcing bar as well as the strain from the embedded concrete gage. Therefore, ten embedded concrete gages were located between the two mats of reinforcement.

The girders provide substantial restraint to the deck; therefore, the strains directly above the girders as compared to midway between the girders were of interest. Girders 1 through 3 as shown in Figure 2.19 were also selected as instrumentation sites. At each of these three locations, it was decided to evaluate the strain in both the concrete and the reinforcing. These locations allow comparison of the strain behavior in-between the girders to the strain behavior directly above the girders.

A concrete gage was also located at midspan near the construction joint for Phase 1. This gage was chosen to compare the concrete strains in the rest of the deck to the strains present near the construction joint at the edge of the slab. The location of all gages is presented in Figure 2.19, and the gage labeling system is presented in Figure 2.20.

a) Plan View

b) Elevation View

- Notes:
- Reinforcement Strain Gauge
 - ◆ Embedded Concrete Strain Gauge
 - All Gauges in Longitudinal Direction
 - Reinforcement Strain Gauge Attached to Top Mat
 - Embedded Concrete Gauge Located in Middle of Deck

Figure 2.19: Locations of Strain Gages in Bridge Deck

In addition to the measurement of strain in the bridge deck, strain gages were also located on the top flange of Girder 2 and 3 at midspan to compare the strains between that experienced by the girders, the reinforcement, and the concrete.

Figure 2.20: Instrumentation Identification

2.4.2 Thermocouples

As temperature response may be related to early age cracking, the thermal gradient across the depth of the bridge was of interest. To measure the temperature gradient across the cross section, it was chosen to install thermocouples in the deck and also on the top flange of a girder. Thermocouples in the deck were located at the same depth as the top and bottom mat of reinforcing steel, while a thermocouple attached to the girder was located at the top flange. In addition to the deck temperatures, an ambient temperature measurement was also desired. To obtain an ambient temperature reading, a thermocouple was located under the deck, near the same location as the thermocouple on the girder flange. All thermocouple locations are shown in Figure 2.21.

Figure 2.21: Thermocouple Locations

2.4.3 Shrinkage Specimens

To compare the restrained shrinkage of the concrete in the deck, it was desired to construct several specimens that could be used as a method to measure the concrete's tendency to shrink.

2.4.3.1 Free Shrinkage Specimens

A representative slab was constructed to measure the free shrinkage tendency of the high-performance concrete used on the bridge deck. It was decided to place this slab next to the structure to provide similar environmental conditions. The slab was sized to be consistent with the thickness of the deck. In addition, it was also important to provide adequate slab dimensions to control the moisture gradient at the center of the specimen. The slab as designed measured 30x30x8 in. and contained two embedded concrete strain gages. One gage was located in the geometric center of the volume and the other gage was located closer to one corner of the slab as shown in Figure 2.22. To eliminate

restraint of the specimen, it was designed to be tested in a vertical position. In addition to this slab, a flexural beam form was shortened to 16-in., resulting in a specimen of 16 x 6 in., and an embedded concrete gage was located in the center of the form to provide a secondary measurement of free shrinkage (Figure 2.23).

Figure 2.22: Free Shrinkage Slab Model

Figure 2.23: Free Shrinkage Beam Model

2.4.3.2 Restrained Ring Specimens

To obtain an understanding of the shrinkage tendency for this concrete, two restrained rings were also included. A sample of the concrete was consolidated in a circular mold which encased a steel ring. The steel rings are equipped with one strain

gage at each quadrant of the ring. A schematic of the rings is presented in Figure 2.24. Compressive strains develop in the steel ring as the concrete shrinks due to restraint. As shrinkage strains increase with time, cracking may initiate in the concrete. Cracking is observed by a sudden decrease in the steel ring strain. The age of cracking and the rate of tensile stress development in the test specimen are indicators of the material's resistance to cracking under restrained shrinkage. Complete details of this test are provided in ASTM C1581-04.

Figure 2.24: Restrained Ring Specimen

2.5 Instrumentation Installation

The installation of the instrumentation was coordinated with the contractor on-site and placed as construction progressed. Complete specifications for the instrumentation installed in this bridge are available in Appendix D.

2.5.1 Strain Gages

Ten of the thirteen reinforcement strain gages were installed on a sister bar in the laboratory. These gages were installed on a #5 epoxy-coated reinforcing bar. The positioning of the gages on the reinforcing bar is shown in Figure 2.25.

Figure 2.25: Gage Layout on Sister Bar

The bar was subsequently transported to the job site and placed in the top mat in the midspan region. As shown in Figure 2.19, the instrumented sister bar was positioned between Girders 2 and 3. The other three reinforcement bar strain gages were installed in the field. These three gages were installed directly over Girders 1, 2, and 3.

For strain gage installation on epoxy-coated reinforcement, a small portion of the epoxy coating and approximately three lugs were removed. The gages were then installed and protected. The complete procedure of strain gage installation is provided in

Appendix D. All reinforcing steel strain gages were style CEA-06-250UN-350/2P (0.250-in.) manufactured by Vishay Micro-Measurements.

Two strain gages were attached to the top flange of Girders 2 and 3. Access to the underside of the bridge deck was provided by the contractor in the form of an all-terrain fork truck with a work platform attached. Installation of the girder strain gages is shown in Figures 2.26 and 2.27. An epoxy (AE-10) was spread in a thin layer on the flange surface and allowed to cure overnight. Once the epoxy was cured, it was sanded to create a smooth surface. Once sanded, the gage application was the same as for the gages installed on reinforcing bars. For installation on the surface of the concrete flange, style N2A-06-20CBW-350/P (2.000-in.) gages were used.

Figure 2.26: Fork Truck Providing Accessibility to Girder Flange for Gage Installation

Figure 2.27: Cleaning Flange Surface for Strain Gage Installation

Embedded strain gages were also installed in the bridge deck. These gages are composed of a polymer concrete outer body housing a 4 in. sensing grid made of nickel-chromium (EGP-5-350). The wet concrete conforms to the deformations in the embedded gage and cures to be securely attached to the gage.

At each gage location, a gage was placed between the two mats of reinforcement and securely attached to the top mat using reinforcing tie wire. An effort was made to place the embedded gage directly under the reinforcing bar strain gage. Where this was not possible due to chairs or other obstructions, the embedded gage was placed as close as possible to the reinforcement gage.

2.5.2 Thermocouples

Thermocouples were made of American National Standards Institute (ANSI) Type T thermocouple wire from Omega Engineering, Inc. This thermocouple wire consisted of two insulated 24 AWG solid wires, one copper and one constantan. When the two wires are twisted together, temperature can be measured through changes in wire resistance.

A single length of wire was run to each location where a temperature reading was desired. Once the thermocouple wire was in place, each conductor of the wire was stripped back approximately 0.75 in., twisted together, and soldered. Once the two ends were securely together, heat shrink was used to seal and protect the wire. Finally silicone sealant was placed on the free end of the thermocouple.

The thermocouples were subsequently installed. For thermocouples in the deck, the top thermocouple was attached to the top mat transverse bars, and the bottom thermocouple was fastened to the bottom mat of transverse bars. Plastic ties were used to securely fasten the gages. For the thermocouple attached to the concrete flange, M-Bond

200 adhesive was used to attach the thermocouple to the area of the flange where AE-10 was previously applied for adhering the strain gage. The thermocouple was then secured to the flange using aluminum tape.

In addition to Type T thermocouples, a relative humidity and temperature probe was located on-site to record the ambient temperature as well as the relative humidity of the site.

2.5.3 Datalogger Systems

To collect and record data from the installed strain gages and thermocouples, a datalogger system was installed. The location of the data acquisition system is shown in Figure 2.28. The datalogger system consisted of a Campbell Scientific CR10X datalogger to which three AM16/32 multiplexers were connected. To communicate with the datalogger system, a Redwing CDMA cellular modem by Airlink Communications was installed. Power was provided to the system via a 12 volt, 26 Ahr rechargeable battery. The battery charge was maintained using a 20 watt solar panel connected to a Campbell Scientific CH100 charger. The datalogger and multiplexers were installed in a 20x16x8 in. Nema type 4X fiberglass enclosure (Figure 2.29) to provide protection. This enclosure was mounted in a Type M traffic controller cabinet and securely fastened to a cabinet foundation (Figure 2.30). A 4x4 in. post was placed next to the traffic cabinet to hold the solar panel and cellular antenna as shown in Figure 2.31

Figure 2.28: Datalogger System Location

The datalogger system was grounded using a 1/2-in. diameter by 8-ft long copper grounding rod with a 4 AWG copper wire connecting the grounding rod to the Nema enclosure. Once the enclosure was grounded, a 12 AWG copper wire connected the copper lug of the enclosure to the datalogger.

Figure 2.29: Datalogger System in Fiberglass Enclosure

Figure 2.30: Datalogger System in Type M Cabinet

Figure 2.31: Datalogger System Cabinet with Solar Panel

2.5.4 Shrinkage Specimens

The shrinkage specimens discussed previously were cast at the bridge site and subsequently connected to data acquisition systems.

2.5.4.1 Free Shrinkage Specimens

The free shrinkage slab and beam were connected to the same datalogger system used for collection of the data from the bridge. The specimens were placed next to the datalogger system and kept on-site to simulate curing conditions of the bridge deck. The placement of the embedded strain gages is shown in Figure 2.32. Once the concrete was placed, it was consolidated using a needle vibrator and subsequently finished (Figure 2.33). Wet burlap and plastic were then placed to wet cure the concrete.

The modified flexural beam form was instrumented with an embedded concrete strain gage at the center of the specimen. The specimen was then cast, consolidated, and finished (Figure 2.34).

Figure 2.32: Instrumentation in Free Shrinkage Slab

Figure 2.33: Concrete Cast in Free Shrinkage Slab

Figure 2.34: Concrete Cast in Free Shrinkage Beam

The forms were stripped from both free shrinkage specimens on the third day after casting. Aluminum tape was used around the perimeter of the specimens to seal the edges from moisture loss simulating deck behavior. The slab was then placed on end and secured so that the specimen was able to shrink freely (Figure 2.35). The beam and slab were wet-cured for seven days at which point the burlap and plastic were removed.

Figure 2.35: Free Shrinkage Slab After Form Removal.

To connect the instrumentation for the restrained ring tests, a secondary datalogger system was used and temporarily secured on-site. This system consisted of a

Campbell Scientific CR10X datalogger and a AM416 mutliplexer. The system ran independently from the main datalogger system using only battery power since the system was only required to be in the field for 28 days. The datalogger system was mounted in a Campbell Scientific fiberglass enclosure and secured to the 4x4 in. post supporting the solar panel (Figure 2.36). The restrained rings were cast and consolidated at the same time as the other shrinkage specimens (Figure 2.37).

Figure 2.36: Connection of Secondary Datalogger System

Figure 2.37: Concrete Cast in Restrained Rings

After 28 days, the secondary datalogger system was removed from the bridge site. The specimens along with the enclosure were transported to the Bowen Laboratory and placed outside where measurements were continued.

2.5.4.2 Conduit and Wiring

Conduits were installed for the connection of the gages to the datalogger. From the traffic cabinet to the end bents, a 3 in. PVC conduit was used. At the top of the end bent, the 3 in. conduit was reduced to a 2 in. PVC conduit so that it could be placed

between the mats of reinforcement in the deck in the coping line (Figure 2.38). The wires were tied to the reinforcement once they emerged from the conduit and ran to their appropriate locations in the deck.

Figure 2.38: PVC Conduit between Mats of Steel in Deck

Belden 3-conductor shielded 24 AWG cables were pulled from the datalogger to the location of each strain gage. Each cable was then spliced to the lead-wire supplied by the gage manufacturer. The splice was protected with heat shrink over each conductor, and a larger heat-shrink was installed over the entire splice. Further protection was achieved using electrical tape and finally Scotchkote Electrical Coating.

After splicing, the ends of the wires at the datalogger were pulled into the enclosure, and the wires were connected to the multiplexers. Prior to connection to the multiplexers, the insulation of the wire was stripped back ¼-in. and soldered to provide a solid connection.

2.6 Data Collection

The program of the datalogger system was downloaded from a laptop computer at the site and began reading on November 1, 2004. The datalogger system was programmed to read every 10 minutes. The data was downloaded once a week via the cellular modem.

The secondary datalogger unit, for connection of the restrained rings, was connected the morning of the cast. The program for this unit was downloaded, and the system began reading approximately 3 hours before the restrained rings were cast. The system was also programmed to take readings at 10 minute intervals. The internal clocks of the datalogger systems were set to ensure time synchronization.

CHAPTER 3

FIELD INSTRUMENTATION RESULTS

3.1 Introduction

A summary of the data acquired from the bridge from the day of the deck cast through the 90th day following casting for Phase 1 of the bridge construction will be presented in this chapter. In particular, the thermal and shrinkage behavior of the bridge are of interest. Strains in the longitudinal reinforcement and in the concrete are investigated to estimate shrinkage strains in the concrete and investigate the development of cracking.

3.2 Thermal Response

Temperatures were recorded at midspan to investigate the heat of hydration and to provide a history of the thermal response. The ambient temperature of the site was recorded at several different locations. Three thermocouples were provided: under the deck, in the traffic cabinet, and mounted on the side of the bridge using a temperature and humidity probe housed in a solar radiation shield.

3.2.1 Ambient Temperature and Relative Humidity

Ambient air temperature and relative humidity was obtained from a CS500 temperature and relative humidity probe. The probe was mounted on the post supporting the solar panel and housed in a solar radiation shield to protect the probe from direct sunlight as well as direct contact with water. Temperatures obtained from the probe were compared to temperatures obtained from the thermocouple located under the bridge deck; this comparison is presented in Figure 3.1. The temperature data from the thermocouple from under the bridge deck followed very closely to that measured from the temperature probe with a maximum difference of approximately 4° F on January 12, 2005 and an average difference of 1.2° F.

