High Performance Concrete Bridge Deck Investigation

November 2009

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FOREWORD

In 1993, the Federal Highway Administration (FHWA) initiated a national program to implement the use of high-performance concrete (HPC) in bridges. The program included the construction of demonstration bridges throughout the United States. As a result the State Departments of Transportation started implementing the use of HPC on their bridges. The construction of these bridges has provided a large amount of data on the use of HPC.

Information about the 18 bridges included in the FHWA program plus one bridge in Louisiana was compiled as part of the FHWA Contract DTFH61-00-C-00009 entitled *Compilation and Evaluation of Results from High Performance Concrete Bridge Projects.* A compact disc (CD) containing the compilation was prepared. The CD contains photographs and cross-sectional drawings of the bridges, as well as details about the materials and methods used in construction.

After the bridges had been in service for several years, they were inspected and their performance evaluated relative to the compiled data as part of the FHWA Contract DTFH61-04-C-00029. On this project, a review and analysis of the field data was performed along with that of the data from the CD. Based on these reviews and analyses, parameters of HPC mixture designs were identified that can produce relatively crack free concrete bridge decks.

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16. Abstract
In 1993, the Federal Highway Administration (FHWA) initiated a national program to implement the use of high performance concrete (HPC) in bridges. The program included the construction of demonstration bridges and dissemination of the technology and results at showcase workshops. Eighteen bridges in 13 States were included. In addition to the joint State-FHWA HPC initiative, State Departments of Transportation started implementing the use of HPC on their bridges. These bridges demonstrate practical application of high performance concretes. In addition, construction of these bridges provided opportunities to learn more about the placement and actual behavior of HPC in bridges. Many of the bridges were instrumented to monitor their short- and long-term performance. Concrete material properties were measured for most of the bridges.

Following completion of the demonstration bridges, information about each bridge was collected and compiled into a single source compact disc (CD). After the bridges had been in service for several years, they were inspected and their performance evaluated relative to the compiled data. The measured properties of the HPC used in the bridges showed that the concrete met design specifications. Field surveys on the individual bridge decks, the major focus of this study, found that the material was generally performing well with no indication of any significant deterioration due to material properties. The study demonstrated that HPC can be designed and fabricated in a cost effective and efficient manner to produce durable concrete bridges. Parameters of HPC mixture design have been identified that can produce relatively crack free concrete bridge decks.

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APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

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CHAPTER 1. INTRODUCTION

In 1993, the Federal Highway Administration (FHWA) initiated a national program to implement the use of high performance concrete (HPC) in bridges. The program included the construction of demonstration bridges in each of the FHWA regions and dissemination of the technology and results at showcase workshops. Eighteen bridges in 13 States were included in the national program. In addition to the joint State-FHWA HPC initiative, other States have independently implemented the use of HPC in various bridge elements.

The bridges are located in different climatic regions of the United States and use different types of superstructures. The bridges demonstrate practical application of high performance concretes. In addition, construction of these bridges provided opportunities to learn more about the placement and actual behavior of HPC in bridges. Consequently, many of the bridges were instrumented to monitor their short- and long-term performance. In addition, concrete material properties were measured for most of the bridges.

Information about the 18 bridges included in the FHWA program plus one bridge in Louisiana was compiled as part of FHWA Contract No. DTFH61-00-C-00009 entitled "Compilation and Evaluation of Results from High Performance Concrete Bridge Projects [1,2]." A compact disc (CD) containing the compilation was prepared. The CD contains photographs and cross-sectional drawings of some of the bridges, as well as details about the materials and methods used in construction.

A list of the bridges included in the compilation is given in table 1. A summary of some features of the bridges is given in table 2.

The compilation for each bridge is divided into 12 sections as follows:

1. **DESCRIPTION.** This section contains a summary of the overall bridge features.

2. **BENEFITS OF HPC AND COSTS.** Highlights of why HPC was used in the bridge and provides total cost, cost per ft^2, cost per ft, or any other information that was obtained.

3. **STRUCTURAL DESIGN.** This section presents essential features about the structural design of the bridge.

4. **SPECIFIED ITEMS.** This section includes only items that were required by the HPC Specification. If items were not identified as being specified, the line is left blank.

5. **CONCRETE MATERIALS**. This section lists information obtained before actual construction of the bridges. It represents the information that would normally be submitted for approval of concrete mix proportions plus additional data that were available because of the research component of each project.

6. **CONCRETE MATERIAL PROPERTIES.** This section contains information obtained during the actual construction. It is separated into sections on material properties from quality control (QC) tests and material properties from research tests. Separate sections are provided for each HPC element used in the bridge such as girders and deck.

7. **OTHER RESEARCH DATA.** This section contains research data specifically obtained during the construction of the showcase bridge. The information varies considerably from one compilation to the next depending on the approach and interests of the researchers.

8. **OTHER RELATED RESEARCH**. This section contains other related research information that was usually obtained prior to construction of the bridge.

9. **SOURCES OF DATA.** References of documents used for the compilation are listed. Some of the data were obtained directly from the States and do not appear in the published data. The names of individuals who supplied the data are listed.

10. **DRAWINGS.** This section contains miscellaneous details to clarify the written information.

11. **HPC SPECIFICATIONS.** When available, the special provisions for HPC in the bridge are included.

The compilation does not contain information about the durability performance of the bridge decks and girders after their construction. Since a range of concrete constituent materials and construction procedures were used and the bridges are located in a variety of climates, information was needed concerning the performance of the bridges so that a comparison of their performance could be made.

Table 1. HPC Bridges Included in the Compilation.

State	Bridge Name	Location
Alabama	Highway 199	Highway 199 over Uphapee Creek, Macon County
Colorado	Yale Avenue	Interstate 25 over Yale Avenue, Denver
Georgia	SR 920	SR 920 (Jonesboro Rd) over I-75
Louisiana	Charenton Canal Bridge	LA 87 over Charenton Canal in St. Mary Parish
Nebraska	120th Street	120th Street and Giles Road Bridge, Sarpy County
New Hampshire	Route 104, Bristol	Route 104 over Newfound River, Bristol
New Hampshire	Route 3A, Bristol	Route 3A over Newfound River, Bristol
New Mexico	Rio Puerco	I 40 Westbound Frontage Road over the Rio Puerco
North Carolina	U.S. 401	Northbound U.S. 401 over Neuse River, Wake County
Ohio	U.S. Route 22 near Cambridge	U.S. Route 22 over Crooked Creek at Mile Post 6.57 near Cambridge in Guernsey County
South Dakota	I-29 Northbound	I-29 Northbound over Railroad in Minnehana County, Structure No. 50-181-155
South Dakota	I-29 Southbound	I-29 Southbound over Railroad in Minnehana County, Structure No. 50-180-155
Tennessee	Porter Road	Porter Road over State Route 840, Dickson County
Tennessee	Hickman Road	Hickman Road over State Route 840, Dickson County
Texas	Louetta Road	Louetta Road Overpass, SH 249, Houston
Texas	San Angelo	U.S. Route 67 over North Conch River, U.S. Route 87, and South Orient Railroad, San Angelo
Virginia	Route 40, Brookneal	Route 40 over Falling River, Brookneal in Lynchburg District
Virginia	Virginia Avenue, Richlands	Virginia Avenue over Clinch River, Richlands
Washington	State Route 18	Eastbound lanes of State Route 18 over State Route 516 in King County

Table 2. HPC Bridge Features.

Bridge Name	Total Length, m (ft)	Overall Width, m (ft)	No. of Spans	Crossing	Deck Thickness, mm (in.)	Girder Type
Highway 199	362 (798)	19.5 (43)	7	Creek	178 (7)	BT-54
Yale Avenue	98 (215)	63 (138)	2	Road	292 (11.5)	Box
SR 920	160 (353)	43 (94)	4	Interstate	203 (8)	II, IV
Charenton Canal	166 (365)	21 (47)	5	Canal	203 (8)	III
120th Street	102 (225)	38.6 (85)	3	River	191 (7.5)	NU1100
Route 104, Bristol	30 (65)	26 (58)	1	River	229 (9)	III
Route 3A, Bristol	27 (60)	18 (40)	1	River	229 (9)*	NE 1000
Rio Puerco	133 (293)	22 (48)	3	River	221 (8.7)	BT 1600
U.S. 401	136 (299)	2@21 (47)	4	River	216 (8.5)	IV, III
U.S. Route 22	53 (116.5)	22 (48)	1	River	140 (5.5) w/asphalt	B42-48
I-29 Northbound	78 (172)	19.5 (43)	3	Railroad	229 (9)	II
I-29 Southbound	78 (172)	19.5 (43)	3	Railroad	229 (9)	II
Porter Road	144 (318)	14.5 (32)	2	Road	209.6 (8.25)	BT-72
Hickman Road	132 (291)	14.5 (32)	2	Road	209.6 (8.25)	BT-72
Louetta Road	177 (391)	71-82 NB 81-120 SB	3	Road	184 (7.25)*	U 54
San Angelo	431 (950) EB 435 (958) WB	18 (40) EB 22 (48) WB	8 EB 9 WB	Road, River and Railroad	190.5 (7.5)*	IV
Route 40, Brookneal	145 (320)	20 (44)	4	River	216 (8.5)	IV
Virginia Av. Richlands	67 (148)	18 (40)	2	River	216 (8.5)	III
State Route 18	135 (297)	17 (38)	3	Road	190.5 (7.5)	W74G

* Includes precast panels

BACKGROUND

Chloride ion induced corrosion of reinforcing steel in concrete bridge decks presents a major problem in the United States. Chloride ions are present in deicer salts that are used on roadways and bridges. The chloride ions can reach the concrete-steel interface either through cracks in the concrete material or by diffusing through the concrete pore water. If moisture and oxygen are available at this interface along with chloride ions, corrosion of the reinforcing steel can be initiated. Once the corrosion process begins, expansive corrosion products are produced that can cause additional cracking, spalling, and delamination of the concrete material.

According to the 2009 Report Card for American Infrastructure produced by the American Society of Civil Engineers (ASCE)[(3)], approximately 26 percent of the nation's bridges are either structurally deficient or functionally obsolete. This means that one in four bridges is deficient or obsolete in the nation. ASCE also estimates that a $17 billion annual investment is required over the next 50 years to eliminate all deficiencies as they arise.

Many of the existing structures were built with, and are fast approaching a 50-year design life. Traffic has also increased over this period, increasing the loading, which in some cases may have accelerated the deterioration of these structures. These numbers and circumstances show the critical state of the nation's bridges and the need to design and fabricate longer lasting, more durable structures. The performance of a concrete material is influenced by its material properties as well as the environment and loading that it is exposed to during its service life. It is important that a concrete material is designed and fabricated to withstand the environmental and loading conditions that it will experience during its service life.

Concrete Cracking in Bridge Decks

There are many factors that may contribute to the cracking of concrete in bridge decks. Some of these factors can be related to the material itself, while other factors can be related to the environment and loading conditions that the concrete material is subjected to while in service. The material related factors are accounted for in the design of the concrete material by specifying and using good quality materials and proper mix proportions. The environmental and loading factors are also accounted for in the design of the concrete material by enhancing the material to meet certain durability and strength characteristics. Construction practices may also influence the performance of bridge decks as it relates to cracking. The proper placement, consolidation, and timely curing are imperative to producing a long-lasting, durable structure.

Materials Related Influence on Cracking

During the design of concrete mixtures, it is important to use the proper materials and proportions to produce a durable concrete. These materials include the cement, supplementary cementitious materials, water, aggregate, and chemical admixtures. All of these can influence the cracking of the concrete material.

Concrete experiences volume changes throughout its service life, and one of these types of deformations is shrinkage. The volume changes in concrete due to shrinkage can lead to cracking of the concrete material. The four main types of shrinkage associated with concrete are plastic, autogenous, carbonation, and drying shrinkage. Plastic shrinkage is due to moisture loss from the concrete before the concrete sets. Autogenous shrinkage is associated with the loss of water from the capillary pores due to hydration of the cement. This type of shrinkage tends to increase at higher temperatures and higher cement contents. Carbonation shrinkage is caused by the chemical reaction between various cement hydration products with carbon dioxide present in the air, and is usually limited to the surface of the concrete. Drying shrinkage is the shrinkage associated with the loss of moisture from the hardened concrete. By carefully designing and proportioning concrete mixtures, cracking can be limited due to shrinkage of the concrete.

Cement:

The type and amount of cement can influence the cracking of the concrete material. Typically, cements that produce higher heats of hydration such as Type III cements, tend to increase the probability of cracking in concrete materials (4). An increased amount of cementitious material in a concrete mixture can increase the shrinkage of the mixture, thus increasing the potential for cracking.

Supplementary Cementitious Materials:

Supplementary cementitious materials are used in concrete mixtures to reduce the permeability of the concrete. The use of these materials enhances the long-term performance of the concrete material. Some of these materials include fly ash, ground granulated blast-furnace slag, and silica fume. The use of fly ash and slag reduces the early strength gain; however, the long-term strength gain is generally enhanced (4). Silica fume is a finer material, thus the initial strength gain and heat generated during hydration is increased. This early heat generation makes the material more susceptible to cracking.

Water Content:

The water content in concrete can have an effect on shrinkage and cracking of the material. In general, increased water content can lead to increased shrinkage due to greater loss of moisture during drying of the concrete. The increased shrinkage can lead to increased cracking.

Aggregate:

The quantity and quality of aggregate used in concrete mixtures can influence the shrinkage and cracking of the concrete material (4). In general, aggregates with a higher modulus of elasticity, indicating lower absorption, will exhibit less shrinkage than aggregates possessing lower moduli of elasticity. Also, aggregates with lower absorption values are less prone to freezing and thawing damage. In some cases, the use of higher absorption aggregates, such as lightweight aggregate, has been found to provide internal curing which may reduce cracking.

Chemical Admixtures:

A variety of chemical admixtures can be used in concrete to enhance such properties as air content, slump, and shrinkage reduction. In general, most high performance concrete (HPC) mixtures incorporate lower water-to-cementitious material ratios, which in turn can lead to difficulty in placing the material. Water reducers are typically used to increase the slump of concrete mixtures to aid in placing the material. Air-entraining admixtures are used to create an adequate air-void system to enhance freezing and thawing resistance in concrete materials. Shrinkage reducing admixtures are utilized to reduce the surface tension of the water in the mixture, thus reducing the shrinkage of the concrete.

Environmental and Loading Related Influence on Cracking

During its service life, concrete may be exposed to a variety of environmental and loading conditions that can both influence and, in some cases, accelerate deterioration of the material. One of the major environmental concerns is freezing and thawing cycles. When moisture in the concrete freezes, expansion occurs, this in turn can lead to cracking of the concrete material. Typically, air-entraining admixtures are used to produce an air-void structure in the concrete material that will resist freezing and thawing damage. Another concern involves the presence of sulfates in soils and water. These sulfates can react with hydrated compounds in the cement paste, causing deterioration and cracking in the concrete material. There are different types of portland cement such as Type II (moderate sulfate resistance) and Type V (high sulfate resistance) that can be used to reduce the influence of sulfates on the concrete material. The use of pozzolans and slag cement as partial replacement can also be used to improve the resistance of concrete to sulfate attack. This improved resistance is achieved by reducing the permeability of the concrete material.

Load-related influences on concrete materials can be related to the strength and stiffness of the concrete member. Higher strengths and moduli of elasticity can increase the probability of cracking because the material is more brittle. Traffic loads, especially heavy trucks, can influence the amount of cracking, especially in stiffer structural members.

Construction Practices

Proper construction practices are critical to the concrete material being durable and long lasting. These practices include the placement, consolidation, and curing of the material. With the use of HPC, it is imperative that the material is placed and consolidated properly. Typically, a HPC mixture is designed to have low permeability to provide a more durable material, and it is important to place and consolidate the material properly to ensure that the in-place material meets the design requirements for permeability. If the material is not properly placed and consolidated, problems associated with cracking can occur.

Curing is another vital element in the use of high performance concrete. In many cases, the increased cementitious materials contents and use of pozzolans in HPC can effect the temperature and moisture in the concrete material during hydration and curing. Higher cementitious material contents can increase the heat of hydration of the material, thus increasing the probability of cracking due to stresses generated by excessive heat during curing. The use of pozzolans can also reduce the amount of bleed water in the system, and if left exposed can lead to accelerated drying of the surface. This accelerated drying of the surface can in turn lead to plastic shrinkage cracking of the concrete material.

The control of concrete temperature and moisture loss during curing is critical to producing a durable concrete material. Moisture loss can usually be controlled by providing an adequate moist cure system and keeping the system in place for an adequate time period of at least 7 days.

Thus, in summary, in order to produce a durable, long-lasting concrete material, it is important to design, place, and cure the concrete properly. The mix design phase of the process involves selecting the proper materials and proportions to produce a concrete material that will be able to withstand the various environmental and loading conditions that it will be exposed to during its service life.

The placement phase involves using the proper placement techniques to ensure that the concrete material is placed and consolidated correctly. The placement and consolidation of the concrete materials need to be done in a timely and efficient manner producing a uniform material. The curing phase is critical to the performance of the concrete material. Proper curing will aid in reducing the probability of cracking in the concrete material.

HIGH PERFORMANCE CONCRETE

Definition and Classification

There are a variety of different concrete mixture designs and alternative reinforcing steel systems that have been used to produce longer lasting, more durable structures. One of these systems is high performance concrete (HPC). The definition and classification of HPC has evolved over the last 15-20 years. The Strategic Highway Research Program (SHRP) developed a definition for HPC that took into account water-to-cementitious material (w/cm) ratio, durability, and strength [(5)]. The HPC was defined as having the following characteristics:

- Maximum w/cm ratio of 0.35,
- Minimum durability factor of 80 percent (in accordance with ASTM C666-Procedure A),
- Minimum compressive strength:
  - 20.7 MPa (3,000 psi) at 4 hours (Very Early Strength),
  - 34.5 MPa (5,000 psi) at 24 hours (High Early Strength),
  - 69 MPa (10,000 psi) at 28 days (Very High Strength).

The American Concrete Institute (ACI)[(6)] defines HPC as “concrete meeting special combinations of performance and uniformity requirements that cannot always be achieved routinely using conventional constituents and normal mixing, placing, and curing practices.”

In 1996, Goodspeed et al [7] provided a more in-depth, quantitative view of HPC. They looked at factors related to climate, exposure, and loading. Eight performance characteristics were identified to define and classify high performance concrete:

- Freezing and Thawing Resistance,
- Scaling Resistance,
- Abrasion Resistance,
- Chloride Penetration,
- Compressive Strength,
- Modulus of Elasticity,
- Shrinkage, and
- Creep.

The first four characteristics related to durability, while the second four related to structural design and strength. Each characteristic had an associated standardized test method. The characteristics were further divided into performance grades based on ranges for their associated tests.

Russell and Ozyildirim [8] proposed a revision to the HPC classification. This revision was based on experiences and data collected on bridges built using HPC after the 1996 definition provided by Goodspeed et al. They included the eight performance characteristics used by Goodspeed and added three other characteristics. These three characteristics were alkali-silica reactivity (ASR), sulfate resistance, and workability. All three were related to durability, and workability also affected strength. It was proposed that the limits for each characteristic be updated to reflect collected data and that there be only three performance grades, instead of the maximum of four used by Goodspeed et al, for each characteristic. They also proposed specifying only those characteristics and performance grades that are relevant to the particular application and environment for which the HPC is being designed and fabricated for.

From these definitions and classifications, it can be seen that HPC has evolved from being characterized qualitatively to being characterized quantitatively. The later classifications take into account many durability and strength factors related to experiences and data collected on existing HPC structures. This provides for a better understanding of the material and the affects of the environment and loading that it is subjected to during its service life. By taking into consideration these factors, a longer lasting, more durable concrete material is produced.

Characteristics of High Performance Concrete

High performance concrete is a material that has been enhanced to improve a specific property or properties of the concrete material. The two main properties that are enhanced in HPC are durability and/or strength. In many instances the enhancement of durability properties results in the enhancement of strength properties. One example of this is the use of pozzolans and slag cement as supplementary cementitious materials. The use of these materials reduces the permeability of the concrete material, thus improving durability, and it also produces a higher strength concrete material. This section will present a description of the durability and strength properties presented in the previous section that are enhanced in HPC and the materials and procedures that are used to enhance these properties.

Durability Properties

From the Goodspeed et al and Russell and Ozyildirim classifications of HPC, there are six durability characteristics that can be enhanced in HPC. They are freezing and thawing durability, scaling resistance, abrasion resistance, chloride penetration, alkali-silica reactivity, and sulfate resistance.

Freezing and thawing durability is a major factor for concrete structures that are exposed to freezing and thawing cycles during their service life. As moisture in concrete freezes, expansion occurs, this expansion generates hydraulic pressure in the concrete and subsequent cracking can occur in the concrete material. For reinforced concrete members such as bridge decks, these cracks provide a path for aggressive chemicals and moisture to the reinforcing steel. The migration of these aggressive chemicals and moisture to the reinforcing steel surface can lead to initiation and active corrosion of the reinforcing steel. Corrosion products lead to expansion in the concrete material and consequently to additional cracking, spalling, and delamination.

Typically, air-entraining admixtures are used to improve resistance to freezing and thawing damage. These admixtures create small, closely spaced air bubbles in the concrete material which allow for moisture movement through the cement paste matrix during freezing and thawing cycles. A lower w/cm ratio can also aid in reducing freezing and thawing damage. Another factor that increases freezing and thawing resistance in concrete materials is the proper finishing and curing of the material. This helps create a dense material that will inhibit the ingress of moisture once the material has hardened. It is also beneficial to allow the concrete to properly cure and dry out before it is exposed to freezing and thawing cycles.

Testing for freezing and thawing resistance is done in accordance with ASTM C666 (9) (AASHTO T 161) (10). The specimens are subjected to 300 or more freezing and thawing cycles and are monitored throughout the testing for changes in dynamic modulus, mass, and volume.

Scaling is the disintegration and flaking of the surface of hardened concrete (11). In many cases it is due to frequent freezing and thawing cycles, which cause expansion near the surface of the concrete. Overfinishing can cause excessive moisture loss and reduction of entrained air near the surface of the concrete material. Scaling resistance is improved with entrained air and in some cases by using supplementary cementitious materials. Testing for scaling resistance is performed in accordance with ASTM C672 (12).

Abrasion resistance is related to the strength and aggregate type of a concrete material (11). The hardness of the concrete material contributes to abrasion resistance, with a harder material being more resistant. In general, stronger concrete materials are harder and more resistant to abrasion than weaker concrete materials. The same can be said for aggregate types, a harder aggregate is generally more resistant to abrasion and impact than a softer aggregate. The use of supplementary cementitious materials can increase the abrasion resistance of a concrete by increasing the strength. Silica fume appears to increase the abrasion resistance particularly well by increasing strength (11). Abrasion resistance can be tested in accordance with ASTM C944 (13), which gives an indication of the wear resistance of specimens subjected to a rotating cutter.

As discussed previously, chloride ions present in deicer salts and seawater can migrate and diffuse through reinforced concrete and initiate corrosion of the reinforcing steel. One method of enhancing the chloride penetration resistance of a concrete material is through reducing permeability. This can be achieved in many ways including the use of supplementary cementitious materials, lower w/cm ratio, and proper curing. These three methods reduce the permeability of the concrete material, and thus inhibit the ingress of chloride ions. Another method to reduce the influence of chloride penetration is to increase the cover depth over the reinforcing steel, this increases the distance chloride ions are required to migrate in order to reach the steel surface and initiate corrosion. Testing for chloride penetration is performed in accordance with ASTM C1202 [(14)] (AASHTO T 277) [(15)], which determines the electrical conductance of the concrete material, giving an indication of the resistance to chloride ion penetration.