The ambient temperature at the beginning of the cast was 37° F and the temperature at the end of the cast was 45° F. The peak ambient temperature reached during casting was approximately 50° F (Figure 3.2).

Figure 3.1: Ambient Temperature at Bridge Site

Relative humidity data collected from the relative humidity and temperature probe are presented in Figure 3.3. Relative humidity is the ratio, expressed as a percent, of water vapor present in air to the water vapor present in saturated air at the same temperature and pressure. Since the water vapor pressure in saturated air is dependent on temperature, the relative humidity is also both a function of the temperature and moisture content. This quantity is of importance due to the fact that at high temperatures and low relative humidity, the evaporation rate is highest which negatively affects the drying and plastic shrinkage of the concrete. Likewise, with high relative humidity, the evaporation of moisture is slowed on the concrete surface. The water vapor pressure in saturated air increases as the air temperature increases; furthermore, the relative humidity for a given amount of water vapor decreases as the temperature increase. Therefore, there is an inverse relationship between temperature and relative humidity as shown in Figure 3.4.

Figure 3.2: Ambient Temperature During Casting

Figure 3.3: Relative Humidity Profile at Bridge Site

Figure 3.4: Relationship between Relative Humidity and Temperature.

3.2.2 Deck Temperature

Fresh concrete undergoes an exothermic reaction referred to as the heat of hydration. This increase in heat may continue to occur for several days while the chemical reactions are still occurring. The temperature data for the first 11 days can be seen in Figure 3.5. The temperature immediately increased 18° F when the concrete was cast over the thermocouples in the deck. Later at 7:00 am on the second day after casting (11/05/04), the temperature difference between the ambient probe and the deck temperature reached another maximum which was approximately 17° F. For this specific bridge deck, the thermocouples in the deck showed a consistently higher than ambient temperature for a total of seven days. Following this time frame, the response of the deck was fairly consistent with the ambient temperature.

Figure 3.5: Heat of Hydration

3.3 Girder Strains

Strain gages were installed on the top flange of the prestressed girders to investigate the magnitude of strains experienced by the girders. After casting, both gages remained operational. A problem with one of the multiplexers in the data acquisition system resulted in the loss of seven days of data spanning from December 10 to December 17, two days of data from January 18 to January 20, and six days from January 27 until the end of the 90 days. After the last multiplexer failure, the unit was replaced and no further signs of malfunctioning have been present. The periods of lost data can be seen in all strain gages which were connected to the faulty unit.

The gages connected to the girders were zeroed at 6:00 pm on the night before the cast (11/02/04). There was a large increase of approximately $125 \mu\epsilon$ immediately after the deck was cast. The top flanges of the girders were found to be in compression which agrees with values expected at midspan considering flexural theory for a continuous beam. These results can be seen in Figure 3.6.

Figure 3.6: Girders 2 and 3 Top Flange Strains

3.4 Longitudinal Strains in Reinforcing Steel

A number of longitudinal strain gages were installed to investigate the behavior of the deck. Both embedded concrete strain gages as well as reinforcement strain gages were used.

The longitudinal gages were zeroed after the concrete deck was cast. Zeroing took place after the deck was cast because embedded concrete gages fluctuate considerably with change in temperature until they are encased in concrete due to the coefficient of thermal expansion of the gage. It was desired to zero the gages at a point in time where the embedded gages had become encased in concrete and also a point in time where the heat of hydration would not significantly affect the strains read by the gages. In determining this time, it was also important that the gages were zeroed early enough that shrinkage measurements would not be lost. It was concluded, given the heat of hydration shown in Figure 3.5, to zero all longitudinal gages at 12:00 PM on November 5, 2004. Therefore, all strains presented should be considered as changes relative to this point in time.

3.4.1 Strain Data from Reinforcing Steel

After casting, all strain gages attached to the reinforcement remained operational. Gages were attached to the longitudinal bar in the top mat above Girders 1, 2 and 3. The measurements from these strain gages are presented in Figure 3.7. Similar to the girder strain data, several spans of data were lost due to malfunctioning equipment.

In addition to the strain gages attached to the longitudinal bars above the girders, there were also gages attached to a sister bar as discussed in Chapter 2. The data

collected from the instrumentation of this bar is presented in Figure 3.8. To provide a similar scale for comparison with the embedded concrete gage plots presented later, Figure 3.9(a) presents the same plot as Figure 3.8, on a modified scale. Figure 3.9(b) shows a plot of the average strain history from the ten gages on the sister bar.

Figure 3.7: Longitudinal Reinforcing Steel Strain above Girders 1, 2, and 3.

Figure 3.8: Longitudinal Strain in Sister Bar

a) Strain Data From All Ten Gages

b) Average Strain on Sister Bar

Figure 3.9: Longitudinal Reinforcing Steel Strains on Sister Bar

3.4.2 Summary of Data Collected from Reinforcing Steel

From review of the data obtained from the longitudinal reinforcing steel, it can be observed that only gage RB4 experienced abrupt changes in the strain readings during the 90 day time frame monitored in this report. Abrupt changes were of interest as they are indicative of the formation of transverse cracks in the deck.

The average values from each longitudinal strain gage placed on the reinforcing steel are presented in Table 3.1. The average of the gages connected to the sister bar as well as the average of the three gages connected to the reinforcing bars above girders is presented for comparison purposes. The average strains are extremely small relative to the minimum yield strain for the 60-ksi reinforcing bars (approximately 2,070 $\mu\epsilon$).

Table 3.1: Average Longitudinal Reinforcing Steel Strains after Re-zeroing

Gage Location	Gage Name	Average Strain, $\mu\epsilon$, (in./in. x 10^{-6}), for each Gage	Average Strain, $\mu\epsilon$, (in./in. x 10^{-6}), for each Gage Set
Above Girder	RG1	17	16
	RG2	10	
	RG3	21	
Sister Bar	RB1	-23	-16
	RB2	-7	
	RB3	-6	
	RB4	-35	
	RB5	-8	
	RB6	-1	
	RB7	-12	
	RB8	-4	
	RB9	-25	
	RB10	-41	

Notes: 1. Average Strain is Measured Relative to Re-zeroing Time.
2. Positive Values Indicate Tension

The measurements from midspan indicate that longitudinal reinforcing bars in the top mat are in tension over the girders while the longitudinal reinforcing bars in the top mat are in compression between the girders. Because the strains are of such small magnitude, it is difficult to detect any trends occurring over time. A plot of the data from each reinforcing steel strain gage is provided in Appendix E.

3.5 Longitudinal Strain in the Concrete Deck

Embedded gages were installed between the two mats of reinforcing steel orientated in the longitudinal direction to investigate the longitudinal strain in the concrete. At each location where a strain gage was installed on the reinforcing steel, an embedded concrete gage was also installed at the mid-height of the deck. Prior to casting,

Gage EM5 which was located at midspan next to Girder 5 showed signs of erratic readings. The bridge completion module was replaced for this channel, and the readings continued to provide unreliable results. Therefore, this gage was considered as malfunctioning and will not be used for quantitative analysis. A similar tendency was noticed for Gage EB1 which was located along the sister bar. Because there were nine other embedded gages along the bar length, this gage was also eliminated in the analysis. All of the embedded gages showed significant scatter before the gages were cast into concrete; therefore, only the readings after casting should be considered. This behavior was expected as they are temperature compensated for concrete. Prior to embedment in concrete, the results from these gages are not appropriate.

3.5.1 Strain Data from Concrete Gages

The behavior of the three embedded strain gages located above the girders can be seen in Figure 3.10. After casting, the embedded gage located above Girder 2 showed signs of possible malfunctioning (EG2). The gage experiences the same diurnal trends as the other two similar gages (EG1, EG3) but is approximately 100 $\mu\epsilon$ higher. The strain gage attached to the reinforcing steel directly above the location of this embedded gage did not experience strains higher than the other two gages. Also, in several locations, there are very sharp changes in strain which is indicative of a gage problem. If a crack was to initiate in the deck at a location close to an embedded gage, the strain recorded should drop, and the steel strain should increase to maintain equilibrium. Shortly after casting, the gage steadily decreased in strain for approximately two days at which time the strain from that gage increased by 220 $\mu\epsilon$ over a period of 14 hours. Therefore, the results of this gage should be considered questionable.

Figure 3.10: Longitudinal Concrete Strains above Girders 1, 2, and 3.

Once the sister bar containing the strain gages was in place, embedded concrete gages were tied between the sister bar and the bottom mat of reinforcing steel. As stated earlier, one of these gages (EB1) was not working correctly prior to the cast. The readings after casting continued to provide fluctuations (Figure 3.11). As can be seen, Gage EB1 began to show more reliable readings shortly after the beginning of 2005. Due to the noise exhibited by this gage, its results in general were discounted. The results from the nine remaining gages are presented in Figure 3.12(a), while the average response is shown in Figure 3.12(b). As can be seen, the measured strains in general are very small and in general agreement with the data obtained from the reinforcing bars (Figure 3.9).

Figure 3.11: Malfunctioning Gage EB1 Compared to EB2

a) Strain Data From Nine Gages (EB1 Excluded)

b) Average Strain on Sister Bar

Figure 3.12: Longitudinal Strains in Concrete along Sister Bar

3.5.2 Summary of Data Collected from Embedded Concrete Gages

The longitudinal strains presented here represent the strains experienced in the concrete between the two mats of steel reinforcement. A comparison with ambient temperatures also indicates that the deck also responds with this input. The average measurements provided by the embedded gage are presented in Table 3.2.

Table 3.2: Average Longitudinal Concrete Strains after Re-zeroing

Gage Location	Gage Name	Average Strain, $\mu\epsilon$, (in./in. $\times 10^{-6}$), for each Gage	Average Strain, $\mu\epsilon$, (in./in. $\times 10^{-6}$), for each Gage Set
Above Girder	EG1	-1	38
	EG2	108	
	EG3	6	
Below Sister Bar	EB1	Malfunctioning Gage	-20
	EB2	-8	
	EB3	-13	
	EB4	-18	
	EB5	-29	
	EB6	-20	
	EB7	-16	
	EB8	-13	
	EB9	-25	
	EB10	-41	

Notes: 1. Average Strain is Measured Relative to Re-zeroing Time.
2. Positive Values Indicate Tension

The concrete at midspan in the longitudinal direction was in tension over the girders while it was in compression between the girders below the sister bar. The largest average concrete strain along the sister bar occurred on January 25, 2005 with a compressive strain of 71 $\mu\epsilon$. This value occurred shortly after the ambient temperature reached its lowest value during the monitoring period (Figure 3.1). This relationship should be anticipated due to the fact that the concrete is expected to contract as the temperature decreases. Appendix E provides data from all embedded concrete gages.

3.6 Free Shrinkage Specimens

After casting of the free shrinkage specimens, all three gages appeared to be working properly. The embedded gages used in these specimens were zeroed at the same time as the gages in the deck. Data collected from the free shrinkage slab is presented in Figure 3.13, and the data collected from the free shrinkage beam is presented in Figure 3.14. Upon further investigation of the results, however, the embedded gage at the center of the free shrinkage slab showed several signs of unusual readings. As can be seen in Figure 3.13, the readings from both gages correspond closely over most of the time

period; however, significant variations are observed on several occasions. Overall, by the end of the monitoring period, both of the gages indicate essentially identical readings and overall strain. The embedded gage installed in the free shrinkage beam was connected to the multiplexer that malfunctioned which explains the periods of lost data seen in Figure 3.14.

The shrinkage strain in both specimens experienced increasing values until approximately twelve days after casting. Past that time, diurnal variations were exhibited due to thermal expansion and contraction.

Figure 3.13: Free Shrinkage Strains from Slab Specimen

Figure 3.14: Free Shrinkage Strain from Beam Specimen

3.7 Restrained Ring Strains

Two restrained rings were also instrumented as explained in Chapter 2. On each of these rings, four strain gages were attached and monitored. The strain gages for the restrained rings were zeroed at 5:00 P.M. on November 3, 2004. The results from these gages were averaged for each ring, and the results are presented in Figure 3.15. The strain obtained from each gage is provided in Appendix E. The behavior of the restrained ring specimens matches the behavior of the free shrinkage specimens. There was a steady increase in measured strain for approximately twelve days after casting. After that time temperature variations seemed to control the behavior. No sudden changes in strain occurred during the monitoring period; therefore, no cracking was expected in the rings. This assumption was verified by visual inspection.