Alkali-silica reaction (ASR) is the reaction between alkali in cements and reactive silica in aggregate. The reaction forms an expansive gel that causes deterioration of the concrete material. Some of the methods used to reduce the incidence of ASR include the use of supplementary cementitious materials and the use of low alkali (< 0.6 percent) cements. Lower w/cm ratios can also aid in reducing ASR by inhibiting moisture ingress into the concrete material. For HPC, which typically uses pozzolans and slag cement, ASTM C441 [(16)] can be used to determine the effectiveness of these materials in preventing excessive expansion due to alkali-silica reaction.

Sulfate attack is the result of reactions between hydrated compounds in the concrete material and sulfates in soil and water causing deterioration and cracking. Some of the methods used to enhance sulfate resistance include the use of supplementary cementitous materials, lower w/cm ratio, and sulfate resistant cements. The use of supplementary cementitious materials and lower w/cm ratios increases sulfate resistance by reducing the permeability of the concrete material. The use of sulfate-resistant cements, such as Type II (moderate sulfate resistance) and Type V (high sulfate resistance), increase the sulfate resistance by reducing the tricalcium aluminate content of the cements. Sulfate resistance can be tested in accordance with ASTM C1012 [(17)].

Strength Properties

The four strength properties that are typically enhanced in HPC are compressive strength, modulus of elasticity, shrinkage, and creep. Compressive strength can be enhanced in a variety of ways, most commonly through the use of supplementary cementitous materials and lower w/cm ratios. In both cases, a denser matrix is formed providing higher compressive strengths. These two uses are employed in most HPC mixtures as a means of reducing permeability of the concrete material and as a result higher compressive strengths are achieved.

The modulus of elasticity is also enhanced through the use of supplementary cementitous materials and lower w/cm ratios. For both compressive strength and modulus of elasticity, higher does not always mean better. In some cases if they are too high, the resulting material is stiffer and more brittle. This can increase the occurrence of cracking in the concrete material that is exposed to harsh environmental and loading conditions.

Throughout its service life, concrete experiences volume change. The total in-service volume change is the result of applied loads and shrinkage. When loaded, concrete experiences an instantaneous recoverable elastic deformation and a slow inelastic deformation called creep. Creep is composed of two components, basic creep and drying creep. Basic creep is the deformation under constant load without moisture loss or gain, and drying creep is the time-dependent deformation of a drying member under constant load minus the sum of basic creep and shrinkage (18). Deformation of concrete in the absence of applied loads is called shrinkage. Both of these properties can act together to not only to reduce volume change, but to also reduce stresses created by volume change in restrained concrete members. This in turn can reduce the probability of cracking due to volume change in the concrete material. Shrinkage reducing admixtures have also been used to reduce the shrinkage of concrete materials. These properties can be tested in accordance with ASTM C157 (19) (AASHTO T 160) (20) for shrinkage, and ASTM C512 (21) for creep.

In fresh concrete, workability is the ease with which the material can be mixed, placed, and finished (11). It can influence both durability and strength. There are many factors that influence the workability of a concrete material. Some of these factors are related to the type and quantity of materials used such as cementitious material, aggregate, water, concrete temperature, and admixtures. Other factors are related to non-material influences such as method of placement, and environmental conditions during placement.

Some of the properties related to workability include consistency, segregation, and finishability. Consistency is a measure of the ability of the concrete material to flow; slump is measured to indicate the consistency of a concrete material (11). This is an important property for placement of the concrete material. A low-slump concrete is stiff and can be difficult to place, this in turn can lead to improper consolidation of the material that can reduce the strength and durability of the in-place material. However, if the concrete has a high slump and is more fluid, segregation may occur during placement. Segregation is the separation of the aggregate and cement paste. This also can reduce the strength and durability of the in-place material. It is also important to properly finish the in-place concrete material. This includes consolidating the material to produce a uniform, dense in-place material, which if properly done can increase the strength and durability of the material.

The workability of concrete material is measured in accordance with ASTM C143 (22) (AASHTO T 119) (23) and ASTM C1611 (24). The ASTM C143 (AASHTO T 119) procedures measure the slump of the concrete material, while the ASTM C1611 procedure measures the slump flow of the concrete material. The ASTM C1611 procedure is used for concrete materials that are more fluid such as self-consolidating concrete (SCC).

In summary, workability is important in producing a uniform and durable concrete material. It is a key factor in ensuring that the material can be properly placed, consolidated, and finished. If careful attention is not paid to the workability of the material, the strength and durability of the in-place material can be reduced.

There are many factors related to both durability and strength that affect the performance of high performance concrete. The previous discussion of these factors illustrates the importance of each factor, and how each can be enhanced to produce a longer lasting, more durable material.

Advantages of High Performance Concrete

High performance concrete has many advantages related to its material and structural properties. These properties in turn enhance the durability, strength, and constructability of the material, making it an attractive and sometimes necessary alternative to conventional concrete.

One of the major advantages of HPC is the enhanced durability that it provides. This enhanced durability is achieved mainly through the use of pozzolans, slag cement, and air entrainment. The use of pozzolans and slag cement reduce the permeability of HPC, in turn this reduces the infiltration of moisture and aggressive chemicals such as chlorides and sulfates. This is important for reinforced concretes, as the introduction of moisture and aggressive chemicals can initiate corrosion of the reinforcing steel. Air entraining agents are used in HPC to create a suitable air void structure to resist freezing and thawing damage.

Another way that durability is enhanced in HPC is through controlling the temperature of the concrete as it is placed and cured. The use of fly ash and slag cement can reduce the heat generated in fresh concrete. Although the early strength development can be slowed, the reduction in heat generated allows the material structure to form more uniformly, creating a denser, more durable finished product. This also can reduce the occurrence of cracking in the material.

The structural advantages of HPC are related to the enhanced strength that is provided by the material. For bridge structures using HPC, longer spans, wider girder spacing, and shallower girders can be used [(25)]. Because of the higher strength associated with HPC, longer spans can be produced, reducing the number of substructure elements to support the superstructure. The enhanced strength of HPC also allows for wider girder spacing, reducing the number of girders for a bridge structure. And the enhanced strength of HPC allows designers to design shallower girders for bridge structures. This allows for increases in clearance without altering grades. These structural advantages allow for the use of less material, thus reducing the construction costs.

Constructability is enhanced through the enhanced workability associated with high performance concrete. The enhanced workability allows for improvements in placing, consolidating, and finishing of the concrete material.

The use of HPC is advantageous in the sense that it is designed to withstand the effects of the environmental and structural loading conditions that it will be exposed to during its service life. Critical properties are enhanced in HPC to suit the service conditions that it is exposed to, allowing for a longer lasting, more durable concrete material.

Implementation of High Performance Concrete

The program by the Federal Highway Administration (FHWA) that implemented the use of HPC in bridges includes 19 bridge structures in this study. However, there are many other bridge structures that have been constructed using high performance concrete. This section will present the implementation of HPC in some states and some of the bridge structures that have been constructed that are not included as part of this study. It will discuss the performance and some of the lessons learned from these bridge structures.

In 1996, a committee was formed in Maryland to develop a specification for the use of HPC in bridge structures that would achieve a 75-year service life for bridge decks [26]. The specification was implemented in 2000 with the construction of a bridge on MD Route 64 over the CSX railroad in Washington County. The specifications for the bridge deck included a maximum cement content of 326 kg/m^3 (550 lb/yd^3) and a maximum w/cm ratio of 0.45. The limit on cementitious material content was specified to reduce early thermal stress development. Pozzolans were allowed at 35 percent of the total cementitious material content to reduce permeability and inhibit alkali-silica reactivity. The permeability was specified to average no higher than 2000 coulombs at 56 days, with no individual value being greater than 2500 coulombs. The specification also called for the use of a corrosion inhibitor and the use of polypropylene fibers. The compressive strength was specified at 29 MPa (4,200 psi) at 28 days, and the shrinkage was specified at 400 microstrain at 28 days. It is expected that the use of HPC will increase the corrosion initiation period to 50 years. It is also expected that the use of epoxy-coated reinforcement will add an additional 25 years to the service life. From this, it is expected that the bridge deck will not require significant repair for 75 years.

In 2002, Delaware reported that five bridges had been constructed using HPC and another two were in design [27]. The HPC was specified and used to increase durability, reduce permeability, and increase strength. The specification had minimum compressive strength and permeability requirements. The Delaware Department of Transportation (DelDOT) specified a requirement that the producer cast trial batches of the HPC at least 28 days before the HPC was used in the project. One of the problems encountered was the occurrence of random cracking in one of the bridge decks. This was attributed to the use silica fume in some of the mixtures and the contractors having difficulty placing and finishing the concrete material. The curing was also not started early enough, and this may have contributed to the cracking. Consequently, pre-placement meetings were made mandatory for construction personnel, materials personnel, contractors, and concrete suppliers to ensure that everyone involved understood the material and the importance of proper placing, finishing, and curing.

The 17-mile Interstate 15 Reconstruction Project in Utah included the design of 142 bridges [28]. The bridge decks were designed for a 75-year service life and all cast-in-place decks included 5 percent silica fume or an initial overlay. After finishing, the decks were required to have a 7-day wet cure followed by the application of a curing compound. The concrete temperature was required to be at least 10 ºC (50 ºF) for seven days. The specified compressive strength for all cast-in-place concrete was 35 MPa (5,000 psi) at 28 days. Early difficulties were encountered with the workability and finishing of the concrete. This was attributed to the use of silica fume in the mixture, which gave it a sticky consistency. Consequently, the slump of the concrete mixture was increased to accommodate proper placing and finishing. Controlled fogging was also used to increase the humidity over the concrete surface and reduce the moisture loss from evaporation until the deck could be finished and cured. A research project has been initiated to evaluate the cracking that has occurred in some of the bridge decks.

New Jersey now requires the use of HPC in bridge decks on the state highway system [(29)]. The New Jersey Department of Transportation (DOT) initiated research by Rutgers University to develop several baseline mixtures suitable for state highway structures. The research found that mixtures with at least 5 percent silica fume produced concrete with good mechanical and durability properties. The research also found that mixtures with 10-15 percent fly ash also produced good concrete. The New Jersey DOT now requires that HPC mix designs be laboratory fabricated and tested to verify the following: maximum scaling resistance rating of 3, minimum freezing and thawing durability of 80 percent, maximum permeability of 1,000 coulombs at 56 days, and a minimum compressive strength of 37 MPa (5,400 psi) at 28 days. For production concrete, the DOT bases acceptance on a maximum permeability of 2,000 coulombs at 56 days, and a minimum compressive strength of 30 MPA (4,400 psi) at 56 days. If any individual permeability value is greater than 2,000 coulombs at 56 days, the contractor is required to remove the defective concrete or submit a corrective action plan.

In 1994, the New York State Department of Transportation (NYSDOT) developed a HPC mixture in an effort to produce longer lasting, more durable bridge decks [(30)]. The average compressive strength increased by 20 percent over that of the previously used conventional concrete. The field permeability was reduced by 30-50 percent. The cracking was also reduced and the cracks were found to be finer than in the past. Inspections were performed on 84 bridges that were constructed between 1995 and 1998 [(31)]. The inspection found that 49 percent of the bridges had no cracking. There was less cracking and the cracks were shorter and narrower than the past. It was also noted that most of the cracking occurred within two weeks of placement. The results of the inspection and study indicated that the HPC was performing well.

The Great Bend Bridge on Route 11 over the Susquehanna River in Pennsylvania is an example of a successful application of high performance concrete [(32)]. The concrete mixture included Type F fly ash (20 percent replacement) and silica fume (6 percent replacement). Compressive strength and permeability were specified for the bridge deck. The compressive strength was specified at a minimum of 27.6 MPa (4,000 psi) and maximum of 42.7 MPa (6,200 psi) at 28 days. The permeability was specified at a maximum of 1,600 coulombs at 28 days. Follow-up inspection of the bridge deck indicated only a few hairline cracks, and it was estimated that the bridge deck would have a 75-100 year service life.

The Nevada Department of Transportation (NDOT) created a task force in 1999 to develop HPC specifications utilizing local aggregates [(33)]. As a result of research performed by the University of Nevada-Reno, it was found that none of the local aggregate met all of the HPC requirements suggested by FHWA. Therefore, the HPC Task Force selected permeability and modulus of elasticity as requirements for the northern part of the state and only permeability for the southern part of the state. One bridge that was constructed in the northern part of the state used HPC and specified compressive strength, modulus of elasticity, and permeability. It was also decided that shrinkage and creep were important parameters and they were incorporated into the mixture design. The specifications were as follows: minimum compressive strength of 31 MPa (4,500 psi) at 28 days, minimum modulus of elasticity of 3,480 ksi at 28 days, and maximum permeability of 2,000 coulombs at 56 days. It was also decided that shrinkage and creep were important parameters and they were incorporated into the mixture design. The specifications for shrinkage and creep were 700 microstrain at 56 days for shrinkage and 0.50 microstrain/psi at 56 days for creep. The main objective of implementing HPC in Nevada was to increase the service life of structures and reduce life-cycle costs. It was estimated that the implementation of HPC in bridge structures will result in a 35-50 percent increase in service life.

In 1990, Concrete Canada was formed to coordinate and focus on high performance concrete (34). Canada has extreme weather conditions and uses a large amount of deicer salts on their bridge structures. This program was instrumental in developing typical specifications for HPC in Canada. Some of these specifications include cement contents of 350 to 450 kg/m^3 (590 to 760 lb/yd^3), supplementary cementitious material contents of 0-25 percent for fly ash and slag cement and 6.0 to 9.5 percent for silica fume. The w/cm ratio was specified between 0.32 and 0.37, and permeability was specified less than 1,000 coulombs at 28 days. Based on experience, it was found that the rapid chloride permeability test (RCPT) was a reliable index of durability. It was also found that pre-construction and pre-concreting meetings were essential for the successful implementation of high performance concrete. Experience also showed that fog misting was a must after finishing, followed by a wet cure of at least seven days.

An example of success in Canada was the use of HPC in bridges at the Toronto Airport (35). The contractor was responsible for the HPC mixture designs within the following parameters: 8-10 percent silica fume pre-blended, up to 25 percent fly ash or slag cement, minimum compressive strength of 50 MPa (7,250 psi) at 28 days, and maximum permeability of 1,000 coulombs at 56 days. The HPC was to be fog misted until covered with wet burlap and a vapor barrier. It was then required that the HPC be wet cured for seven days. The resulting bridge decks have performed well with no visible signs of cracking.

Related Research

Some of the more recent research has focused on the control of cracking in HPC mixtures. This research has investigated the material influence on cracking as well as factors related to placement and curing.

Nassif et al (36) investigated 16 HPC mixtures that are typically used by the New Jersey DOT. The research focused on reducing shrinkage of the concrete material to reduce the probability of cracking. They found that an increase in coarse aggregate content and a CA/FA ratio greater than 1.48 reduced the rate of shrinkage in HPC mixtures as well as their ultimate shrinkage. They also found that to reduce the probability of cracking in HPC due to shrinkage, the maximum cementitious material content should be limited to 415 kg/m^3 (700 lb/yd^3) and the maximum silica fume content should be limited to 5 percent. They also recommended that the shrinkage be limited to 450 microstrain at 56 days.

Lindquist et al (37) investigated various laboratory mixtures and 14 bridges that used high performance concrete. These mixtures used an optimized combined aggregate gradation and

100 percent portland cement. Lindquist et al found that cracking in HPC mixtures could be limited by the following: maximum cement content of 320 kg/m^3 (540 lb/yd^3), w/c ratio between 0.43 and 0.45, and a maximum slump of 89 mm (3 ½ in.). They also found that increasing the curing period from 7 to 14 days reduced the occurrence of cracking.

Browning et al [(38)] developed specifications for use in the construction of 20 low-cracking, high performance concrete (LCHPC) bridge decks. These specifications were developed from crack surveys from actual constructed decks and laboratory work at the University of Kansas. Portland cement was used in the mixtures, with no supplementary cementitious materials. The maximum cement content was specified at 317 kg/m^3 (535 lb/yd^3) and a maximum w/c ratio of 0.42. The slump range was specified at 38 to 76 mm (1 ½ to 3 in.). The specifications also required that the concrete placement temperature be between 13-21 ºC (55-70 ºF) and air content between 7 to 9 percent. A thorough 14-day wet cure was also required.

Summary

High performance concrete is a specially designed material that offers many advantages. It is a material that can be designed for a multitude of uses to suit the required application. Much effort has been dedicated to the development and implementation of high performance concrete, and development and implementation continues even today with research focusing on improving HPC for the future. There have been many successes and lessons learned that will help enhance the performance of HPC in the future. It has been shown that it is critical for all involved to understand the material from the design phase through the completion of the bridge structure. This knowledge will ensure that HPC can be used to produce durable bridge structures that will meet or exceed their expected service lives.

CHAPTER 2. OBJECTIVE AND SCOPE

The overall objective of the project was to inspect, assess, and evaluate the in-service condition of the 19 HPC bridges decks that were part of the FHWA Contract No. DTFH61-00-C-00009 entitled "Compilation and Evaluation of Results from High Performance Concrete Bridge Projects [1,2]." A limited inspection of the bridge girders was also made.

The Federal Highway Administration (FHWA) retained the services of Professional Service Industries, Inc. (PSI) to conduct the inspection of HPC bridge decks. PSI's scope of services on this project also included a series of tasks and sub-tasks, to collect all available information relevant to the construction of each bridge.

CHAPTER 3. METHODOLGY

The objective and scope were accomplished using the following tasks:

Task A. Collect Relevant Information about the Construction of Each Bridge Deck

The information collected about each bridge included as much of the following as possible:

1. Specified concrete properties including minimum cementitious materials content, minimum percentages of mineral admixtures, maximum aggregate size, slump, air content, compressive strength, chloride permeability, freeze-thaw resistance, deicer scaling resistance, and abrasion resistance.

2. Specified deck concrete construction procedures including placement, finishing, and curing.

3. Approved concrete mix proportions for deck concrete.

4. Measured properties from quality control (QC) tests of production concrete for deck including slump, air content, and compressive strength.

5. Other measured properties of deck concrete including chloride permeability, freeze-thaw resistance, modulus of elasticity, modulus of rupture, creep, and shrinkage.

6. Actual method of deck concrete placement, finishing, and curing.

7. Average daily traffic (ADT) and average daily truck traffic (ADTT).

8. Exposure condition of the bridge including amount and type of deicing chemicals applied since construction.

9. Any performed maintenance.

10. Any inspection reports.

Much of the information listed above was available on the CD and has been summarized in reports on FHWA Project No. DTFH-00-C-00009 [1,2].

Task B. Perform an Inspection of Each Bridge

In cooperation with the FHWA and State DOTs, an inspection of each bridge was made. Initial contact with each State was made by the FHWA to determine the willingness of the State to cooperate in the inspection and to provide traffic control as necessary. Inspection of each bridge included the following:

1. Visual inspection of the top surface of the deck to identify locations of cracks, areas of scaling, freeze-thaw damage, abrasion damage, or any other deterioration. Cracks were documented according to orientation as transverse, diagonal or longitudinal. Transverse cracks occur perpendicular to the centerline of the roadway, diagonal cracks occur at an angle other than 90 degrees to the centerline of the roadway, and longitudinal cracks occur parallel to the centerline of the roadway.

2. Determination of maximum crack widths in each span using a clear comparator card having lines of specified widths.

3. Visual inspection of the underside of the deck, where practical and economically feasible, to identify cracks and any areas of deterioration.

4. Visual inspection of the girders, where practical and economically feasible, to identify any areas of deterioration.

5. Photograph any areas of significant deterioration.

6. Preparation of drawings identifying locations of cracks, locations of crack width measurements, and areas of deterioration. Types of deterioration were identified using the definitions in ACI 201.1R [39]. Individual craze cracks, D-cracks, and pattern cracks were not shown, but areas where those cracks occur were identified.

7. Obtain concrete cores for subsequent evaluation by the FHWA. For bridge decks that show no or limited amounts of deterioration, core locations were selected to represent un-deteriorated concrete. For bridge decks that showed areas of significant deterioration, core locations were selected to represent both deteriorated and un-deteriorated concrete. All core locations and core hole repair procedures were subject to approval by the State DOT. The actual number of cores from each bridge varied depending on observed conditions and size of the bridge.

   For bridge decks exposed to deicing salts or salt water, the intent was to use the cores to determine chloride penetration profiles. The petrographic analysis was performed on cores from the bridge decks; however, not all information was obtained such as w/cm ratio, cementitious materials content, and air content. Chloride penetration profile tests were also not performed on the cores.

   Unless State DOTs were willing to provide vehicles for access to the underside of the bridges, the visual inspection of the underside of the decks and girders was accomplished from ground level with the aid of binoculars.

Task C. Evaluate Information

Information collected in tasks 1 and 2 together with results of testing and examination of the concrete cores by the FHWA were evaluated to identify possible cause or causes for any observed distress. For concrete decks with little or no deterioration, factors contributing to the good performance were identified whenever possible. The goal of this task was to identify practices that were successful and those that should be improved or avoided.

Factors that could contribute to concrete performance include specified concrete strength; actual concrete strength and modulus of elasticity; actual concrete tensile strength; plastic shrinkage; drying shrinkage; autogenous shrinkage; concrete creep; restraint to temperature changes and shrinkage; placement, finishing, and curing practices; cementitious materials content; constituent materials; contractor experience; appropriate specifications; quality control; and exposure conditions at the bridge site.

Comparisons between different construction practices were made where it was appropriate to do so.

Task D. Document Information

This report was prepared to document the collected information, results of the inspection, and evaluation of the information. In the next chapter of this report, a synthesis of individual bridge reports is provided. The following chapter then includes a discussion of the results with respect to the structural systems of the bridges, concrete constituent materials and properties, environmental conditions, and construction practices.

CHAPTER 4. OBSERVATIONS ABOUT INDIVIDUAL BRIDGES

This section of the report includes a summary description of each of the 19 bridges. The full report for each bridge is available in an appendix to this report.

Uphapee Creek Bridge, Alabama Highway 199 (Macon County, Alabama)

The Uphapee Creek Bridge on Alabama Highway 199 in Macon County, Alabama (see figure 1), is one of the first high performance concrete (HPC) bridges built in Alabama. It replaced a bridge built in the 1940's that had suffered from streambed scour resulting from sand and gravel mining upstream. The bridge carries heavily loaded gravel and loading trucks traffic. After the completion of the HPC bridge project, the Uphapee Creek Bridge opened to traffic in April 2000.

Figure 1. Photo. Uphapee Creek Bridge, Alabama Highway 199 (Macon County, Alabama).

The Uphapee Creek Bridge has seven spans on both northbound and southbound lanes. The overall length of the bridge is 243 m (798 ft). The clear width of the bridge is 12.2 m (40 ft), carrying four lanes of traffic with shoulders. The overall length of each span is 34.8 m (114 ft), and the length between the centerlines of the bearing is 34.2 m (112.25 ft).

The Uphapee Creek Bridge has a deck thickness of 178 mm (7 in.) HPC was used on all girders and cast-in-place deck in the Uphapee Creek Bridge. On the same project, within 1.61 km (1 mile) of the Uphapee Creek Bridge, the Uphapee Creek Relief Bridge was constructed utilizing HPC only for the cast-in-place concrete.