Figure 3.15: Strain Values from Restrained Ring Specimens

CHAPTER 4

FIELD INVESTIGATION ANALYSIS

4.1 Introduction

The results presented in Chapter 3 will be analyzed and discussed in this chapter. Comparisons will be made between the bridge instrumented in this project and a similar instrumented bridge carrying SR18 over I65 (Erickson 2004). The SR-18 Bridge was instrumented to investigate the effects of reinforcement spacing and reinforcement cross-sectional area on cracking. The bridge is a continuous two-span steel superstructure bridge measuring 242 ft-1/2 in. with a 30° skew and semi-integral abutments. For the SR-18 over I-65 bridge, one span of the bridge used the AASHTO Empirical Design maximum reinforcement spacing requirement of #5 bars spaced at 18-in. for both directions in the top mat, and #5 bars spaced at 12-in. in both directions for the bottom mat. The other span on the bridge used the Purdue Empirical Design Method and consisted of #4 bars spaced at 6-in. in both directions in both mats (Figure 4.1). This span closely matches the reinforcement layout for the instrumented structure of this project. The construction of the SR-18 Bridge took place in two phases, and both spans were constructed using conventional INDOT Class C Concrete. Comparisons can be made between the two structures which may provide insight regarding the effect of high-performance concrete.

a) AASHTO Span Top Layer

b) Purdue Span Top Layer

c) AASHTO Span Bottom Layer

d) Purdue Span Bottom Layer

Figure 4.1: Reinforcing Layout for SR-18 Bridge

4.2 Longitudinal Strain Response

The sister bar installed in the deck provides a measure of the change in strain of the reinforcement over a 20 ft length of the bridge deck. By following this strain profile over time, an insight into the behavior of the concrete deck can be achieved. Figure 4.2 shows the strain variations along the length of the reinforcement at 12:00 P.M. for 3, 7, 14, 28, 56, and 90 days after casting. For comparison, the strains from the embedment gages located directly below the sister bar gage locations are provided in Figure 4.3.

The majority of the drying shrinkage in the deck is expected to occur within the first 12 to 15 days based on the shrinkage measurements collected from the free shrinkage specimens as well as the restrained ring specimens. By looking at the strains from Days 3, 7, and 14, an increasing amount of compression is seen as time progresses. The observed trend indicates that shrinkage is occurring during this period. After 14 days, the trend is more difficult to see. The strain along the bar is greater for 56 days than for 28 days which could indicate that a higher level of compression was experienced by the gages. Day 90 strains along the bar are less than that of Day 28 or Day 56 with the exception of Gage RB4 (Location 66 in., Figure 4.2). This phenomenon may be explained by temperature cycles. As the temperature in the deck decreases, concrete and steel will shorten proportional to the temperature change if they were free to move. Since the elements are restrained, a decrease in temperature will develop tension. This temperature effect causes the diurnal cycle observed in the data. The daily temperature variation and resulting strain variation for the 28th day is presented Figure 4.4. Figure 4.4a shows the results from the embedded concrete gage and reinforcement bar gage (RG3) over Girder 3; whereas, Figure 4.4b shows the results from an embedded concrete gage (EG3) and reinforcement gage at the end of the sister bar, which is positioned midway between two girders.

The change in strain caused by temperature effects are significant given the small magnitudes of strains measured in this structure. The maximum compressive strain measured was recorded by Gage RB4 (located 154 in. from mid-span end of sister bar) with a value of $199 \mu\epsilon$ at 58 days. The maximum tensile strain in the structure was $71 \mu\epsilon$ at 85 days recorded by Gage RB6 (located 88 in. from mid-span end of sister bar). The maximum compressive and tensile strains are presented in Figure 3.8. As shown in Figure 4.4, a change of only 8°F causes a strain change of approximately $20 \mu\epsilon$.

a) Strain Profile at 3, 7, and 14 days

b) Strain Profile at 28, 56, and 90 days

Figure 4.2: Strain on Reinforcement Along Length of Sister Bar

a) Strain Profile at 3, 7, and 14 days

b) Strain Profile at 28, 56, and 90 days

Figure 4.3: Strain in Concrete Along Length of Sister Bar

a) Relationship between Strain and Temperature over Girder 3

b) Relationship between Strain and Temperature Between Girders 2 and 3

Figure 4.4: Strain Variations from Temperature Changes

4.3 Comparison To SR-18 over I-65

To investigate the effects of the high-performance concrete, a comparison was made with the SR-18 over I-65 Bridge. The strains compared here are the strains of each bridge relative to a zeroing time occurring 52 hours after the beginning of casting. The results of the steel reinforcing strains are presented in Tables 4.1 and 4.2. During casting of Phase 1 of the SR-18 Bridge deck, a number of reinforcement strain gages were lost. The averages presented are from a location between two girders at midspan, and directly above a girder at midspan. In the AASHTO Span of Phase 1, no reinforcing gages survived at midspan; therefore, the average was taken at the quarter span between two girders. Averages were computed using data collected until 56 days after casting of both decks. The strain response for SR-18 is provided in Figure 4.5 for Phase 1 and Figure 4.6 for Phase 2. The strain response of Gages RB10 (between girders at midspan) and RG3 (over girders at midspan) from the SR-23 Bridge are provided for comparison (Figure 4.7).

Table 4.1: Average Reinforcement Strains for SR-18 over I-65

Construction Phase	Location	Average Strain, $\mu\epsilon$ (in./in. $\times 10^{-6}$), Tensile Positive	
		AASHTO Empirical Span	Purdue Empirical Span
Phase 1	Over Girder	-292	12
	Between Girders	112	91
	Overall Average	-90	52
Phase 2	Over Girder	-80	-84
	Between Girders	120	-579
	Overall Average	20	-332

Table 4.2: Average Reinforcement Strains for SR-23 over US-20

Location	Average Strain, $\mu\epsilon$ (in./in. $\times 10^{-6}$), Tensile Positive
Over Girders	16
Between Girders	-40
Overall Average	-12

From review of the tables and figures, it can be seen that the AASHTO Empirically designed span of SR-18 experienced the largest amount of strain with the Purdue Modified span of SR-18 experiencing reduced strains. The use of HPC in SR-23 further improves the performance by providing an additional reduction in the strain response.

a) Purdue Span

b) AASHTO Span

Figure 4.5: Reinforcing Steel Strain Response for SR-18 (Phase 1)

a) Purdue Span

b) AASHTO Span

Figure 4.6: Reinforcing Steel Strain Response for SR-18 (Phase 2)

Figure 4.7: Reinforcing Steel Strain Response for SR-23

In addition to comparing the reinforcing steel strains, the strains experienced by the concrete of both bridges were also studied. The average values of concrete strains for the SR-18 Bridge and SR-23 Bridge are provided in Tables 4.3 and 4.4, respectively. It should be noted that no concrete gages were installed between the girders at midspan during Phase 2 construction of the SR-18 structure. A similar trend is seen for the concrete data as for the reinforcement data for SR-18 Phase 1 and SR-23 gages. The AASHTO span experienced the largest strains, followed by the Purdue Empirical Span using conventional Class C concrete. The high-performance deck exhibited the lowest average concrete strain overall. Phase 2 strains above the girders were higher for the Purdue Span than for the AASHTO Span. For further comparison, Figures 4.8, 4.9, and 4.10 show the strain response for each structure. As mentioned previously, no mid-span, mid-bay concrete gages were installed during Phase 2 construction; therefore, only one gage is plotted in Figure 4.9 for Phase 2. The gage used to monitor the strain above the girder in the Purdue Span experienced higher than usual readings until approximately 2 days after casting; after which, the readings became more reasonable. The average for this gage was taken at a time where the readings were felt to be reliable (52 hours). The fluctuating readings were most probably caused by casting operations.

From comparison of the SR-18 and SR-23 bridge, it can be observed that the strain response of the SR-23 bridge utilizing high-performance concrete is significantly lower than that of the SR-18 bridge utilizing conventional concrete. This comparison also clearly indicates that reduced spacing of reinforcement also significantly contributes to an improvement in reinforcement and deck strain response.

Table 4.3: Average Concrete Strains for SR-18 over I-65

Construction Phase	Location	Average Strain, $\mu\epsilon$ (in./in. $\times 10^{-6}$), Tensile Positive	
		AASHTO Empirical Span	Purdue Empirical Span
Phase 1	Over Girder	53	-28
	Between Girders	55	-31
	Overall Average	54	-30
Phase 2	Over Girder	-25	-44
	Between Girders	Not Available	Not Available

Table 4.4: Average Concrete Strains for SR-23 over US-20

Location	Average Strain, $\mu\epsilon$ (in./in. $\times 10^{-6}$), Tensile Positive
Over Girders	4
Between Girders	-30
Overall Average	-13

Figure 4.8: Concrete Strain Response for SR-18 (Phase 1)

Figure 4.9: Concrete Strain Response for SR-18 (Phase 2)

Figure 4.10: Concrete Strain Response for SR-23

4.4 Restrained Shrinkage

All strains collected from the bridge deck were strains experienced under restraint. To understand the effect of restraint in the system, a comparison was made between the embedded concrete gage over Girder 1 (EG1), an embedded gage between girders Gage EB10, and the free shrinkage slab (Figure 4.11). For the free shrinkage slab, the gage which was offset from the center was used because it was felt that it produced more reliable readings as discussed in Section 3.6. By subtracting the strain measured in the deck from the strain measured in the free shrinkage slab, the effect of the restraint can be estimated. The strain difference between EG1 and the free shrinkage gage is presented in Figure 4.12. The average difference from 3 November to the end of 90 days was 46 $\mu\epsilon$. An increasing difference is observed during the first ten days while significant plastic and drying shrinkage is occurring. This comparison is provided for strains directly over the girders at midspan. A comparison was also made between the free shrinkage slab and EB10, an embedded gage at the sister bar location (Figure 4.13) which is located between Girders 2 and 3. It should be noted that EB10 was chosen because it was also positioned at midspan, to be consistent with the previous comparison. All gages under the sister bar, except EB1, exhibited essentially the same strain behavior over the monitoring period (Figure 3.12a). The average difference in strain between the free shrinkage slab and EB10 was 18 $\mu\epsilon$. By comparing Figures 4.12 and 4.13, it can be seen that the magnitude of restraint is higher directly over the girders as opposed to between the girders. This outcome should be expected.

Figure 4.11: Strains used for Restraint Comparison

Figure 4.12: Difference in Strain Between Free Shrinkage Slab and EG1

Figure 4.13: Difference in Strain Between Free Shrinkage Slab and EB10

4.5 Cracking Tendency

Concrete in the deck will crack when tensile stresses in the concrete exceed its tensile strength. The strain for which the concrete is anticipated to crack can be estimated using the measured concrete properties discussed in Chapter 2 as follows:

$$f_t = 6\sqrt{f'_c}$$

where:

f_t = tensile strength of concrete, psi.

f'_c = specified compressive strength at 28 days, psi.

$$f_t = 6\sqrt{6930} = 500 \text{ psi}$$

$$\epsilon_{crack} = \frac{f_t}{E}$$

where:

ϵ_{crack} = cracking strain

E = Young's Modulus = 5,200 ksi

$$\epsilon_{crack} = \frac{f_t}{E} = \frac{500}{5,200,000} = 96 \mu\epsilon$$

From the above calculation, the cracking strain of concrete is estimated as approximately 96 $\mu\epsilon$ at 28 days. It should be noted that the compressive strength and modulus used in the above equation is at 28 days. Prior to 28 days, the cracking strain

may be less than $96\ \mu\epsilon$. The maximum tensile strain in the concrete was $160\ \mu\epsilon$ (Figure 3.10) if Gage EG2 is considered to be working properly. As discussed in Chapter 3, it appears that this gage was likely damaged. For gages considered to be in proper working order, the maximum tensile strain in the concrete is $36\ \mu\epsilon$. This value of measured tensile strain is only approximately one-third of the expected cracking strain. Even accounting for tensile strength growth prior to 28 days, the cracking strain was never exceeded.

By analyzing the data recorded from this bridge, it is expected that no transverse cracking occurred within the first 90 days. To confirm this finding, a visual inspection of the deck was conducted on March 15, 2005, 132 days after casting. During this inspection, the edge of the concrete deck was inspected at the construction joint. It is often observed that transverse cracks initiate at the construction joint and propagate across the width of the deck. Close visual inspection of the travel lanes was not possible due to the amount of traffic on the bridge.

During the visual inspection, no full depth transverse cracks were observed. Although no full depth cracks were seen, eight small partial depth cracks were noticed along the 202-ft length of the structure. All eight of the partial depth cracks occur near the tie-in bars in the section (Figure 4.14). The average spacing between cracks was 17 ft-9 in., and the location of each crack is presented in Table 4.5.

Figure 4.14: Partial Depth Crack

Table 4.5: Location of Partial Depth Cracks at Construction Joint

Crack No.	Location ¹ (ft)	Spacing, (ft)
1	38.25	30.75
2	69.00	22.92
3	91.92	16.25
4	108.17	8.83
5	117.00	12.83
6	129.83	5.09
7	134.92	27.75
8	162.67	
Maximum:		30.75
Minimum:		5.09
Average:		17.77
Standard Deviation:		9.68

¹ Along construction joint from west end control joint

No longitudinal or transverse cracks were observed on the top surface of the SR-23 deck. The only source of cracking noted on the surface was initiated by construction practices. Nails were driven into the top of the deck to secure formwork. When the nails were subsequently removed, a hole remained accompanied by a small crack as shown in Figure 4.15.

Figure 4.15: Holes in Deck Caused by Nails

For comparison, a total of 32 full depth transverse cracks were observed on the SR-18 Bridge during a crack inspection 15 days after casting. Of these cracks, 15 were found in the AASHTO Span, 16 in the Purdue Span, and one at the centerline. The average crack spacing was 6.85 ft for the AASHTO Span and 6.18 ft for the Purdue Span. Although the average crack spacing is approximately the same, the crack widths in the AASHTO Span were on average 32% wider. Crack maps of the SR-23 and SR-18 Bridges are provided in Figures 4.16 and 4.17. A plan and section view is included for the SR-23 map as the cracking was only observed partial depth. Only Phase 1 of the SR-18 Bridge was mapped because an epoxy sealer was placed on the surface of the Phase 2 deck 10 days after casting, before crack mapping could be performed.

Figure 4.16: Crack Map, SR-23

Figure 4.17: Crack Map, SR-18 (Phase 1)

By comparing crack maps, it can be seen that the early age cracking behavior of the SR-23 Bridge is superior to that of the SR-18 Bridge. When comparing the cracking behavior between two decks, it is important to note the similarities and differences of the construction of the two decks.