There are five AASHTO BT-54 girders per span spaced at 2.7 m (8.75 ft) in the Uphapee Creek Bridge. Typical bridge girders designed by Alabama Department of Transportation (ALDOT) are based on 28 MPa (4,000 psi) at release and 35 MPa (5,000 psi) at 28 days, with the prestressing force provided by 12.7-mm (0.5-in.) -diameter 7-wire strand. The HPC girders utilize the 15-mm (0.6-in.) -diameter strand, which allows a higher prestressing force to be applied. The #7 crushed limestone was allowed in the prestressed concrete girders for the first time in ALDOT projects. Compressive strength of the girder was specified as 55 MPa (8,000 psi) at release and 70 MPa (10,000 psi) at 28 days. The use of HPC enabled the bridge to be designed with one less line of girders than would be required if regular concrete was used.

The specified compressive strength of the cast-in-place concrete was 41 MPa (6,000 psi). Design consideration for the concrete members was based on a compressive strength of 28 MPa (4,000 psi). While the higher strength of the cast-in-place concrete was not fully utilized, HPC was specified to provide enhanced performance and durability. ALDOT conducted the Uphapee Creek Bridge project in cooperation with Auburn University.

The visual inspection of the bridge decks was performed about two years after the bridge opened to traffic. A total of 121 transverse, diagonal, and longitudinal cracks were recorded on the bridge with a combined total crack length of 528 m (1,732 ft) over a bridge deck area of 2,969 m^2 (31,920 ft^2). All cracks on the bridge were hairline cracks with a width of less than 0.08 mm (0.031 in.). No major distress was observed in the bridge survey.

With respect to the types of cracking, 108 transverse, 8 diagonal, and 5 longitudinal cracks were observed. The total crack length for the transverse, diagonal, and longitudinal cracks was 510 m (1,673.5 ft), 9.2 m (30.0 ft), and 8.7 m (28.5 ft), respectively. This yielded crack densities of 0.172 m/m^2 (0.052 ft/ft^2) transverse, 0.003 m/m^2 (0.001 ft/ft^2) diagonal, and 0.003 m/m^2 (0.001 ft/ft^2) longitudinal. The crack density for the entire deck including all transverse, diagonal, and longitudinal cracks was 0.178 m/m^2 (0.053 ft/ft^2).

In general, the work on the Uphapee Creek Bridge shows that the use of HPC provides significantly higher strength that can lead to more efficient designs and improved durability.

Interstate 25 Bridge over East Yale Avenue (Denver, Colorado)

The I-25 Bridge over East Yale Avenue in Denver, Colorado is a two-span bridge that carries Interstate 25 over Yale Avenue (see figure 2). HPC was used in the construction of box beams, bridge deck, and substructure. The new two-span HPC Bridge replaced a four-span bridge. The total length of the bridge is 65.5 m (215 ft) and the two spans are 34.5 (112 ft) and 30 m (98 ft) long, respectively. The 138-ft (42-m) -wide bridge was built in phases to permit traffic flow in both directions during construction. The bridge has a 175-mm (7-in.) -thick cast-in-place deck. HPC was used in the construction of the precast prestressed side-by-side box girders that were used in the new bridge. The HPC, with specified compressive strength of 69 MPa (10,000 psi), enabled the superstructure to attain a high span-to-depth ratio. This allowed longer spans while maintaining a shallow superstructure depth.

Figure 2. Photo. Interstate 25 Bridge over East Yale Avenue (Denver, Colorado).

The prestressed concrete box girders are 1700 mm (67 in.) wide and 750 mm (30 in.) deep. Prestressing strand, 15.2 mm (0.6 in.) in diameter and 51-mm (2-in.) center-to-center spacing, was used in the girders.

The replacement bridge for Interstate 25 over Yale Avenue in Denver, Colorado, is an excellent example of using HPC to meet the demands of urban bridge replacement. Construction on this project began in November 1996 and was completed in June 1998.

The visual inspection of the bridge decks as part of our study was performed about 9 years after the construction. The bridge deck was covered by an asphalt overlay of about 76- to 102-mm (3- to 4-in.) thickness. No cracks were visible on the asphalt pavement. There was a pothole in the inner northbound lane close to the parapet, adjacent to the expansion joint. There were some hair-size cracks on the parapet, but no significant damage was observed.

Jonesboro Road Bridge over Interstate 75 (Atlanta, Georgia)

The Jonesboro Road Bridge over I-75 on State Route 920, located in Henry County, south of Atlanta, is the first HPC bridge built in Georgia (see figure 3). It replaced a steel girder bridge carrying Jonesboro Road, a route connecting Lovejoy, Georgia to the west and McDonough, Georgia to the east. All girders and cast-in-place deck were fabricated using HPC. The bridge has four spans on both eastbound and westbound lanes, with lengths of 16.25, 38.75, 38.75, and 13.75 m (54.4, 127.1, 127.2, and 41.7 ft). The clear width of the bridge is 27.4 m (90 ft), carrying five lanes of traffic with bike lanes and shoulders. The bridge has a skew that varies from 27 to 31 degrees to accommodate a horizontal curve. The Jonesboro Road Bridge was constructed in two stages to handle traffic during construction. After the completion of the first stage, the bridge opened to traffic in February 2002.

Figure 3. Photo. Jonesboro Road Bridge over Interstate 75 (Atlanta, Georgia).

The Jonesboro Road Bridge was designed in accordance with the AASHTO Standard Specifications for Highway Bridges (1996) using MS 18 and/or military design live load. Each of the four spans was simply-supported with 13 HPC girders made with design strengths of 70 MPa (10,280 psi). AASHTO Type IV girders were used for the 38.75-m (127-ft) -long spans and AASHTO Type II girders were used for the 16.5-m and 12.8-m (54-ft and 42-ft) -long shorter spans. The prestressing strands were 15.2 mm (0.6 in.) in diameter. Concrete diaphragms were used at mid-span locations for spans 1 and 4. Spans 2 and 3 had diaphragms at 1/3 span lengths. The use of 38.75-m (127-ft) -long AASHTO Type IV beams minimized the overall depth of the superstructure. Beam spacing is 2.31 m (7.60 ft).

The Jonesboro Road Bridge has an 205-mm (8-in.) -thick cast-in-place composite bridge deck. The deck was formed with stay-in-place (SIP) galvanized steel deck forms that were connected to the girders with welded shear connectors. The cast-in-place concrete was reinforced with epoxy-coated reinforcement. The top reinforcing mat had a specified cover of 70 mm (2.75 in.) while the bottom mat had a specified cover of 25.4 mm (1 in.) above the metal decking. A cast-in-place normal strength 24 MPa (3,500 psi) concrete barrier was constructed on each side of the bridge. The specified compressive strength for the deck concrete was 50 MPa (7,250 psi) at 56 days.

The visual inspection of the bridge decks was performed about 2 years after the bridge opened to traffic. A total of 91 transverse and diagonal cracks were recorded on the bridge with a combined total crack length of 191.0 m (626.2 ft) over a bridge deck area of 2,970 m^2 (31,937.6 ft^2).

However, about 89 percent of the cracks on the two bridges were hairline cracks with a width of less than 0.8 mm (0.031 in.). The remaining 11 percent of the cracks were classified as fine cracks with widths in the range of 0.8 to 1.6 mm (0.031 to 0.063 in.). No major distress was observed in the bridge survey.

With respect to the types of cracking, 61 transverse, 30 diagonal, and 0 longitudinal cracks were observed. The total crack length for the transverse, diagonal, and longitudinal cracks was 125.8 m (412.4 ft), 65.2 (213.8 ft), and 0.0 m (0.0 ft), respectively. This yielded crack densities of 0.042 m/m^2 (0.013 ft/ft^2) transverse, 0.022 m/m^2 (0.007 ft/ft^2) diagonal, and 0.000 m/m^2 (0.000 ft/ft^2) longitudinal. The crack density for the entire deck including all transverse, diagonal, and longitudinal cracks was 0.064 m/m^2 (0.020 ft/ft^2).

In general, the work on the Jonesboro Road Bridge shows that the use of HPC provides significantly higher strength that can lead to more efficient designs and improved durability.

Charenton Canal Bridge (Charenton, Louisiana)

The Charenton Canal Bridge on LA 87 in St. Mary Parish is the first HPC bridge built in Louisiana (see figure 4). HPC was used in all structural components. The bridge is 111 m (365 ft) long and it replaced a 55-year-old cast-in-place concrete bridge. Clear width of the bridge is 14.2 m (46.5 ft). It consists of two 3.66-m (12-ft) -wide lanes, one 3.66-m (12-ft) -wide shoulder on the westbound side and one 2.44-m (8-ft) -wide shoulder on the eastbound side. The Charenton Canal Bridge opened to traffic on November 4, 1999.

Figure 4. Photo. Charenton Canal Bridge (Charenton, Louisiana).

The Charenton Canal Bridge has five spans with an average length of 22.3 m (73 ft). Each span consists of five Type III AASHTO girders made of precast, prestressed HPC. The girders are evenly spaced at 3.1 m (10 ft) centers and support the cast-in-place concrete deck. The substructure of the bridge consists of cast-in-place concrete bent caps supported on 610- and 762-mm (24- and 30-in.) -square precast, prestressed concrete piles. The use of HPC enabled the bridge to be designed with one less line of girders than would be required if regular 41 MPa (6,000 psi) concrete was used.

The deck of the Charenton Canal Bridge is 203-mm (8-in.) -thick cast-in-place reinforced concrete. The main reinforcement perpendicular to the supporting prestressed concrete girders consists of truss bars measuring 19 mm (¾ in.) in diameter and top and bottom straight bars measuring 13 mm (½ in.) in diameter. Longitudinal deck reinforcing steel included 13-mm (½-in.) -diameter top and bottom bars. Negative moment continuity for live loads over the piers was provided by the longitudinal reinforcing steel in the deck. No reinforcement was provided to resist a positive moment over the piers. Diaphragms were provided at the end bents, the piers, and the mid-spans.

The visual inspection of the bridge deck was performed about 4 years after the bridge opened to traffic. The eastbound and westbound lanes are exhibiting a comparable magnitude and pattern of cracking. A total of 46 transverse cracks were recorded on the bridge with a combined total crack length of 57.2 m (187.4 ft) over a bridge deck area of 1,494 m^2 (16,060 ft^2). However, all these cracks were hairline cracks with width less than 0.4 mm (0.016 in.). There were no diagonal or longitudinal cracks observed. No major distress was observed in our bridge survey.

The crack density for the entire deck including all transverse, diagonal, and longitudinal cracks was 0.038 m/m^2 (0.012 ft/ft^2). Though the number of transverse crack counts for eastbound lanes (33 cracks) is more than that for the westbound lanes (13 cracks), the crack densities on eastbound and westbound lanes appear to be similar (i.e., 0.039 m/m^2 (0.013 ft/ft^2) for the eastbound lanes and 0.031 m/m^2 (0.010 ft/ft^2) for the westbound lanes).

Compared to other spans in the bridge, the crack count in span 1 is greater on both eastbound and westbound lanes. A higher crack density is calculated. Span 1 ends along the skew. Some of these cracks were exhibiting spalling due to breaking of the edges. The layout of the cast-in-place deck at span ends may partly be attributed to the development and widening of these cracks. In addition, the structural system of the Charenton Canal Bridge is flexible compared to conventional bridges considering the wider beam spacing and longer span length. This relatively flexible structural system might have contributed to the development and widening of some cracks.

It is noted that relatively large numbers of short-length transverse cracks were observed in span 4 eastbound lanes and span 5 westbound lanes. Settlement of foundation supporting the eastern end of the westbound bridge piers may contribute to the observed transverse cracks.

In general, the work on the Charenton Canal Bridge shows that HPC designs provide significantly higher strength that can lead to more efficient designs requiring fewer piers and, more important, improved durability. The HPC bridge components have a 56-day permeability of 1,079 coulombs in accordance with the mix design. Its ability to resist chlorides and protect steel reinforcement from corrosion will reduce maintenance costs during the life span. A 75- to 100-year service life instead of the normal 50-year service life is anticipated.

120th Street and Giles Road Bridge near Omaha, Nebraska

The 120th Street and Giles Road Bridge in Sarpy County, near Omaha, Nebraska is the first HPC bridge built by the Nebraska Department of Roads (see figure 5). HPC was used in girders and bridge deck. The bridge was built in the summer of 1995 and opened to traffic in July 1996.

Figure 5. Photo. 120th Street and Giles Road Bridge near Omaha, Nebraska.

The 120th Street and Giles Road Bridge consists of three equal 22.9-m (75-ft) -long spans. Total length of the bridge is 68.6 m (225 ft). It utilizes seven lines of NU1100 (1100-mm high) pretensioned concrete girders. Clear width of the bridge is 25.8 m (82 ft). The girders were pretensioned with thirty or thirty-four (depending on the span) 12.7-mm (0.5-in.) -diameter strands at 50.8-mm (2-in.) center-to-center spacing. The cast-in-place deck has a thickness of 190.5 mm (7½ in.). The HPC bridge used NU1100 simple-span girders with negative-moment reinforcement in the deck.

Nebraska uses deicing salts and is in a region of high freeze/thaw cycles; therefore, the focus was to specify a durable deck concrete. The compressive strength specified for the concrete girders was 83 MPa (12,000 psi) at 56 days. Compressive strength of 55 MPa (8,000 psi) at 56 days and a chloride permeability of less than 1,800 coulombs at 56 days were specified for the bridge deck concrete. Fly ash was used in the deck concrete to meet the chloride permeability requirement. The specified strengths for the girders and deck were intentionally higher than required by design as part of the implementation strategy. The water-to-cementitious material ratio for the girders was specified as less than 0.28.

The visual inspection of the bridge deck was performed about 8 years after the bridge was opened to traffic. A total of 259 cracks were recorded during the visual survey of the bridge deck. The sum of crack lengths was 507.7 m (1,664.5 ft) over a bridge deck area of 1,716 m^2 (18,450 ft^2).

With respect to the types of cracking, 170 transverse, 64 diagonal, and 25 longitudinal cracks were observed. The total crack length for the transverse, diagonal, and longitudinal cracks was 330.2 m, 106.4 m, and 71.1 m (1,082.5 ft, 349.0 ft, and 233.0 ft), respectively. This yielded crack densities of 0.192 m/m^2 (0.059 ft/ft^2) transverse, 0.062 m/m^2 (0.019 ft/ft^2) diagonal, and 0.041 m/m^2 (0.013 ft/ft^2) longitudinal. The crack density for the entire deck including all transverse, diagonal, and longitudinal cracks was 0.296 m/m^2 (0.090 ft/ft^2).

All cracks measured are hairline cracks with a width of less than 0.8 mm (0.031 in.). The relatively flexible bridge structural system, combined with the heavy ADT on the bridge, might have contributed to the development of some of the cracks.

In general, the top surface of 120th Street and Giles Road Bridge was in good condition, with only hairline cracks found. It shows that the use of HPC provides significantly higher strength that can lead to more efficient designs and improved durability. The Sarpy County project has demonstrated that HPC can be mixed, transported, placed, finished, and cured with relative ease.

The Route 104 Bridge (Bristol, New Hampshire)

The Route 104 Bridge over the Newfound River in Bristol, New Hampshire, was the first HPC bridge project built in New Hampshire (see figure 6). It was completed in summer 1996 and opened to traffic thereafter. HPC was used for the girders and the cast-in-place deck.

Figure 6. Photo. The Route 104 Bridge (Bristol, New Hampshire).

The Route 104 Bridge is a simple-span structure about 19.8 m (65 ft) long. The clear width of the deck is 17.5 m (57.5 ft), including two through-traffic lanes, a shoulder, and a right-turn lane. The 229-mm (9-in.) -thick cast-in-place deck is supported by five prestressed concrete Type III AASHTO I-girders at 3.8 m (12.5 ft) on center. The specified concrete compressive strengths were 45 MPa (6,500 psi) at transfer and 55 MPa (8,000 psi) at 28 days. The deck concrete was specified to have a strength of 41 MPa (6,000 psi) at 28 days.

Researchers from University of New Hampshire performed material testing, bridge instrumentation, and bridge monitoring throughout this project. It was reported that several inspections have been conducted. Until year 2000, only some microscopic longitudinal flexural cracks over the girder lines were observed, but no transverse or shrinkage cracks were found. Also, there was no scaling and no freeze-thaw damage.

The visual inspection of the bridge deck was performed about 8 years after the bridge opened to traffic. Only two longitudinal cracks were recorded on the bridge with a combined total crack length of 3.1 m (10 ft) over a bridge deck area of 299 m^2 (3,217.5 ft^2). No transverse or diagonal cracks were observed. Crack density (total crack length / deck area) for the eastbound and westbound lanes combined was calculated to be 0.010 m/m^2 (0.003 ft/ft^2). All cracks on the bridge were hairline cracks with a width of less than 0.8 mm (0.031 in.). No major distress was observed in the bridge survey. Compared to data reported by the University of New Hampshire, which mentioned microscopic longitudinal cracks, it is believed that more cracks have not occurred in the Route 104 Bridge deck. Considering the heavy ADT on the bridge, the Route 104 Bridge was in excellent condition. HPC designs provide significantly higher strength that can lead to more efficient designs and improved durability.

The Route 3A Bridge (Bristol, New Hampshire)

Following the success of the Route 104 Bridge in Bristol, NHDOT decided to construct another HPC bridge—the Route 3A Bridge over the Newfound River in Bristol, New Hampshire, about 1 mile away from the Route 104 Bridge. HPC was used for the girders, the precast, prestressed deck panels, and the cast-in-place deck in the Route 3A Bridge (see figure 7). The Route 3A Bridge opened to traffic on June 25, 1999.

Figure 7. Photo. The Route 3A Bridge (Bristol, New Hampshire).

The Route 3A Bridge is a simple-span structure about 18.3 m (60 ft) long. There are two traffic lanes and two shoulders for a clear deck width of 9.1 m (31.5 ft). The superstructure contains four New England Bulb-Tee (NEBT) prestressed concrete girders, spaced at 3.5 m (11.5 ft) apart on center. The HPC girders also contain 15-mm (0.6-in.) -diameter low-relaxation prestressing strands. The use of HPC allowed the designers to reduce the number of girders from five to four, resulting in substantial cost savings. The deck of Route 3A Bridge is composed of twenty-one 89-mm (3.5-in.) -thick precast prestressed deck panels covered with 140 mm (5.5 in.) of cast-in-place concrete.

Researchers from University of New Hampshire performed material testing, bridge instrumentation, and bridge monitoring throughout this project. It was reported that as of Fall 2001, five longitudinal cracks were observed. Four of these cracks were located at the ends of the bridge above the abutments. One crack was located toward mid-span.

The visual inspection of the bridge deck was performed about 2 ½ years after the bridge was inspected by the researchers at University of New Hampshire. A total of seven cracks were recorded during visual survey of the bridge deck. Two longitudinal cracks on the bridge were in the approach slabs and had a crack width of 0.5 mm (0.02 in.). Compared to data reported by the University of New Hampshire, which mentioned five longitudinal cracks, it is suspected that some hairline cracks may have gone through the self-healing process and became invisible. However, it should also be noted that the bridge inspection was performed on a raining day. It is possible that smaller cracks may not be visible in such weather condition. In addition, five transverse cracks were reported from our inspection.

The longitudinal cracks at span ends in the approach slabs may be attributed to the different support conditions. The relatively flexible bridge structural system combined with the heavy ADT on the bridge might have contributed to the development and widening of some cracks.

With respect to the types of cracking on the bridge deck, five transverse, zero diagonal, and two longitudinal cracks were observed. The total crack length for the transverse cracks was 5.6 m (18.5 ft). This yielded a crack density of 0.032 m/m^2 (0.001 ft/ft^2).

In general, the top surface of Route 3A Bridge was in excellent condition, with only very limited hairline cracks found, showing that the use of HPC provided significantly higher strength that can lead to more efficient designs and improved durability.

Old Route 66 Bridge over Rio Puerco, New Mexico

Old Route 66 Bridge over Rio Puerco, west of Albuquerque, New Mexico, was the first HPC bridge project by the New Mexico Highway and Transportation Department (see figure 8). The purpose of the project was to establish the viability of HPC in New Mexico. HPC was used throughout the superstructure. The Rio Puerco Bridge was completed and opened to traffic in December 2000.

Figure 8. Photo. Old Route 66 Bridge over Rio Puerco, New Mexico.

The Rio Puerco Bridge has three spans of 29.3, 30.8, and 29.3 m (96.1, 101.1, and 96.1 ft), respectively. Each span consists of four 1.6-m (63-in.) -deep bulb-tee beams spaced at 3.8 m (12.6 ft) centers. The prestressed concrete beams had specified concrete compressive strengths of 48 MPa (7,000 psi) at release and 69 MPa (10,000 psi) at 56 days. The specified strength for the deck concrete was 41 MPa (6,000 psi) at 28 days with a mix requirement of 52 MPa (7,500 psi) at 56 days. Class F fly ash was used to mitigate the potential for alkali-silica reactivity. The Rio Puerco Bridge has a 220-mm (8.7-in.) -thick cast-in-place concrete deck. The clear width of the deck is 14.5 m (47.6 ft).

The visual inspection of the bridge deck was performed about 3 ½ years after the bridge opened to traffic. A total of 169 cracks were recorded during visual survey of the bridge deck. The sum of crack lengths was 198.6 m (651.3 ft) over a bridge deck area of 1,299 m^2 (13,964.1 ft^2).

With respect to the types of cracking, 50 transverse, 89 diagonal, and 30 longitudinal cracks were observed. The total crack length for the transverse, diagonal, and longitudinal cracks was 92.0 m (301.8 ft), 79.3 m (260.0 ft), and 27.3 m (89.5 ft), respectively. This yielded crack densities of 0.071 m/m^2 (0.022 ft/ft^2) transverse, 0.061 m/m^2 (0.019 ft/ft^2) diagonal, and 0.021 m/m^2 (0.006 ft/ft^2) longitudinal. The crack density for the entire deck including all transverse, diagonal, and longitudinal cracks was 0.153 m/m^2 (0.047 ft/ft^2).

All cracks on the bridge were hairline cracks with a width of less than 0.8 mm (0.031 in.). No major distress was observed in the bridge survey. The majority of the cracks observed were short and randomly distributed diagonal cracks. The three spans have similar bridge deck width and length. Cracks were typically limited at span ends. Other defects such as small surface spalls occurred due to breaking of tined edges or the crack edges were observed.

Considering the heavy ADT on the bridge, the Rio Puerco Bridge was in good condition. The use of HPC provides significantly higher strength that can lead to more efficient designs and improved durability.

U.S. 401 Bridge Over the Neuse River (Raleigh, North Carolina)

The U.S. 401 bridge over the Neuse River in Wake County, just north of Raleigh, North Carolina, was the first HPC bridge built in North Carolina (see figure 9). The U.S. 401 bridge consists of two parallel structures. HPC was used in the girders and decks of the northbound and southbound bridges. After the completion of the northbound bridge, it opened to traffic in July 2000. The southbound U.S. 401 bridge opened to traffic in September 2002.

Figure 9. Photo. U.S. 401 Bridge Over the Neuse River (Raleigh, North Carolina).