Both of the decks mentioned in this report were wet cured for seven days following casting. On the SR-18 Bridge, the burlap was rewetted daily to maintain a moist environment, whereas, for the SR-23 Bridge, it was felt that there was sufficient moisture in the burlap during the seven days and thus was not rewetted. After seven days for both decks, curing was removed, and the decks were allowed to air dry. Given that both bridges were cured essentially identically, differences in curing methods should not provide for significant differences in behavior.

Besides curing, other aspects should also be considered. The SR-18 deck was cast in the summer (81° F), whereas the SR-23 deck was cast in late fall (45° F). This difference may have contributed to the superior performance of the SR-23 Bridge. Figure 4.18 shows the relationships of the temperatures during the first 14 days after casting. In addition to the warmer weather, the SR-18 Bridge is supported by steel girders instead of prestressed concrete girders as for the SR-23 Bridge. It has been noted that increased transverse cracking is observed on steel girder bridges as compared to concrete girder bridges (PCA 1970, Radabaugh 2001). Another factor that could affect the comparison between the two structures is relative humidity of the site during the monitoring period. The relative humidity of the site can cause the rate of drying shrinkage to change; therefore, it is important to compare the two sites. The average relative humidity over the 56 day period for the SR-23 and SR-18 bridges was 77% and 67%, respectively. The lower average relative humidity at the SR-18 site may have induced a more severe drying shrinkage rate than that of the SR-23 deck.

One final difference that should be noted when comparing the two bridges is the reinforcement area in the deck. SR-23 used #5 bars spaced at 6 in. in both directions, and both mats. For an 8 in. deck, this corresponds to 1.29% A_g . In comparison, the east end span of SR-18 incorporated a reinforcement area of 0.83% A_g and 0.54% A_g was used in the west span. This increased area of reinforcement possibly could have influenced the behavior of the deck.

Figure 4.18: Comparison of Temperatures between SR-23 and SR-18

4.6 Scaling

It is important to note that during the visual inspection on March 15, 2005, significant surface scaling was noticed along the length of the deck. Figure 4.19 illustrates the wearing of the deck surface. A considerable difference was observed in the wearing tendencies of the approach slab, which uses conventional concrete (INDOT Class C), versus the high-performance bridge deck (Figure 4.20). The scaling observed on this deck may indicate a performance issue with the high-performance concrete supplied. The relative difference in behavior between the Class C mix and the HPC could have originated due to the fact that the Class C has a faster strength gain than does the HPC. The high-performance concrete incorporates fly ash and silica fume; therefore, the strength gain is slower than conventional concrete. This slower strength gain coupled with the low temperatures experienced at the site as early ages, could explain the difference in wearing tendencies between the two concretes. Studies should be performed to determine the cause of this phenomenon, and the deck should be further monitored to determine if this presents a performance issue.

Figure 4.19: Surface Wearing on Bridge Deck

Figure 4.20: Comparison of Bridge Deck to Approach Slab

CHAPTER 5

SUMMARY AND CONCLUSIONS

5.1 Introduction

Transverse deck cracking has proven detrimental to the durability of bridge decks across the United States. These cracks occur at early ages, often before traffic is carried by the deck. The occurrence of transverse cracks is most commonly attributed to restrained shrinkage of the concrete. This restraint is provided by several different sources including composite action of the underlying girder, internal restraint by reinforcing steel, and other internal and external contributors such as differential shrinkage of the concrete and stay-in-place forms.

To reduce the tendency for transverse cracking, a potential solution is the development of a concrete for which shrinkage is minimized such that restrained shrinkage stresses developed in the concrete are lower than the tensile strength of the deck concrete. Therefore, high-performance concrete has the potential of reducing transverse cracking as it can exhibit certain characteristics such as reduced shrinkage. A ternary mixture containing cement, silica fume, and fly ash can create a concrete which experiences less shrinkage than that of conventional concrete. Besides a low shrinkage tendency, high-performance concrete including silica fume is less permeable than normal concrete. This reduction in permeability has the potential of providing a denser, more durable concrete bridge deck.

The elimination of all transverse cracks in every newly constructed bridge deck is futile. Precautions must be taken to control the cracks that do form. It has been observed that reinforcing steel in bridge decks conforming to AASHTO requirements have yielded after transverse cracks form. Once the steel yields, undesirable crack widths can develop. It is also important to note that crack width growth is proportional to bar spacing; therefore, the further the bar spacing, the wider the crack widths. To create a deck that does not experience excessive crack widths and crack growth, a reduced bar spacing and increased reinforcement area should be considered.

The objective of this research was to determine the performance, particularly in terms of transverse cracking and shrinkage, of a bridge incorporating design details which will likely reduce cracking as well as the use of a low shrinkage, high-performance concrete. To determine this performance, a bridge was instrumented and its behavior was monitored specifically in terms of transverse deck cracking.

5.2 Field Instrumentation

A bridge deck containing a reduced bar spacing (#5 at 6-in. in both directions and both mats) and a high-performance concrete was instrumented to evaluate the behavior of a system designed to control transverse deck cracking. The bridge carrying SR-23 over US-20 in South Bend, Indiana was chosen for this study. Data collected from the instrumented structure as well as visual crack inspections were used to evaluate the behavior of the bridge deck.

In total, 13 reinforcement strain gages, 14 embedded concrete strain gages, and 4 thermocouples were installed to obtain an understanding of the behavior of the bridge

deck. Data was collected from each gage for a period of 90 days. The information accumulated was used to measure the amount of shrinkage experienced by the concrete. In addition, the gages were used to monitor transverse crack formation. To gain an understanding of the magnitude of restraint experienced by the deck concrete, two free shrinkage specimens were cast concurrently with the deck. A comparison of the strains from the deck and from the free shrinkage specimens provided an estimate of this restraint.

5.3 Results

Behavior of the deck was quantitatively investigated by the strain gages and thermocouples installed in the bridge deck.

The following observations were noted regarding the reinforcing steel:

1. The strains measured in the reinforcing steel directly above the girders resulted in an average of only $16 \mu\epsilon$ in tension.
2. The strains measured in the reinforcing steel on the sister bar, between the girders, resulted in an average of $16 \mu\epsilon$ in compression.
3. An increasing tensile strain in the steel directly over the girders was exhibited during the hydration period and through the tenth day after casting. No such trend was observed along the sister bar located between the girders.
4. Over the entire 90 day period, on average, the reinforcing steel between the girders was in compression, and the reinforcement directly above the girders was in tension.
5. The strain changes measured by the gages throughout the project period were primarily caused by thermal expansion and contraction of the deck.

To gain further insight into the behavior of the deck, embedded gages were placed in the concrete during the cast. These gages were placed at the same location as the reinforcing steel gages. The following observations were noted regarding the deck concrete:

1. The strains measured in the embedded gages directly above the girders resulted in an average of only $3 \mu\epsilon$ in tension (Gage EG2 not included in average).
2. The strains measured in the embedded gages below the sister bar, between the girders, resulted in an average of $20 \mu\epsilon$ in compression (Gage EB1 not included in average).
3. Over the entire 90 day period, on average, the concrete between the girders was in compression, and the concrete directly above the girders was in tension.
4. Tensile strains measured in the concrete were not sufficient to initiate transverse cracking at the locations instrumented.

In addition to the gages installed in the deck, gages were installed in free shrinkage specimens. These specimens were constructed to determine the magnitude of unrestrained shrinkage provided by the high-performance mix. The following

observations were made from the free shrinkage specimens, as well as from comparison of their behavior with the deck:

1. Both specimens experienced increasing amounts of compressive strain for the first 12 days after casting.
2. After the primary shrinkage period (first 12 days), only minor variations in strain were experienced and were primarily caused by temperature changes.
3. The maximum amount of shrinkage experienced in the first 12 days was 53 $\mu\epsilon$. After the initial period, the absolute maximum strain experienced was 110 $\mu\epsilon$ on December 24, 2004, during the coldest weather of the monitoring period.
4. By comparing the results of the free shrinkage specimens to that of the deck, it was observed that there was more restraint provided directly above the girders than at mid-bay between the girders. Furthermore, as installed in the deck, the concrete was restrained, on average, approximately 32 $\mu\epsilon$.

From a visual inspection of the deck, it was observed that no transverse cracks occurred during the first 132 days. This is in agreement with the strain measurements provided during the monitoring period. No full-depth transverse cracks were observed over the entire area inspected; however, eight partial depth transverse cracks were found. The cracks were observed at the construction joint located between the two phases of construction. The surface area at this location is increased which may lead to an increase in the drying shrinkage of the concrete. On average, the cracks were spaced at approximately 18 ft apart with a depth of approximately 4 in. originating from the bottom of the slab.

5.4 Recommendations

From consideration of both the measured strains and the visual inspections of this deck, it was found that the high-performance concrete, coupled with modified reinforcement details can provide for a deck with less cracking than a conventional bridge deck. This conclusion must be clarified as several factors may have contributed to the improved performance. First, the bridge deck was cast in early November in northern Indiana. The cooler temperatures experienced at early ages may have contributed to the improved behavior by reducing the rate of drying shrinkage. Second, steel girders typically experience more transverse cracking due to increased shrinkage restraint as compared to concrete girders. Third, the area of steel was much greater in the SR-23 deck as compared to the SR-18 deck; this difference should have contributed to the dissimilarity in behavior. Finally, the humidity on-site is of importance as it may cause more rapid drying shrinkage if it is low. For this bridge, the ambient relative humidity of the site averaged 75% during the first 28 days.

As discussed earlier, the contractor experienced problems in maintaining the specified air content during placement operations. It appears that the majority of the problem was associated with air loss through the pump due to improper pumping practices. Sufficient air was obtained prior to pumping, but was lost through the pumping operation. To insure that the air content problems are not repeated, trial batch demonstrations are of utmost importance. For this mix to be used on a large scale project, such as *Accelerate 465*, a more intensive trial batching and handling procedure should be

performed. A 5 cyd trial batch should be placed using the contracted pump truck to ensure that air is not lost in placement operations. The Chicago Wacker Drive construction included a trial batch similar to the one suggested, and it was found that having a larger scale trial batch along with a pump truck can help reduce problems experienced during actual bridge deck casting. A more detailed description of the trial batch procedure used by Illinois Department of Transportation for this project is provided in Section 1.2.4.2.

While no full depth transverse cracks were observed on the deck, eight partial depth cracks were observed at the construction joint between the two phases. Transverse deck cracks are expected to occur as a result of restrained shrinkage of the deck concrete. The early, full-depth exposure of the deck at the construction joint between construction phases increases the surface area from which moisture in the concrete can be lost. Increased drying shrinkage strains may result at an early age when the tensile strength of the concrete is not fully developed, and the concrete is more susceptible to cracking. To avoid this increase in drying shrinkage at the construction joint, care should be taken to ensure that the concrete is properly cured at the joint.

To reduce the amount of transverse cracking experienced in bridge decks, a modified reinforcement amount coupled with a low shrinkage high-performance concrete should be used. The results of this study indicate that a ternary concrete mix incorporating cement, fly ash, and silica fume combined with a modified reinforcement area limited to a spacing of 6 in. can reduce transverse cracking. To further deck performance, a concrete superstructure is recommended as it can provide for reduced restraining stresses.

5.5 Future Research

Continued monitoring of the SR-23 over US-20 Bridge is recommended to document transverse crack growth and evaluate the behavior of the concrete and reinforcement in the longitudinal direction over a longer duration of approximately two years. During this time frame, periodic visual inspections should be conducted, especially before Phase 2 is completed, to observe any crack growth or new crack initiations. As temperatures increase over the summer months, further information may be obtained on drying shrinkage behavior.

It is recommended that future research be performed on other structures incorporating these design details aimed at reducing cracking. It has been found that transverse deck cracking occurs more in steel girder bridges than concrete (PCA 1970, Radabaugh 2001). Therefore, a steel girder superstructure should be investigated to determine the effects of the proposed design details and high-performance concrete. While this research compared results of two similarly designed and constructed decks, the differences in superstructure type may have contributed to the improved performance. The influence of the superstructure type can be illuminated through this additional research.

The strains measured from this structure may have benefited from the temperature of the deck. Since the majority of bridge decks are cast in the summer months, it is recommended that the next instrumented structure be constructed in the summer. To isolate variables, it would be preferable that both a steel and concrete superstructure be constructed in reasonably close vicinities at approximately the same time of year.

An analytical model that can incorporate the interaction of high-performance concrete and reinforcement details should be developed and used to investigate the behavior of a deck designed to minimize cracking. The model should account for the variations in the estimated shrinkage of the various high-performance concretes and can be calibrated based on the field data. Using this model, a parametric study investigating the effects of varying reinforcement amounts, reinforcing bar size and spacing, span length, girder spacing, deck thickness, deck width, overhang width, and shrinkage of high-performance concrete on the behavior of the deck should then be conducted. Through this analysis, refined reinforcement details and concrete material specifications may be achieved that can improve deck cracking and performance while minimizing costs.

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APPENDICES

Appendix A: Bridge Plans

A.1 Introduction

Selected sheets from the Bridge Plans for the SR23 over US31 bridge replacement (INDOT Bridge No. 23-71-8282) are provided for reference. The plans were prepared by Transportation Consulting Engineers (TCE) for the Indiana Department of Transportation (INDOT). Complete plans may be obtained from the Indiana Department of Transportation, Indianapolis, Indiana.