The U.S. 401 bridge has four spans on both the southbound and northbound sides— two spans of 28 m (91.9 ft) using AASHTO Type IV girders and two spans of 17.5 m (57.4 ft) using AASHTO Type III girders. The overall length of the bridge is 91 m (299 ft). Each bridge is 14.4 m (47.1 ft) wide and carries a 12.0-m (39.4-ft) roadway section and a 1.9-m (6.2-ft) sidewalk. The 215-mm (8.5-in.) -thick deck was placed on stay-in-place metal forms. The AASHTO Type IV prestressed concrete I-girders are 1.37 m (54 in.) deep and the AASHTO Type III prestressed I-girders are 1.15 m (45 in.) deep. There were five girders per span at 3.12 m (10.25 ft) on center. Girders were pretensioned with 15.2-mm (0.6-in.) -diameter draped and straight strands. The use of 69 MPa (10,000 psi) HPC in the girders and 41 MPa (6,000 psi) HPC in the deck allowed the designer to reduce the number of girder lines from six to five.

The visual inspection of the bridge deck was performed about 4 years after the northbound bridge opened to traffic, and 1 ½ years after the southbound U.S. 401 bridge opened to traffic in September 2002. A total of 166 transverse, longitudinal, and diagonal cracks were recorded on the bridge with a combined total crack length of 399.0 m (1,308.3 ft) over a bridge deck area of 2,183 m^2 (23,501 ft^2). All cracks on the bridge were hairline cracks with a width of less than 0.8 mm (0.031 in.). No major distress was observed in the bridge survey.

With respect to the types of cracking, 129 transverse, 7 diagonal, and 30 longitudinal cracks were observed. The total crack length for the transverse, diagonal, and longitudinal cracks was 377.4 m (1,237.3 ft), 5.2 m (17.0 ft), and 16.5 m (54.0 ft), respectively. This yielded crack densities of 0.173 m/m^2 (0.053 ft/ft^2) transverse, 0.002 m/m^2 (0.001 ft/ft^2) diagonal, and 0.008 m/m^2 (0.002 ft/ft^2) longitudinal. The crack density for the entire deck including all transverse, diagonal, and longitudinal cracks was 0.183 m/m^2 (0.056 ft/ft^2).

In general, the work on the U.S. 401 Bridge shows that HPC designs provide significantly higher strength that can lead to more efficient designs and improved durability.

State Route 22 Bridge at Milepost 6.57, Near Cambridge, Guernsey County, Ohio

The State Route 22 Bridge located at Milepost 6.57 (Bridge GUE-22-6.57) in Guernsey County, near Cambridge, Ohio, is the first showcase HPC box girder bridge built in Ohio (see figure 10). HPC was used in both the beams and the stub abutments. The bridge opened to traffic in November 1998.

Figure 10. Photo. State Route 22 Bridge at Milepost 6.57, Near Cambridge, Guernsey County, Ohio.

Bridge GUE-22-6.57 is a 35.2-m (118.66-ft) -long single-span structure over Crooked Creek and is composed of 12 side-by-side prestressed concrete box beams. The total deck thickness is 216 mm (8.5 in.), including a 140-mm (5.5-in.) -thick concrete flange and 76-mm (3-in.) -thick asphalt wearing surface. The bridge deck has a clear width of 14.6 m (48 ft), including two lanes and two shoulders in southbound and northbound directions.

Originally, the bridge was designed to consist of three spans using 533-mm (21-in.) -deep concrete box beams. To lower construction costs by eliminating piers and to improve flow characteristics of the Crooked Creek, Bridge GUE-22-6.57 was redesigned as a single span box girder bridge. 69 MPa (10,000 psi) at 56 days compressive strength concrete was used. The beams are ODOT type B 42-48. Each beam measures 1,219-mm (48-in.) -wide and 1,067-mm (42-in.) -deep. The bridge is supported by stub-type abutments on a single row of H-section steel pile supports. All concrete used in Bridge GUE-22-6.57 was required to have a rapid chloride permeability value of below 1,000 coulombs at 56 days. Concrete mixtures containing silica fume were specified to obtain the required strength and durability requirements. The cast-in-place abutment concrete met the 55 MPa (8,000 psi) design strength in 28 days. Using HPC concrete, the box beam's span range was increased, enabling a lowest cost single span bridge design. In addition, the structure's service life will be enhanced because of the durability benefits associated with HPC's low permeability.

The visual inspection of the bridge deck was performed about 5 ½ years after the bridge opened to traffic. A total of 21 longitudinal, transverse, and diagonal cracks were recorded on the bridge with a combined total crack length of 94.7 m (310.5 ft) over a bridge deck area of 530 m^2 (5,695.7 ft^2). No major distresses were observed in the bridge survey.

With respect to the types of cracking, 1 transverse, 2 diagonal, and 18 longitudinal crack were observed. The total crack length for the transverse, diagonal, and longitudinal cracks was 0.5 m (1.5 ft), 1.5 m (5.0 ft), and 92.7 m (304.0 ft), respectively. This yielded crack densities of 0.001 m/m^2 (0.0003 ft/ft^2) transverse, 0.003 m/m^2 (0.001 ft/ft^2) diagonal, and 0.175 m/m^2 (0.053 ft/ft^2) longitudinal. The crack density for the entire deck including all transverse, diagonal, and longitudinal cracks was 0.179 m/m^2 (0.055 ft/ft^2).

Bridge # GUE-22-0657 has a deck thickness of 216 mm (8.5 in.), including a 140-mm (5.5-in.) -thick concrete flange and 76-mm (3-in.) -thick asphalt wearing surface. It appears that the asphalt wearing surface has protected the concrete underneath from cracking and deterioration.

In general, the work on Bridge # GUE-22-0657 shows that HPC designs provide significantly higher strength that can lead to more efficient designs and improved durability.

I-29 Northbound Bridge (Sioux Falls, South Dakota)

The South Dakota Department of Transportation's (SDDOT) first time use of HPC in an entire superstructure was the construction of I-29 Northbound Bridge in Minnehaha County, near Sioux Falls (see figure 11). The I-29 Northbound Bridge was built in the summer of 1999. HPC was used in the girders, deck, and bent diaphragms.

Figure 11. Photo. I-29 Northbound Bridge (Sioux Falls, South Dakota).

The I-29 Northbound Bridge is a railroad overpass structure with a 27 degree skew. The bridge consists of typical three-span precast, prestressed concrete girders with standard integral abutments and integral bent diaphragms. AASHTO Type II girders were used in the 16.5-m (54-ft) -long end spans and the 18.6-m (61-ft) -long main span. The total length of the I-29 Northbound Bridge is 52.4 m (172 ft). There are two traffic lanes and two shoulders for a clear deck width of 12.2 m (40 ft). The deck of I-29 Northbound Bridge is composed of 229-mm (9-in.) -thick cast-in-place concrete.

The reason that the I-29 Northbound Bridge was chosen was mainly because of the high traffic counts and heavy use of deicing salts. This provided a test of the strength and durability of HPC in bridge decks. The use of HPC allowed designers to reduce the number of girders in each span from five to four. Design compressive strength of the girder concrete was 68.3 MPa (9,900 psi) at 28 days and 56.9 MPa (8,250 psi) at release of the strands. The deck utilized a 31 MPa (4500 psi) compressive strength concrete. To improve durability, the cementitious materials in the deck concrete consisted of fly ash (17 percent) and silica fume (8 percent). The girders had a low water-cementitious materials ratio of 0.25. Curing was required for a minimum of 7 days.

The visual inspection of the bridge decks was performed about 4 ½ years after the bridge opened to traffic. A total of 143 cracks were recorded during the visual survey of the bridge decks. The sum of crack lengths was 271.6 m (890.5 ft) over a bridge deck area of 629 m^2 (6,760 ft^2).

With respect to the types of cracking, 101 transverse, 30 diagonal, and 12 longitudinal cracks were observed. The total crack length for the transverse, diagonal, and longitudinal cracks was 217.3 m (712.5 ft), 39.2 (128.5 ft), and 15.1 (49.5 ft), respectively. This yielded crack densities of 0.346 m/m^2 (0.061 ft/ft^2) transverse, 0.062 m/m^2 (0.019 ft/ft^2) diagonal, and 0.024 m/m^2

(0.007 ft/ft^2) longitudinal. The crack density for the entire deck including all transverse, diagonal, and longitudinal cracks was 0.432 m/m^2 (0.132 ft/ft^2).

The longitudinal cracks were very limited and tend to connect to the diagonal cracks near the span joints. The relatively flexible bridge structural system combined with the heavy ADT on the bridge might have contributed to the development of some cracks.

In general, the top surface of I-29 Northbound Bridge was in good condition, with only hairline cracks found, showing that HPC designs provide significantly higher strength that can lead to more efficient designs and improved durability.

I-29 Southbound Bridge (Sioux Falls, South Dakota)

Following the success of the I-29 Northbound Bridge in Minnehaha County, near Sioux Falls, the SDDOT decided to construct another HPC bridge - the I-29 Southbound Bridge, less than a half mile away from the I-29 Northbound Bridge (see figure 12). I-29 Southbound Bridge was built in the summer of 2000. HPC was used in girders, deck, and bent diaphragms. The I-29 Southbound Bridge would serve for comparison purposes and additional research.

Figure 12. Photo. I-29 Southbound Bridge (Sioux Falls, South Dakota).

I-29 Southbound Bridge is at a 27 degree skew to the railroad. The bridge consists of typical three-span precast, prestressed concrete girders with standard integral abutments and integral bent diaphragms. AASHTO Type II girders were used for the 16.5-m (54-ft) -long end spans and the 18.6-m (61-ft) -long main span. The total length of the I-29 Southbound Bridge is 52.4 m (172 ft). There are two traffic lanes and two shoulders for a clear deck width of 12.2 m (40 ft).

The deck of I-29 Southbound Bridge is composed of 229-mm (9-in.) -thick cast-in-place concrete.

The use of HPC allowed designers to reduce the number of girders in each span from five to four. Design compressive strength of the girder concrete was 68.3 MPa (9,900 psi) at 28 days and 56.9 MPa (8,250 psi) at release of the strands. The deck utilized a 31 MPa (4,500 psi) compressive strength concrete. To improve durability, the cementitious materials in the deck concrete included fly ash. The girders had a low water-cementitious materials ratio of 0.25. Curing was required for a minimum of 7 days.

The visual inspection of the bridge deck was performed about 4 years after the bridge opened to traffic. A total of 119 cracks were recorded during visual survey of the bridge decks. The sum of crack lengths was 341.9 m (1,121 ft) over a bridge deck area of 629 m^2 (6,760 ft^2).

With respect to the types of cracking, 75 transverse, 42 diagonal, and 2 longitudinal cracks were observed. The total crack length for the transverse, diagonal, and longitudinal cracks was 266.0 m (872.0 ft), 74.7 m (245.0 ft), and 1.2 m (4.0 ft), respectively. This yielded crack densities of 0.423 m/m^2 (0.129 ft/ft^2) transverse, 0.119 m/m^2 (0.036 ft/ft^2) diagonal, and 0.002 m/m^2 (0.001 ft/ft^2) longitudinal. The crack density for the entire deck including all transverse, diagonal, and longitudinal cracks was 0.544 m/m^2 (0.166 ft/ft^2).

Diagonal cracks were typically limited to span ends. Small surface spalls, which either occurred due to breaking of tined edges or the crack edges, were observed. The longitudinal cracks were very limited and tend to connect to the diagonal cracks near the span joints. The relatively flexible bridge structural system combined with the heavy ADT on the bridge might have contributed to the development some cracks. All cracks measured are hairline cracks with a width of less than 0.8 mm (0.031 in.). In general, the top surface of I-29 Souththbound Bridge was in good condition, with only hairline cracks found, showing that HPC designs provide significantly higher strength that can lead to more efficient designs and improved durability.

Porter Road (Dickson County, Tennessee)

Porter Road Bridge over State Route 840 in Dickson County was constructed in 2000 (see figure 13). The structure is 97 m (318 ft) long and 9.8 m (32 ft) wide. It carries one eastbound lane and one westbound lane of Porter Road. The structure consists of 210-mm (8¼-in.) -thick concrete deck with stay-in-place forms on two 48.5-m (159-ft) -long continuous spans with concrete bulb-tee prestressed concrete girders. The superstructure consists of one concrete pier and two concrete abutments. The structure was built with a 27 degree skew at both abutments and the pier. Four precast concrete bulb-tee girders, BT-72, on 2.5-m (8-ft 4-in.) centers support each span. The concrete stub abutments are separated from the State Route 840 with loose riprap slope protection. The concrete pier is comprised of a cast-in-place concrete hammerhead cap on a cast-in-place pier stem.