Figure A.1: Bridge Location

Figure A.4: General Plan Sections, Notes and Design Data

Figure A.7: Floor Details (Showing Phased Construction Details)

Figure A.8: Section View of Floor Details

Figure A.9: Approach Slab Details

Appendix B: Special Provisions

B.1 Introduction

Special Provisions regarding the SR23 over US31 bridge replacement (INDOT Bridge No. 23-71-8282) are provided for reference. The Special Provisions provided were developed under a collaboration of Purdue University and the Indiana Department of Transportation (INDOT). In addition to the Special Provision of the Standard Specifications, additional requirements were presented to the contractor regarding Purdue University Testing. These documents are also included below.

B.2 Special Provisions to Standard Specifications

The Standard Specifications are revised as follows:

QUALITY CONTROL/QUALITY ASSURANCE, QC/QA, SUPERSTRUCTURE CONCRETE, MODIFIED

Description. This work shall consist of furnishing and placing a high performance concrete (HPC) for bridge superstructure in accordance with 105.03. This HPC contains portland cement and two pozzolanic materials to produce a concrete of high durability, low permeability, and low cracking potential. The HPC and the bridge structure are to be studied by the Department under JTRP (Joint Transportation Research Program) SPR-2911, authorized by the board in October 2003.

Quality Control. Quality control of the HPC during production, placement, finishing, and curing shall be the responsibility of the Contractor. The concrete shall be produced by an approved plant in accordance with ITM 405, transported, and placed according to a Quality Control Plan, QCP, prepared and submitted by the Contractor in accordance with ITM 803. The QCP shall be submitted to the Engineer at least 15 days prior to commencing concrete operations.

Concrete operations shall not begin before the QCP has been accepted and successful trial batch demonstration completed. Concrete mix designs and trial batch demonstrations may be submitted for approval prior to completion of the QCP submittal. A certified concrete technician shall supervise all sampling and testing for process control as defined by the QCP. A certified concrete technician is a Contractor, producer, or consultant employee who has been certified by the Department.

MATERIALS

Materials. Materials shall be in accordance with the following:

<i>Admixtures.....</i>	<i>912.03</i>
<i>Castings</i>	<i>910.05</i>
<i>Cast Iron Soil Pipe.....</i>	<i>908.10</i>
<i>Coarse Aggregate, Class A or Higher, Size No. 8*</i>	<i>904</i>

<i>Curing Materials</i>	912.01
<i>Fabric For Waterproofing</i>	913.16
<i>Fine Aggregate, Size No. 23*</i>	904
<i>Fly Ash</i>	901.02
<i>Permanent Metal Forms</i>	910.03
<i>Portland Cement</i>	901.01
<i>Reinforcing Steel, Epoxy Coated</i>	910.01
<i>Steel Drain Pipe</i>	910.07
<i>Silica Fume</i>	901.04
<i>Utility Asphalt, UA-1</i>	902.01(d)
<i>Water</i>	913.01

** Or gradation as identified in the QCP*

If the contract requires stay-in-place metal forms for the superstructure or if the Contractor elects to use such forms, the coarse aggregate shall be Class AP. Shipping and storage of cement shall be in accordance with 702.04.

Concrete Mix Design. *The concrete shall be designed utilizing three cementitious materials as part of the binder systems. Portland cement shall be combined with silica fume & fly ash and proportioned in accordance with Concrete Mix Criteria. A concrete mix design, CMD, and trial batch shall be in accordance with requirements detailed elsewhere in this special provision. The CMD shall be submitted in a format acceptable to the Engineer and include the following:*

- (a) a list of all ingredients*
- (b) the source of all materials*
- (c) the gradation of the aggregates*
- (d) the absorption of the aggregates*
- (e) the SSD bulk specific gravity of the aggregates*
- (f) the specific gravity of each pozzolan*
- (g) the batch mass (weights)*
- (h) the names of all admixtures*
- (i) the range of admixture dosage rates as recommended by the manufacturer*
- (j) the linear equation of unit mass (weight) vs. air content ($UW = m(\text{Air}) + b$)*

A change in material source, class or type requires a new CMD.

A CMD in accordance with this special provision may be substituted only for concrete used in the RC Bridge Approach. QC/QA superstructure concrete, modified, shall not be substituted for Class C concrete.

Concrete Mix Criteria. *The CMD shall produce workable high performance concrete mixtures having the following properties:*

- (a) The paste volume of total cementitious material and water shall not exceed 28.0 % of the concrete volume design value (e.g 7.56 ft³)*

maximum paste volume per cubic yard concrete). Each cementitious material shall be batched within a tolerance not to exceed 1.0% in accordance with 702.06.

- (b) The cement content in the ternary binder system shall be at least 231 kg (390 lbs) per cubic meter (cubic yard) of concrete. Air-entraining cements will not be permitted.*
- (c) Class F or C fly ash shall be used as part of the total cementitious content in the ternary binder system. Fly ash shall constitute 20.0 to 30.0 percent by mass (weight) of the total cementitious content in the mix design. Fly ash shall not be used in conjunction with Type IP cement.*
- (d) Silica fume shall constitute 5.0%-7.0% of the total cementitious content in the mix design.*
- (e) The water-cementitious ratio shall be no less than 0.380 and not exceed a maximum of 0.420.*
- (f) The CMD target air content shall be set at either 6.5 % or 7.0%.*
- (g) The slump shall be within a range of 100 mm to 190 mm (4 in. to 7.5 in.).*
- (h) The target compressive strength at 28-days shall be a minimum of 33.78 MPa (4900 psi).*

The Contractor may elect to use fine and coarse aggregate gradations in accordance with 904 or may propose the use of alternate gradations. If alternate gradations are proposed, the tolerances shall be as stated in the QCP. In either case, 100% of the coarse aggregate shall pass the 25 mm (1.0 in.) sieve. The combined amount of fine and coarse aggregates passing the 75 μ m (No. 200) sieve shall be from 0.0 to 2.0% for sand and gravel, and from 0.0 to 2.5% for sand and crushed stone, or crushed slag.

Proportions will be based upon SSD saturated surface dry aggregates. The fine aggregate shall be at least 35% but not more than 50% of the total volume of the aggregate in each cubic meter (cubic yard).

Absorption and bulk specific gravity tests shall be performed on the fine aggregate in accordance with AASHTO T 84 and on the coarse aggregate in accordance with AASHTO T 85, by procedures 8.1 and 8.2. When the SSD bulk specific gravity or absorption test result differs from the Department's most recent value for the source by more than the multi-laboratory precision defined within the appropriate test method, the discrepancy will be investigated. Values agreed upon by the Contractor and Engineer shall apply when calculating target batch mass (weights) and determining water/cementitious ratio.

There will be no calendar date restrictions as to the use of high performance concrete with a ternary binder system.

The high performance concrete shall contain an air entraining agent and either a water reducing, high range, admixture (type F) or a water-reducing, high range, and retarding admixture (type G) as identified in the Department's list of approved PCC Admixture and Admixture Systems. The type admixture used shall not be changed during any individual contiguous pour. The type admixture to be used shall be selected based on the expected concrete temperature, ambient temperature, initial set time, lineal rate of deck placement in m/h (ft/h), and dead load deflection of any structural members containing the concrete. When either temperature is expected to be 18°C (65°F) or above and dead load deflection is of concern; type G admixture, or HRWR admixture system shall be used. The dosage of type D or type G may be lowered to an amount as recommended in writing by the manufacturer of the admixture. A type F admixture or HRWR admixture system shall be used when both temperatures are expected to be below 18°C (65°F) or dead load deflection is not of concern. Retardation may be required due to the structure design or the proposed pour sequence in accordance with 704.04. A higher temperature restriction regulating the need to retard the concrete initial set time may be requested in writing and shall substantiate the effects of concrete initial set time, lineal rate of deck placement, and dead load deflection.

The admixture addition rate shall not be reduced below the minimum, or exceed the maximum rate recommended by the manufacturer, regardless of the temperature of the concrete or ambient temperature.

The CMD by absolute volume method shall be submitted to the Engineer for verification at least seven days prior to the trial batch demonstration. An explanation of intended use for each mix design shall be provided.

Trial Batch. *A trial batch shall be produced and tested by the Contractor's certified technician and the Engineer's qualified technician to verify that the CMD meets the concrete mix criteria. Sufficient concrete shall be batched to accurately represent the CMD and provide an amount of concrete to perform all tests from the same batch. The concrete shall be batched and mixed in accordance with 702.06 and 702.07. The Engineer will test the trial batch and provide the Contractor with the results. Trial batch concrete shall not be used for more than one test, except the concrete used for the unit mass (weight) may be used to conduct the air content test. The concrete shall be agitated at least 15 min before testing and not exceeding 45 min.*

The Contractor's test results will be used to validate CMD compliance with the required concrete properties. The air content for the trial batch concrete shall measure a minimum of 5.0%. An additional pair of cylinders shall be cast for determination of average compressive strength at an age of seven days. These cylinders are for information only. The Department and Purdue University will cast additional specimens for laboratory testing. The volume of concrete contained within these specimens will not exceed 1½ cubic yard.

The 28 day compressive strength results by the contractor shall meet or exceed 33.78 MPa (4900 psi). All molds, facilities, and materials necessary to prepare and initially cure cylinders shall be provided. Gradations shall be determined to validate the fine and coarse aggregates used.

The Engineer's qualified technician will measure the concrete properties and verify compliance to the Contractor's results within the following tolerances

<i>Aggregate Correction Factor</i>	<i>±0.1% Pt.</i>
<i>Air Content</i>	<i>±0.5% Pt.</i>
<i>Unit Mass (Weight)</i>	<i>±30 kg/ m³ (1.9 lb/ ft³)</i>
<i>Slump</i>	<i>±25 mm (1.0 in.)</i>
<i>28 day Compressive Strength</i>	<i>±8.5%</i>
<i>Water/Cementitious Ratio</i>	<i>±0.015</i>

Unit mass (weight) is not to exceed a tolerance of ± 0.5% of the CMD's predicted value at the air content measured.

All test results not within tolerance are to be investigated by the Contractor and Engineer as to the cause and determine corrective actions needed to resolve the discrepancy.

The CMD batch mass (weights) may be adjusted by the amount of over, or under yielding. The aggregates may need a re-test for SSD bulk specific gravities and absorption as part of the investigation. The final CMD shall be established for the concrete after all discrepancies are resolved. Following the trial batch demonstration all required test results, final CMD, and the linear equation shall be submitted to the Engineer for approval.

Except for adjustments to compensate for routine aggregate moisture fluctuations, changes in target aggregate (SSD) batch mass (weights) shall be documented and submitted to the Engineer for approval, prior to implementing. A maximum adjustment of ±3 percentage points of fine to total aggregate ratio by volume will be permitted. Changes to the dosage amounts of admixtures will be permitted. A new CMD shall be prepared and successfully demonstrated for changes in the source, type or class of a material, the amounts of cementitious materials, increase in water/cementitious ratio, adjustments of greater than ±3 percentage points of fine to total aggregate ratio, or the addition or deletion of admixtures.

Lots and Sublots. *Sampling and testing of the concrete properties will be performed on random cubic meter (cubic yard) of concrete within each subplot. Lots will be defined as 120.0 m³ (150.0 yd³) for each mix design as measured against the plan quantity. Lots will be further subdivided into six sublots of 20.0 m³ (25.0 yd³) each within a lot. Partial sublots of 8.0 m³ (10.0 yd³) or less will be added to the previous subplot. Partial sublots greater than 8.0 m³ (10.0 yd³) will constitute a full subplot. Partial lots of four or more sublots constitute a full lot. If there are three sublots or less in an incomplete lot then the sublots will be included in the previous lot.*

Test Methods and Procedures. The following test methods and procedures apply with exceptions as listed below.

<i>Air Test</i>	<i>AASHTO T 152 or T 196*</i>
<i>Compressive Strength</i>	<i>AASHTO T 22</i>
<i>Flexural Strength</i>	<i>AASHTO T 97</i>
<i>High Pressure Air Content of Hardened PCC</i>	<i>ITM 401</i>
<i>Making and Curing Specimens</i>	<i>AASHTO T 23</i>
<i>Moisture Content, Aggregate</i>	<i>AASHTO T 255</i>
<i>Obtaining and Testing of Drilled Cores</i>	<i>AASHTO T 24</i>
<i>Sampling Fresh Concrete</i>	<i>AASHTO T 141</i>
<i>Sampling Stockpiled Aggregates</i>	<i>ITM 207</i>
<i>Sieve Analysis of Aggregates</i>	<i>AASHTO T 27</i>
<i>Slump</i>	<i>AASHTO T 119</i>
<i>Specific Gravity and</i>	
<i>Absorption, Coarse Aggregate</i>	<i>AASHTO T 85**</i>
<i>Specific Gravity and</i>	
<i>Absorption, Fine Aggregate</i>	<i>AASHTO T 84</i>
<i>Resistance to Chloride Ion Penetration</i>	<i>AASHTO T 277</i>
<i>Unit Mass (Weight)</i>	<i>AASHTO T 121</i>
<i>Water-Cementitious Ratio</i>	<i>ITM 403</i>
* If slag aggregate is used, the method and procedure for the test shall be in accordance with AASHTO T 196.	
** Sections 8.1 and 8.2	

(a) Exceptions to AASHTO T 23. The exceptions to AASHTO T 23 for making and curing specimens in the field shall be as follows.

1. Initial curing of cylinders shall be no less than 16 h or more than 48 h.
2. Non-watertight beam forms (molds) will be permitted.
3. After 24 h, the molded beam specimens shall be taken to the storage location and removed from the molds.
4. Field stored beams will not require 24 ± 4 h immersion in water saturated with calcium hydroxide prior to the time of testing.

(b) Exceptions to AASHTO T 27. The exceptions to AASHTO T 27 for conducting a sieve analysis are in accordance with 904.06.

(c) Exception to AASHTO T 84. The exceptions to AASHTO T 84 for determining SSD specific gravity and absorption for the fine aggregate shall be as follows:

1. The SSD bulk specific gravity shall be reported to the nearest 0.001 and the absorption reported to the nearest 0.01%.

(d) Exception to AASHTO T 85. The exceptions to AASHTO T 85 for determining SSD specific gravity and absorption for the coarse aggregate shall be as follows:

1. The 15 h soak period shall not be eliminated.
2. The in-water mass (weight) shall be determined following the 15 h soaking period prior to determining the SSD mass (weight).
3. The SSD bulk specific gravity shall be reported to the nearest 0.001 and the absorption reported to the nearest 0.01%.

(e) Exceptions to AASHTO T 97. The exceptions to AASHTO T 97 for conducting a flexural test shall be as follows:

1. The beam size shall be measured to the nearest 1.0 mm (1/16 in.).
2. The test result shall be discarded when the break occurs outside the middle third of the beam.

(f) Exceptions to AASHTO T 121. The exceptions to AASHTO T 121 for determining the unit mass (weight) of concrete shall be as follows:

1. Mass (weight) shall be determined to the nearest 0.005 kg (0.01 lb).

(g) Exceptions to AASHTO T 141. The exceptions to AASHTO T 141 for sampling fresh concrete in the field shall be as follows:

1. The entire sample may be obtained from one portion of the load after at least 0.25 m³ (0.25 yd³) of concrete has been discharged.

(h) Exceptions to AASHTO T 152. The exceptions to AASHTO T 152 for determining the air content in PCC shall be as follows:

1. The aggregate correction factor shall be determined in accordance with 6.4.3 except that
2. The aggregate correction factor test shall be re-run for confirmation if the test results for gravel is greater than 0.4% or if the test results for crushed stone is greater than 0.6%.

Testing Facilities and Equipment. An easily accessible means of obtaining concrete samples at the point of placement and transporting the samples from the bridge deck for testing shall be provided. All molds, facilities, and materials necessary to prepare and initially cure quality control and acceptance cylinders shall be provided at the work site.

CONSTRUCTION

General. Construction operations as applicable shall be in accordance with 702, 703 and 704.

Removal and Re-use of Forms. The forms for any portion of the structure shall not be removed until concrete is strong enough to withstand damage. Field operations concerning modified QC/QA superstructure concrete shall be controlled by the Contractor using the wet cure period or beam testing, whichever is longer. Beams shall be in accordance with the requirements of 702.13(h). If cylinders are used to control form removal, the average of all cylinders tested at the age specified shall meet or exceed the required compressive strengths. The number of cylinders to be tested and the requirement of any individual cylinder shall be in accordance with this special provision as specified under Application of Loads.

<u>Concrete Used In</u>	<u>Required Compressive Strength, MPa (psi) Dead Load Only</u>
Girders, Arches, and Similar units.....	17.25 (2500)
Interior Bent or Pier Caps	22.75 (3300)
Girders, Arches, and Similar units.....	17.25 (2500)
Interior Bent or Pier Caps	22.75 (3300)

The cylinders shall be field cured in accordance with AASHTO T 23, under the same conditions as the concrete they represent.

Falsework and Centering. Falsework and arch centering for structural elements shall be in accordance with 702.14.

Application of Loads. The application of loads to modified QC/QA superstructure concrete shall be determined by the wet cure period or test beam requirements of 702.24(a), whichever is longer. Construction activities shall not interfere with wet curing of the bridge deck throughout the period specified. The Contractor may utilize compressive strength of standard specimens to control application of loads to new concrete. The compressive strength shall be the average of the strengths of all cylinders tested at the age specified, with a minimum of two cylinders. If the compressive strength of one or more cylinders in one strength test is below 75% of the required strength, the entire test will be considered as not meeting the required strength. Test cylinders designated for this purpose shall be prepared in accordance with AASHTO T 23, and field cured by the Contractor under the same conditions as the concrete they represent. If standard cylinders

are used, the application of loads to modified QC/QA superstructure concrete shall be in accordance with the following:

- (a) *Light materials and equipment weighing less than 450 kg (1000 lbs) may be carried on bridge decks only after the concrete has been in place at least 24 h, provided wet curing is not interfered with and the surface texture is not damaged. After completion of the wet cure period; vehicles, material, and equipment needed for construction activities, having a gross mass (weight) between 450 and 1800 kg (1000 and 4000 lbs) will be allowed on any span only after the last placed concrete has attained a minimum compressive strength of 17.25 MPa (2500 psi). Loads in excess of above will not be permitted on structures until a compressive strength of 27.50 MPa (4000 psi) has been attained*

No contract time extension will be considered for delays due to additional time necessary to attain specified strengths.

Substandard Work. *General problems and procedures, which cause an obviously substandard product, shall be promptly corrected. The Engineer and Contractor will jointly review the operations to ensure compliance with the QCP. Continuous violations of compliance with the QCP will result in suspension of operations.*

The option to remove and replace the completed superstructure, or to leave it in place will be as directed. Material in this category will be adjudicated as a failed material in accordance with 105.03.

Finishing. *The concrete shall be finished in accordance with 702.21 and 704.05.*

Wet Curing. *After finishing and texturing in accordance with 704.05, the concrete shall be cured in accordance with 704.06 and 702.22, except as modified herein.*

An evaporation retardant shall be applied, in accordance with the manufacturer's recommendation, to the exposed concrete surface immediately after finishing or texturing operations. Reapplication of the retardant shall be performed whenever the surface is disturbed, or when drying of the surface is observed. The evaporation retardant shall be one of the products listed below. A type D certification for the evaporative retardant shall be in accordance with 916 and submitted to the Engineer prior to use

1. *Confilm, manufactured by Master Builders Technologies; 3715 Bargetown Road, Room 214; Louisville, KY 40218*
2. *Sika-Film, manufactured by Sika Corporation; 2930 Switzer Road; Columbus, OH 43219*
3. *Eucobar, Euclid Chemical Company; 19218 Redwood Road; Cleveland, OH 44110*

Evaporative retardant shall be applied to the finished or textured surface of concrete regardless of the evaporation rate. If the evaporation rate exceeds 0.50 kg/m²/h (0.10 lbs/ft²/h) during placement of concrete, fog misting, as recommended by the silica fume manufacturer, shall be initiated prior to the texturing operation. Fog misting shall not be excessive to cause water to wash the fresh concrete surface, or to stand on the surface during floating or troweling operations.

The rate of water evaporation shall be determined during concrete placement in accordance with ACI 308, Section 1.2.1. or the following metric (English) equation:

$$E = 5[(T_c + 18)^{2.5} - r(T_a + 18)^{2.5}][V + 4] \times 10^{-6}$$

$$(E = [T_c^{2.5} - (r \times T_a^{2.5})][1 + 0.4V] \times 10^{-6})$$

where:

E = Evaporation rate, kg/m²/h (lb/ft²/h)

T_c = Concrete temperature, °C (°F)

T_a = Ambient temperature, °C (°F)

r = (RH %)/100

V = Wind velocity, k/h (mi/h)

Measurements of T_a , r , & V shall be obtained on-site and compared for accuracy with readings from the nearest weather station monitored by the National Climatic Data Center. Measurement of T_c shall be determined from the concrete placed.

Concrete in bridge decks or the top surface of reinforced concrete slab bridges shall be wet cured continuously for at least 168 h commencing immediately after the surface is able to support the protective covering without deformation. The wet cure period shall be 240 h for concrete placed in cold weather.

Membrane forming curing compound shall not be applied to concrete in bridge decks or the top surface of reinforced concrete slab bridges.

Surfaces to be cured shall be protected by covering with cotton mats, burlap, or other satisfactory protective material that is kept continuously and thoroughly wet, through the use of soaker hoses, during the curing period. The protective covering shall be suitably anchored. Curbs, walls, handrails, copings, and other surfaces requiring a finish in accordance with 702.21 may have the covering temporarily removed for finishing, but the cover shall be restored as soon as possible. Water application through the soaker hoses shall be discontinued 24 hours before the cure period ends and the protective covering is removed.

Sealing. The concrete surface shall not be sealed.

Acceptance Testing and Approval. The Engineer will randomly select the quantity for sampling sublots of concrete in accordance with ITM 802. The air content, plastic unit mass (weight), and compressive strength tests will be determined for each subplot sample during concrete production. Sublot samples of concrete will be obtained at the work site at the point of placement. When calculations are performed, rounding will be in accordance with 109.01(a). Sublot acceptance test results will be shared with the Contractor in a timely manner.

(a) Slump Test. For any load of concrete, the slump may be visually estimated. If it is suspected that the slump is not within the allowable limits at the point of placement, the Contractor's certified concrete technician will be informed. The truck in question shall discontinue placement until a slump test is conducted to verify compliance. If the slump is outside compliance, the technician shall test for air content and unit mass (weight). The truck shall not resume discharge in the structure until quality control test results substantiate compliance.

(b) Air Content and Unit Mass (Weight) Tests. The air content and unit mass (weight) will be measured once in each subplot sample.

A line parallel to the CMD Linear Equation will be established which represents an increase in water/cementitious ratio to a value of 0.420. An individual subplot having a unit mass (weight), for the air content measured, at or below the value representing the maximum allowable water/cementitious ratio, will be interpreted as excessive water in the concrete. The Contractor's certified concrete technician will be informed and the Contractor shall implement corrective action as detailed in the QCP.

Calculations for air content percentage will be made and reported to the nearest figure in the first decimal place. Calculations for unit mass (weight) will be made and reported to the nearest 1 kg/m^3 (0.1 lb/ft^3).

(c) Compressive Strength Test. Two cylinders will be cast from each subplot sample. Initial curing of cylinders shall be completed by submerging the specimens in water saturated with calcium hydroxide at a temperature range between 16 to 27 °C (60 to 80 °F) for no less than 16 h or more than 48 h. Each cylinder will be tested for 28 day compressive strength and the values averaged to determine the 28 day subplot compressive strength

Calculations for subplot compressive strength will be made to the nearest figure in the second decimal place for MPa units (whole psi unit).

Pay Factors. Quality Assurance Adjustment for each lot will be determined based on pay factors for concrete properties of air content, range of air content, and compressive strength. The formula for calculating the adjustment is as follows:

$$q_i = L_i \times U_m (PF_{ai} + PF_{ri} + PF_{ci} + PF_{sdi} - 4.00)$$

where:

q_i = quality assurance adjustment for individual(i^{th}) lot
 L_i = quantity for i^{th} lot
 U_m = unit price for material, \$125/ m^3 (\$96/ yd^3)
 PF_{ai} = pay factor for air content in i^{th} lot
 PF_{ri} = pay factor for air content range in i^{th} lot
 PF_{ci} = pay factor for compressive strength in i^{th} lot
 PF_{sdi} = pay factor for compressive strength standard deviation in i^{th} lot

The total quality assurance adjustment will be calculated as follows:

$$Q = \sum q_i$$

For lots $i=1$ to n_l

where:

Q = total quality assurance adjustment
 i = individual lot
 n_l = last lot

The pay factors for lot air content, air content range for lot, and lot compressive strength values will be determined as tabulated below:

- a) **Air Content.** The air content value for each lot will be the average of the respective test values of the sublots within the lot. Air content range will be determined as the difference between the highest subplot air content and the lowest subplot air content within the lot. A pay factor will be assessed on lot values for average air content and range of individual air content values within a lot as follows:

1. Lots

Lot Air Content	
Percent, %	Pay Factor PF_{ai}
10.0 or Greater	*
9.9	0.00
9.7 – 9.8	0.25
9.5 – 9.6	0.55
9.3 – 9.4	0.80
9.0 – 9.2	0.90

5.9– 8.9	1.00
5.2 – 5.8	0.95
4.9 – 5.1	0.90
4.8	0.40
4.7	0.00
4.6 or less	*

<i>Air Content Range Within Lot</i>	
<i>Percentage Points</i>	<i>Pay Factor PF_{ri}</i>
4.1 or Greater	0.94
3.6 – 4.0	0.96
3.0 – 3.5	0.98
0.0 – 2.9	1.00

* The material will be adjudicated as a failed material in accordance with 105.03. The concrete may be subject to removal and replacement or left in place with reduced or no payment.

2. Sublots. An individual subplot having an air content of less than 4.0% or more than 10.0%, will be adjudicated as a failed material in accordance with 105.03.

b) **Compressive Strength Pay Factors.** The mean compressive strength for each lot will be the average of the respective test values of the sublots within the lot. The compressive strength standard deviation for a lot will be determined by the following formula:

$$sd = \{ [\sum (x_j - \text{mean})^2] / (n_j - 1) \}^{0.5}$$

Where:

sd = standard deviation of Lot compressive strength
mean = computed compressive strength for Lot
n_s = total # of random samples, or sublots, in Lot
x_j = individual subplot strength result within Lot
j = subplot number from 1 to n

Pay Factors will be assessed on lot values for average and standard deviation of compressive strength within a lot as follows:

1. Lots.

<i>28 Day Lot Compressive Strength</i>	
<i>MPa (psi)</i>	<i>Pay Factor</i> <i>PF_{ci}</i>
<i>31.65 (4590) or above</i>	<i>1.00</i>
<i>31.51 – 31.64 (4570 – 4589)</i>	<i>0.96</i>
<i>31.37 – 31.50 (4550 – 4569)</i>	<i>0.90</i>
<i>31.23 – 31.36 (4530 – 4549)</i>	<i>0.80</i>
<i>31.10 – 31.22 (4510 – 4529)</i>	<i>0.65</i>
<i>30.96 – 31.09 (4490 – 4509)</i>	<i>0.45</i>
<i>30.82 – 30.95 (4470 – 4489)</i>	<i>0.15</i>
<i>30.68 – 30.81 (4450 – 4469)</i>	<i>-0.35</i>
<i>30.67 (4449) or less</i>	<i>*</i>

** The material will be adjudicated as a failed material in accordance with 105.03. The concrete may be subject to removal and replacement or left in place with reduced or no payment.*

<i>Standard Deviation of Strength Within Lot</i>	
<i>MPa (psi)</i>	<i>Pay Factor</i> <i>PF_{sdi}</i>
<i>2.63 (381) or greater</i>	<i>1.00</i>
<i>1.97– 2.62 (286 – 380)</i>	<i>1.05</i>
<i>1.96 (285) or less</i>	<i>1.10</i>

2. Sublots. A subplot test value for concrete, having a compressive strength value less than 27.58 MPa (4000 psi), will be adjudicated as a failed material in accordance 105.03

Appeal Procedures. *If the Contractor does not agree with the air content or compressive strength acceptance test results for a subplot, an appeal may be submitted. The Department will be the final authority regarding acceptance and assessing adjustment points for air content and compressive strength test results for the lot. The final adjustment points will be used in assessing any credit to the contract. An appealed subplot resulting in a failed material will be adjudicated in accordance with 105.03. Appeals shall satisfy the following criteria:*

- (a) *Appeals shall be submitted in writing to the Engineer within five calendar days of receipt of the Engineer's written results for the lot.*
- (b) *The submission shall contain applicable quality control test results that equals or exceeds the number of tests required.*
- (c) *The difference between the acceptance air content test result and the nearest quality control test result shall be at least 0.5 percentage point.*
- (d) *The difference between the acceptance compressive strength test result and the nearest quality control test result shall be at least 0.70 MPa (100 psi).*

Appeals will be adjudicated by evaluation of cores taken by the Contractor. The diameter of the cores shall be 95 or 100 mm (3.75 or 4.00 in.). Appeal coring shall be completed within 30 days of acceptance of the appeal unless traffic restrictions prevent the coring. All core holes shall be filled with PCC within 24 h of drilling. If the Engineer's cores are subsequently damaged, additional coring within a specific subplot or sublots will be the responsibility of the Department.

The Engineer will determine the location of the cores that most closely approximates the appropriate subplot acceptance sample location. Cores shall be obtained in accordance with AASHTO T 24 and in the presence of the Engineer. The Engineer will take immediate possession of the cores.

The Engineer will identify and mark each core as to its location and submit the cores to the Materials and Tests Division for analysis.

(a) Air Content. *Four cores, 95 or 100 mm (3.75 or 4.00 in.) in diameter, shall be taken for each subplot. The hardened concrete air content will be determined and converted to a value representing air content in the plastic concrete in accordance with ITM 401.*

The average air content from the four cores will be determined for the subplot in question. This value will be used to determine all subsequent actions involving the subplot as well as the lot to which it is part.

(b) Compressive Strength. *Four cores, 95 or 100 mm (3.75 or 4.00 in.) in diameter, shall be obtained from each subplot. Each core will be tested for compressive strength in accordance with AASHTO T 24.*

The average core compressive strength will be determined for the subplot in question. This value will be used to determine all subsequent actions involving the subplot as well as the lot to which it is part.

Method of Measurement. *QC/QA superstructure concrete, modified, will be measured by the cubic meter (cubic yard) in accordance with the neat lines or as*

directed. However, no allowance will be made for variations in beam fillet depths, coping depths, or diaphragm depths. Reinforcing steel will be measured in accordance with 703.07. Castings and cast iron pipe will be measured in accordance with 702.27.

Basis of Payment. *The accepted quantities of QC/QA superstructure concrete, modified, will be paid for at the contract unit price by the cubic meter (cubic yard) of concrete, complete in place. Reinforcing steel will be paid for in accordance with 703.08. Castings and cast iron pipe will be paid for in accordance with 702.28.*

Adjustments to the contract payment with respect to air content, air content range, and compressive strength for concrete produced will be included in a quality assurance adjustment pay item. The unit price for this pay item will be one dollar (\$1.00) and the quantity will be in units of dollars. The quantity is the total calculated in accordance with the requirements stated in this special provision under the Pay Factor heading. An extra work order developed in accordance with 109.05 will be prepared to reflect contract adjustments.

Payment will be made under:

Pay Item

***Metric Pay Unit Symbol
(English Pay Unit Symbol)***

<i>QC/QA Superstructure Concrete, Modified.....</i>	<i>m³ (CYS)</i>
<i>Quality Assurance Adjustment</i>	<i>DOL</i>

The cost of preparing a QCP, conducting trial batch demonstrations, performing quality control testing, and similar requirements included herein will not be paid for directly but shall be included in the cost of QC/QA Superstructure Concrete.

The cost of coring and refilling of the bridge deck holes for appeals shall be supplied with no additional payment.

Traffic control for appeal coring shall be supplied with no additional payment.

B.3 Construction Requirements for Purdue University Testing

CONSTRUCTION REQUIREMENTS FOR PURDUE UNIVERSITY TESTING

DESCRIPTION. This work shall consist of installing a concrete foundation and conduit in accordance with these specifications and in reasonably close conformance with the lines, grades, and locations specified herein or as directed by the Engineer. In addition the Contractor shall provide access to personnel from Purdue University for installation and monitoring of instrumentation of various structural elements of the bridge superstructure for the purpose of research. This research was authorized by the Department in October 2003 under JTRP (Joint Transportation Research Board Project), SPR-2911 to study high performance concrete in bridge decks.

MATERIALS. Materials shall be in accordance with the following:

Anchor Bolts.....	913.15(a)7.c.
Concrete, Class A, B, or C.....	702
Ground Wire, Ground Rod, and Connections.....	807.12
PVC Conduit and Fittings.....	913.15(j)2.
ED Deflection Coupling.....	UL Standard 514B

CONSTRUCTION REQUIREMENTS. The Contractor shall install a concrete foundation, which is to support a controller cabinet, and lay conduit in accordance with these specifications. Access shall be provided to personnel from Purdue University during critical construction activities as noted. The Contractor shall cooperate with Purdue University in accordance with 105.07. The Project Engineer will be responsible for contacting Purdue University. The Contractor shall provide the Project Engineer five business days notice for each critical construction activity requiring coordinated work by Purdue University.

1. A controller cabinet foundation shall be installed in accordance with 805.13 and as detailed in the drawing herein. The location of the foundation shall be at Station 21+40, 51.5 feet right of Line 'S-8-L' or as directed by the Engineer. The anchors within the foundation shall be set for the attachment of a Type M controller cabinet, which is to be supplied and installed by Purdue University. The Contractor shall verify the anchor locations with Purdue University, prior to placement of the foundation. The Contractor shall provide grounding to the concrete foundation in accordance with 805.06 and as detailed in the drawing herein. The concrete foundation shall be constructed during Phase I and shall be cast prior to, or concurrently with, the pile cap of Bent No. 3.
2. PVC conduit and fittings shall be installed underground in accordance with 805.12. The conduit shall be of the size, length, and location detailed in the drawing herein. Corrugated flexible PVC conduit may be used in place of rigid

conduit, if approved by the Engineer and Purdue University. The conduit shall run from the PVC elbow beneath the controller cabinet foundation to, and through, Bent No. 3 as detailed in the drawing herein. The deflection coupling shall allow for $\frac{3}{4}$ inch movement and be installed at the backfill/concrete interface of Bent No. 3. The inside surface of the conduit shall be kept clean and a $\frac{1}{4}$ -inch braided nylon pull cord shall be installed along the entire length of conduit. The pull cord shall be accessible from either end and the conduit ends shall be capped. The conduit shall remain capped until Purdue University is ready to pull wires into the conduit.

3. The Contractor shall provide a work platform underneath the bridge deck at a location to be determined by Purdue University. The purpose of the work platform is to allow Purdue University to instrument one of the concrete Bulb-T beams at the approximate midpoint of span B. The work platform shall be approximately 45 square feet in area and shall consist of planking supported along the bottom flanges of the Bulb-T concrete beams. The Contractor shall provide access to the work platform from above by temporarily removing a deck form. Purdue University may approve an alternate work platform proposed by the Contractor, such as a man lift located on US 20 underneath the bridge. The Contractor shall allow Purdue University to initiate the instrumentation of the concrete Bulb-T beam after the beams and deck forms are in place, but prior to the installation of the bottom mat of steel reinforcement. The Contractor shall notify the Project Engineer five business days prior to the installation of the deck forms. Approved use of an alternate work platform may allow adjustment to the required notification and scheduling of the beam instrumentation.
4. During Phase I construction, the Contractor shall provide access to personnel from Purdue University to install PVC conduit and fittings along the coping of the bridge deck as shown on the plans. The Contractor shall notify the Project Engineer five business days prior to the installation of the bottom mat of steel reinforcement. Purdue University shall also be provided access to pull wires through the conduit, and install a data logger in the controller cabinet. The access shall be provided after the bottom mat of steel reinforcement is in place, but before installing the top mat of steel reinforcement and the 591a deck railing bars. After the top mat of steel reinforcement and 591a deck railing bars are in place, the Contractor shall provide access to Purdue University personnel to pull wires to complete the instrumentation of the Bulb-T beam, and begin instrumenting the mats of steel reinforcement and bridge deck concrete with strain gauges and thermocouples. Placement of modified QC/QA superstructure concrete in the bridge deck shall not begin until Purdue University has completed instrumentation of the bridge. Instrumentation and wiring shall not be disturbed. The Contractor shall notify the Engineer of any instrumentation or wiring that is disturbed during construction activities.
5. The Contractor shall notify the Project Engineer five business days before the modified, QC/QA superstructure concrete is placed in each phase of deck

construction. The concrete shall be placed by hand at the locations of each gauge to protect them from damage. Personnel from Purdue University will be on site during casting of the deck to assist in locating the strain gages and thermocouples. The Contractor shall provide access and concrete material to personnel from Purdue University for sampling at point of placement for the purpose of casting specimens during each phase of construction. The specimens will be tested in the laboratory for research only and the amount of concrete will not exceed one cubic yard for each phase of construction.

6. The Contractor shall provide certified first-aid assistance to Purdue University personnel in the event that these services are needed while Purdue University personnel are on the job-site.

Purdue University will be responsible for supplying the materials, equipment, and labor necessary to install the controller cabinet, data logger, and conduit in the bridge deck coping. Purdue University will be responsible for supplying all the materials, equipment and labor for instrumentation of the concrete Bulb-T beam, steel reinforcement, and bridge deck concrete with strain gauges and thermocouples; including all wiring between the instruments and the data logger. It is estimated that Purdue University will require three to five, 8 hour days to complete its work. The work performed by Purdue University can be accomplished concurrently with bridge deck construction activities. The Contractor may propose changes to the specified work or coordination of activities between the Contractor and Purdue University. Such changes will be as approved by the Engineer and Purdue University. The contact for Purdue University is as follows:

Robert J. Frosch, Ph.D., P.E.
Associate Professor of Civil Engineering
School of Civil Engineering
550 Stadium Mall Drive
West Lafayette, IN 47907-2051
Phone: 765-494-5904
Fax: 765-496-1105
Email: frosch@purdue.edu

METHOD OF MEASUREMENT. This work will not be measured, since it is to be paid as a lump sum.

BASIS OF PAYMENT. This work will be paid for with the item "Testing and Monitoring Equipment Purdue University Testing".

Payment will be made under:

Pay Item	Unit
Testing and Monitoring Equipment Purdue University Testing.....	LS

The cost of all materials, equipment, and labor necessary to install the concrete foundation and conduit; and provide access to Purdue University, shall be included in the

cost of the lump sum pay item. No extension of contract time will be allowed for delays caused by Purdue University.

- GENERAL NOTES**
- 1) Foundation shall conform to all minor differences of actual cabinet to be installed.

CONTROLLER CABINET FOUNDATION
TYPE M - MODIFIED

Figure B.1: Traffic Cabinet Foundation Details

Figure B.2: Conduit Installation Details

Appendix C: Concrete Cylinder Test Results

C.1 Compressive Strength Tests

Table C.1: Compressive Strength Tests (Phase I)

Concrete Age (days)	Compressive Strength (psi)			
	Sample 1	Sample 2	Sample 3	Average
7	4655	4715	4774	4715
14	6307	6306	6190	6268
21	6725	7002	Bad Break	6864
28	6815	6942	7040	6932
56	7517	7408	7391	7439

C.2 Splitting Tensile Strength Tests

Table C.2: Splitting Tensile Strength Tests (Phase I)

Concrete Age (days)	Splitting Tensile Strength (psi)			
	Sample 1	Sample 2	Sample 3	Average
14	479	504	482	488
28	485	630	581	565

Appendix D: Instrumentation Details

D.1 Introduction

Strains were calculated from voltage output data recorded by the datalogger in units of millivolts per volt of excitation (mV/V) using the following relationship derived from Measurements Group, Inc., Tech Note TN-507-1 (Measurements Group, 1997):

$$\varepsilon = \frac{(4 \times 10^6) \Delta V}{GF_{adj} [1000 - 2(\Delta V)]}$$

where:

ε = microstrain, $\mu\varepsilon$ (in./in. $\times 10^{-6}$)

ΔV = gage output minus gage zero reading, mV/V

GF_{adj} = gage factor adjusted for all corrections

The gage factors were adjusted for transverse sensitivity as well as for lead wire resistance. The first correction performed was to compensate for the transverse sensitivity effects experienced by the gage during elongation. Measurements Group, Inc., Tech Note TN-5079 (Measurements Group, 1993) discusses the effects of transverse sensitivity.

$$GF_{adj} = \frac{GF}{1 - \nu K_t}$$

where:

GF = Manufacturers' gage factor

ν = Poisson's Ratio of Material (usually 0.285)

K_t = Transverse sensitivity of gage

The resistance, R_L (Ohms), of the lead wires connecting each gage to the datalogger was measured using a digital multimeter after the gages were installed. The gage factor, GF , after correction for transverse sensitivity, for each 350-Ohm strain gage was adjusted to account for the measured lead wire resistance using the following relationship:

$$GF_{adj} = \frac{GF \times 350}{350 + R_L}$$

D.2 Strain Gages

Micro-Measurements CEA-Series strain gages were installed to measure strain in the longitudinal reinforcing steel bars. Table D.1 lists the longitudinal reinforcing bar strain gage characteristics which were obtained from the Measurements Group, Inc. engineering data sheet provided with each gage. A gage was attached to a small piece of reinforcement and placed in the box to see the temperature variation in a loss piece of reinforcement. The strain history for this gage can be seen in Figure D.1.

Table D.1: Engineering Data for Micro-Measurements Strain Gages Attached to Reinforcing Steel

Micro-Measurements CEA-Series Strain Gages	
Gage Type	CEA-06-250UN-350
Option	P2
Resistance in Ohms at 24° C	350 +/- 0.3%
Gage Factor at 24° C	2.105 +/- 0.5%
Transverse Sensitivity at 24° C	(+0.2+/- 0.2%)
Temperature Range	-100° to +350° F
Self-Temperature Compensation	No
Strain Limits (Approximate)	5%

The constantan strain gages included a polyimide film backing and a polyimide encapsulation layer. These gages came with 5 m lead wires attached. The strain gage wire was spliced to the lead wire, with the splice located in the deck.

The process used to affix the gages to the reinforcing steel is described in detail. The basic procedure consisted of the following steps:

1. Surface preparation
2. Gage installation
3. Adhesive curing
4. Splicing of cables
5. Gage protection
6. Strain gage check

Figure D.1: Strain History for Reinforcement “Dummy” Gage

To prepare the surface of the reinforcing steel for gage installation, a belt sander was used to remove the epoxy coating on the bar and remove three of the lugs on one side of the bar. After the reinforcement bar was down to bare metal from the belt sander, the bar was wet sanded with Conditioner A and 220 grit paper to remove the lines left by sander. Once the worst of the lines were removed, the paper was switched to a smoother 400 grit paper to produce a very smooth surface. Subsequent to the surface becoming smooth, the surface was wiped with Neutralizer 5A using cotton swabs. The surface was wiped until the cotton swabs had no residue or signs of particles on it.

Once the surface was smooth and clean, the strain gage was taped to the surface using PCT-2A cellophane tape. The tape was then pulled back from one end exposing the underside of the strain gage. The bottom of the gage was coated with M-Bond 200 Catalyst-C. To adhere the strain gage to the bar, M-Bond 200 adhesive was applied to the surface and the strain gage was firmly pressed into the adhesive for at least 1 minute. After the adhesive had cured, the cellophane tape was removed, and the gage was covered in M-Coat D sealant. Once the M-Coat D had dried, a second coat was applied to ensure adequate coverage. Following this second coat, a layer of M-Coat FB-2 butyl rubber was placed over the gage. Next, a layer of M-Coat FN-2 neoprene rubber was added. As the final protecting agent, a 100% silicone sealant was applied completely covering the gage area.

After each strain gage was installed and protected, a check of each gage was performed. Resistance was measured from the end of the lead wire or shielded cable attached to the gage using a digital multimeter. Allowing for resistance of the attached wire, a reading of slightly over 350 Ohms was considered to be acceptable.

To attach the strain gages to the concrete girders, AE-10 was used to create a surface for which the gage could be attached. The first step in the process was to use a wire bristle brush and water and scrub the concrete surface and remove any loose debris. After this was accomplished, the AE-10 was mixed and spread in an even coat over the location of the gage and allowed to dry over night. Once the epoxy was cured, it was sanded down to create a smooth surface. Subsequent to the surface becoming smooth, the same procedures were followed as with the reinforcing bars to attach the gages. Table D.2 lists the longitudinal reinforcing bar strain gage characteristics which were obtained from the Measurements Group, Inc. engineering data sheet provided with each gage.

Table D.2: Engineering Data for Micro-Measurements Strain Gages Attached to Concrete Girders

Micro-Measurements N2A-Series Strain Gages	
Gage Type	N2A-06-20CBW-350
Option	P
Resistance in Ohms at 24° C	350 +/- 0.5%
Gage Factor at 24° C	2.100 +/- 0.5%
Transverse Sensitivity at 24° C	(+1.5 +/- 0.2%)
Temperature Range	-100° to +200° F
Self-Temperature Compensation	No
Strain Limits (Approximate)	3%

D.3 Concrete Embedment Strain Gages

Micro-Measurements concrete embedment strain gages were installed to measure concrete strain. These gages are composed of a polymer concrete outer body housing a 4 in. sensing grid made of nickel-chromium. The wet concrete conforms to the deformations in the embedded gage and cures to be securely attached to the gage.

At each gage location, a gage was placed between the two mats of reinforcement and securely attached to the top mat using reinforcing tie wire. An effort was made to place the embedded gage directly under the reinforcing bar strain gage. Where this was not possible due to chairs or other obstructions, the embedded gage was placed as close as possible to the reinforcement gage. The embedded strain gages used were designated EGP-5-350 by Vishay Micro-Measurements. Table D.3 lists the concrete embedment strain gage characteristics which were obtained from the Measurements Group, Inc. engineering data sheet provided with each gage. To obtain an understanding of amount of strain measured from these gages due to temperature changes, a gage was cast into a 16x6x6 in. block during the first trial batch mentioned in Chapter 2. The primary drying shrinkage was assumed to be completed before readings were taken for the 90 day investigation. The result of this gage is presented in Figure D.2. It should be noted that the concrete block was placed in the traffic cabinet which held the data acquisition equipment.

Table D.3: Engineering Data for Micro-Measurements Strain Gages

Micro-Measurements EGP-Series Concrete Embedment Strain Gages	
Gage Type	EGP-5-350
Resistance in Ohms at 24° C	350 +/- 0.8%
Gage Factor at 24° C	2.06 +/- 1.0%
Temperature Range	-50° to +135° F
Self-Temperature Compensation	Yes
Strain Limits (Approximate)	0.5%

Figure D.2: Strain History for Embedded “Dummy” Gage

D.4 Thermocouples

Thermocouples were comprised of ANSI Type T thermocouple wire from Omega Engineering, Inc., (Model No. FF-T-24-SLE). This thermocouple wire consisted of two insulated 24 AWG solid wires, one copper and one constantan. This thermocouple wire was rated for a maximum temperature of 392° F with a deviation of 0.7° F at 200.4° F.

D.5 Temperature and Relative Humidity Probe

A Campbell Scientific, Inc CS500 Temperature and Relative Humidity Probe was used to measure ambient air temperature and relative humidity. The CS500 was housed inside a solar radiation shield to protect it from environmental elements and to give a more accurate reading.

D.6 Datalogger System

The datalogger system consisted of Campbell Scientific, Inc. data collection equipment housed inside an aluminum M cabinet located behind the guardrail at the northwest corner of the bridge.

D.6.1 Conduit

The contractor of the bridge installed 3 in. conduit underground from the traffic enclosure foundation to the top of the end bent as required in the contract. Once the

conduit was in place, a reducer was installed on the 3 in. conduit to enable a smaller 2 in. conduit to be attached. The 2 in. conduit was needed to make it possible to run the conduit between the two mats of reinforcing steel placed in the bridge deck. To limit the length of wires ran to the gages, a split was installed in the conduit to allow a portion of the wires to exit the conduit before the termination point (Figure D.3). The conduit was terminated at midspan of the instrumented span on the bridge deck. Once the wires were out of the conduit, they were tied to the top layer of reinforcement using plastic wire ties.

Figure D.3: Splitting of Conduit

Schedule 40 electrical PVC conduits were also installed to protect all cables and wires between the datalogger system and the solar panel, cellular antenna, and the CS500 humidity probe. One 3/4-in. conduit was installed between the solar panel and the traffic cabinet. One 3/4-in. hole was drilled into the aluminum M cabinet. The horizontal portion of the PVC conduits was buried.

D.6.2 Wiring

Shielded cable and double-insulated thermocouple wire were installed between the datalogger system, all gage locations, and thermocouple locations, respectively. Belden 3-conductor 24 AWG shielded instrumentation cable was used to connect all strain gages to the datalogger system.

Strain gage cables and thermocouple wires for instrumentation were pulled through 3-in. diameter PVC conduits from the M cabinet housing the datalogger system to the location of each strain gage and thermocouple lead wire in the bridge deck. A number of spools of cable were transported to the bridge site and placed on a frame built to make the wire pulling easier (Figure D.4). Each spool and the end of the cable or wire coming off the spool were first labeled to identify the appropriate instrument location.

Figure D.4: Frame for Pulling Wires

All strain gage cables and thermocouple wires were bundled and pulled simultaneously through the conduit that extended into the deck. Cable lubricant was added to the bundle of wires as they entered the PVC to reduce friction during pulling. After the bundle was pulled, the end of each cable and thermocouple wire at the datalogger system was labeled before the cable or wire was cut. Once the cables were cut, they were run to the gage locations corresponding to their labels. Figures C.3, C.4, C.5 provide the wiring schematic for Multiplexers 1, 2 and 3, respectively. The gages assigned to Multiplexer 1 are those which experienced periods of lost data due to malfunctions.

MUX 1

Figure D.5: Gage Assignments for Multiplexer 1

MUX 2

Figure D.6: Gage Assignments for Multiplexer 2

MUX 3: Thermocouples

Figure D.7: Gage Assignments for Multiplexer 3

D.6.3 Solar Power

A 20 watt solar panel was securely attached to a 4x4-in. post to provide the necessary power to recharge the 12 volt, 26 Ahr battery used to operate the data acquisition equipment. The solar panel charged the battery through a Campbell Scientific CH100 charger/regulator.

D.6.4 Telephone Service

Cellular phone service was activated to provide remote communication abilities to the system. With this capability, the datalogger program could be accessed and changed, and data stored on the datalogger could be viewed and downloaded using a computer with a data/fax modem.

D.6.5 Modem

A Redwing CDMA cellular modem by Airlink Communications was used to enable communication between a computer and the datalogger. The modem was placed inside the M cabinet. A serial cable connected the modem to a Campbell Scientific (CS) SC932A Interface, and a CS SC12 cable connected the interface to the datalogger. The interface was required with the CR10X datalogger to isolate the electrical system from the datalogger. Isolation from the electrical system protected against ground loop, normal static discharge, and noise through the connection.

On-site communication between computer and datalogger was made at the datalogger without use of the modem. A laptop computer was attached to the SC32B Optically Isolated Interface using a 9-pin to 9-pin computer serial cable. The CS SC12 cable connected the interface to the datalogger.

D.6.6 Data Collection

The program for the datalogger was downloaded from a laptop computer prior to activation of the datalogger. The program was written to record “raw” data only: millivolts (mV) for all strain gages and concrete embedment gages, temperature in degrees Celsius for all thermocouples, and percent relative humidity for the temperature and relative humidity probe. Subsequent calculations to convert the raw data into other units were performed using computer software in a spreadsheet format.

Appendix E: Strain Plots

E.1 Reinforcement Strains in the Longitudinal Direction

Figure E.1: Strain History for Gage RG1

Figure E.2: Strain History for Gage RG2

Figure E.3: Strain History for Gage RG3

Figure E.4: Strain History for Gage RB1

Figure E.5: Strain History for Gage RB2

Figure E.6: Strain History for Gage RB3

Figure E.7: Strain History for Gage RB4

Figure E.8: Strain History for Gage RB5

Figure E.9: Strain History for Gage RB6

Figure E.10: Strain History for Gage RB7

Figure E.11: Strain History for Gage RB8

Figure E.12: Strain History for Gage RB9

Figure E.13: Strain History for Gage RB10

E.2 Concrete Strains in the Longitudinal Direction

Figure E.14: Strain History for Gage EG1

Figure E.15: Strain History for Gage EG2

Figure E.16: Strain History for Gage EG3

Figure E.17: Strain History for Gage EB1

Figure E.18: Strain History for Gage EB2

Figure E.19: Strain History for Gage EB3

Figure E.20: Strain History for Gage EB4

Figure E.21: Strain History for Gage EB5

Figure E.22: Strain History for Gage EB6

Figure E.23: Strain History for Gage EB7

Figure E.24: Strain History for Gage EB8

Figure E.25: Strain History for Gage EB9

Figure E.26: Strain History for Gage EB10

E.3 Free Shrinkage Specimens

Figure E.27: Strain History for Gage FS_Slab_Center

Figure E.28: Strain History for Gage FS_Slab_Offset

Figure E.29: Strain History for Gage FS_6X6

E.4 Restrained Ring Specimens

Figure E.30: Strain History for Gage RR1a

Figure E.31: Strain History for Gage RR1b

Figure E.32: Strain History for Gage RR1c

Figure E.33: Strain History for Gage RR1d

Figure E.34: Strain History for Gage RR2a

Figure E.35: Strain History for Gage RR2b

Figure E.36: Strain History for Gage RR2c

Figure E.37: Strain History for Gage RR2d