Figure 13. Photo. Porter Road (Dickson County, Tennessee).

The retaining wall, abutments, bent, girders, and deck were constructed with high performance concrete (HPC). The factors that led to the use of HPC in this bridge included longer span length and a more durable structure.

The visual inspection of the bridge deck was performed about 1 ½ years after the bridge was opened to traffic. A total of 90 transverse cracks were observed. There were 10 diagonal corner cracks and the deck exhibited map cracks primarily along the centerline and Eastbound roadway. The map cracking encompassed about 112 m^2 (1,200 ft^2) of the deck surface and the crack widths ranged from 0.08 to 0.25 mm (0.003 to 0.010 in.) and the cracks were generally 203 mm (8 in.) apart in both directions. No longitudinal cracks were observed.

With respect to the types of cracking, 90 transverse, 10 diagonal, and 0 longitudinal cracks were observed. The total crack length for the transverse and diagonal cracks was 248.6 m (815.0 ft), and 13.7 (45.0 ft), respectively. This yielded crack densities of 0.265 m/m^2 (0.081 ft/ft^2) (transverse), and 0.015 m/m^2 (0.005 ft/ft^2) (diagonal). The crack density for the entire deck including all transverse and diagonal cracks was 0.280 m/m^2 (0.085 ft/ft^2).

In general, the work on the bridge showed that HPC designs provided significantly higher strength that can lead to more efficient designs and improved durability.

Hickman Road (Dickson County, Tennessee)

Hickman Road Bridge over State Route 840 in Dickson County was constructed in 2000 (see figure 14). The structure is 88.7 m (290 ft 8 in.) long and 9.8 m (32 ft) wide. It carries one eastbound lane and one westbound lane of Hickman Road. The structure consists of 210-mm

(8¼-in.) -thick concrete deck with stay-in-place forms on 42.5-m (139-ft 4-in.) and 46.2-m (151-ft 4-in.) -long continuous spans with bulb-tee prestressed concrete girders. The superstructure consists of one concrete pier and two concrete abutments. The structure was built with a 17.5 degree skew at both abutments and the pier. Four precast bulb-tee girders, BT-72; on 2.5 m (8 ft 4 in.) centers support each span. The concrete stub abutments are separated from the State Route 840 with loose riprap slope protection. The concrete pier is comprised of a cast-in-place concrete hammerhead cap on a cast-in-place pier stem.

Figure 14. Photo. Hickman Road (Dickson County, Tennessee).

The retaining wall, abutments, bent, girders, and deck were constructed with HPC. The factors that led to the use of HPC in this bridge included longer span length and a more durable structure.

The bridge deck was inspected in October, 2002. Defects in the top surface included transverse cracks, map cracks, diagonal corner cracks in the acute corners, patches, small sand pockets, and an area of surface milling.

Transverse cracks were primarily along the centerline of the roadway. A total of 10 transverse cracks were identified on the deck. The crack widths ranged from 0.2 mm to 0.6 mm (0.007 to 0.025 in.). Map cracks were primarily along the centerline and eastbound roadway near the pier. The crack widths ranged from 0.08 to 0.25 mm (0.003 to 0.010 in.) and the cracks were generally in a form of 17 mm by 17 mm (8 in. by 8 in.) network. Diagonal cracks were primarily in the acute corners, SE and NW, of the bridge deck. The widths of the six diagonal cracks ranged from 0.005 to 0.016 in. No longitudinal cracks were observed.

With respect to the types of cracking, 10 transverse, 6 diagonal, and 0 longitudinal cracks were observed. The total crack length for the transverse and diagonal was 25.6 m (84.0 ft), and 7.9 m

(26.0 ft), respectively. This yielded crack densities of 0.032 m/m^2 (0.001 ft/ft^2) transverse, and 0.001 m/m^2 (0.003 ft/ft^2) diagonal. The crack density for the entire deck including all transverse and diagonal cracks was 0.041 m/m^2 (0.013 ft/ft^2).

In addition to the different types of cracking noted, a few isolated small defects were found. These defects included patches, sand pockets, inclusions, a small spall, and a pattern of shallow embossing. Six sand pockets ranged in size from 25.4 mm to 50.8 mm (1 in. to 2 in.) and 25.4 mm to 38 mm (1 in. to 1½ in.) deep. The sand pockets appear to be the result of inadequate mixing of the silica fume at the time of construction, due to the gray coloration. Three inclusions were identified on the surface of the deck. Generally, these inclusions consisted of debris including foam board similar to styrofoam. The embossed areas were due to a rolling screed at the time of construction.

State Highway 249 (Tomball Parkway) over Louetta Road (Houston, Texas)

The Tomball Parkway (S.H. 249) Bridge over Louetta Road in Houston, Texas (see figure 15) consists of two separate bridges, one carrying three lanes of the northbound traffic and the other carrying three lanes of the southbound traffic with an additional exit ramp. Both bridges consist of precast U-beam girders covered with precast concrete deck panels 89-mm thick x 2.44-m long (3.5-in. thick × 8-ft long), which are in turn covered with 95 mm (3.75 in.) of cast-in-place concrete. The substructures consist of concrete columns and concrete abutments at each end.

Figure 15. Photo. State Highway 249 (Tomball Parkway) over Louetta Road (Houston, Texas).

The Tomball Parkway Bridge is a major structure carrying heavy traffic. It is 119.3 m (391 ft) long and consists of three spans in each direction. Span 1, span 2, and span 3 have approximate lengths of 37.1 m (121.5 ft), 41.3 m (135.5 ft) and 40.9 m (134.0 ft), respectively. The width of the bridge is variable, ranging from 48.8 m (160 ft) at the ramp to 36.6 m (120 ft) in the middle. The bridge has a skew of 33 to 39 degrees. Each span in the northbound bridge consists of five Texas U54 beams, and each span in the southbound bridge consists of six Texas U54 beams. Beams are prestressed. The specified compressive strength of the girders at release of prestressing and 56 days ranged from 47.5 to 60.6 MPa (6,900 to 8,800 psi) and 67.5 to 90.3 MPa (9,800 to 13,100 psi), respectively. At the interior bents, each beam is supported by a single post-tensioned pier.

All beams, piers, and precast deck panels were fabricated using high performance, high strength concrete. For comparison purposes, the southbound main-lane bridge has a high performance, high strength cast-in-place deck, whereas the northbound main-lane bridge has a high performance, normal strength cast-in-place concrete deck. The precast deck panels were prestressed utilizing 9.5-mm (⅜-in.) -diameter strands. The cast-in-place concrete overlay was reinforced with #5 bars at a spacing of 6 in. center-to-center in the transverse direction and #4 bars at a spacing of 25.4 mm (12 in.) center-to-center in the longitudinal direction. The rebar used in the cast-in-place deck was Grade 60 and uncoated. The concrete cover over the #5 transverse reinforcing bars in the cast-in-place deck was specified as 50.8 mm (2 in.). The concrete cover below the 9.5 mm (⅜ in.) diameter strands in the precast deck panels was specified as 44 mm (1¾ in.). The decks of each bridge were constructed simultaneously using similar construction techniques by the same personnel.

The construction of the bridge decks started in October 1996 and the bridge was opened to traffic in both directions in June 1998.

The northbound and southbound bridges are exhibiting comparable magnitude and pattern of cracking. A total of 1,703 longitudinal, transverse, and diagonal cracks were recorded on the two bridges with a combined total crack length of 3,385 m (11,098.4 ft) over a bridge deck area of 6,197 m^2 (66,636 ft^2). However, 98 percent of these cracks were hairline cracks with widths less than 0.8 mm (1/32 in.) The remaining 2 percent of the cracks were classified as fine cracks with widths in the range of 0.8 to 1.6 mm (1/32 to 1/16 in.).

With respect to the types of cracking, 228 transverse, 41 diagonal, and 576 longitudinal cracks were observed on the northbound bridge. The total crack length for the transverse, diagonal, and longitudinal cracks was 291.3 m (955.0 ft), 83.8 m (274.9 ft), and 1,232.0 m (4,039.2 ft), respectively. This yielded crack densities of 0.101 m/m^2 (0.031 ft/ft^2) transverse, 0.029 m/m^2 (0.009 ft/ft^2) diagonal, and 0.429 m/m^2 (0.131 ft/ft^2) longitudinal. The crack density for the entire northbound deck including all transverse, diagonal, and longitudinal cracks was 0.560 m/m^2 (0.171 ft/ft^2).

With respect to the types of cracking, 282 transverse, 58 diagonal, and 518 longitudinal cracks were observed on the southbound bridge. The total crack length for the transverse, diagonal, and longitudinal cracks was 444.3 m (1456.7 ft), 98.7 m (323.6 ft), and 1,234.9 m (4,048.9 ft), respectively. This yielded crack densities of 0.134 m/m^2 (0.041 ft/ft^2) transverse, 0.030 m/m^2 (0.009 ft/ft^2) diagonal, and 0.371 m/m^2 (0.113 ft/ft^2) longitudinal. The crack density for the entire southbound deck including all transverse, diagonal, and longitudinal cracks was 0.535 m/m^2 (0.160 ft/ft^2).

The cast-in-place decks of the northbound and southbound bridges were constructed with two different classes of HPC. Normal strength modified class S HPC was used in the northbound bridge, and high strength class K HPC was used in the southbound bridge. However, it appears that mixture proportions did not play a significant role in the cracking of the decks. Class K HPC used in the southbound bridge was reported to have a low w/cm ratio of 0.35 and a fly ash content of 32 percent by weight of the cementitious material content. This class K HPC mixture had a high shrinkage and cracking potential. However, the performance of this mixture was comparable to normal strength modified class S HPC used in the northbound bridge, which was reported to have a w/cm ratio of 0.43 and a fly ash content of 28 percent by weight of the total cementitious material content.

Significant difference in the coefficient of thermal expansion of the precast deck panels and cast-in-place decks may have contributed to the cracks observed in the two bridges. It was reported that the coefficient of thermal expansion of the cast-in-place deck was about 4.0 $\mu\varepsilon$ / ˚F. On the other hand the coefficient of thermal expansion of the precast deck panels was reported to be about 7.3 $\mu\varepsilon$ / ˚F.

It was also reported that the construction of all the spans of the northbound and southbound bridges was done as a single pour construction without properly locating the tooled control joints at the centerline of the skew. This single pour construction might have also contributed to the development of cracks observed at the northbound and southbound bridges.

At span ends along the skew, a number of fine width cracks (1/32 to 1/16 in.) were observed. Some of these cracks were exhibiting spalling due to breaking of the edges. The layout of the cast-in-place decks and precast deck panels at span ends may have contributed to the development and widening of these cracks. At span ends, the cast-in-place decks were skewed but precast deck panels had a straight geometry.

It is noted that for the longitudinal cracking, another factor that could contribute is shortening of the precast panels in the transverse direction. As the panels shorten because of creep and shrinkage, the cast-in-place portion of the deck has to accommodate the movement. This can lead to tensile stresses in the cast-in-place concrete. In addition, the Texas U-beam is stiffer in the transverse direction than the same depth I-beam. This means that any transverse shortening of the deck is going to encounter a lot more resistance with a U-beam than with an I-beam. This will also lead to higher tensile stresses in the deck with a U-beam and greater likelihood of longitudinal cracking.

U.S. Route 67 Bridge (San Angelo, Texas)

The U.S. Route 67 Bridge in San Angelo, Texas is part of a high-speed expressway and carries traffic over the North Concho River, U.S. Route 87, and South Orient Railroad tracks (see figure 16). It was constructed in 1997 and opened to traffic in January 1998. The bridge consists of two separate structures, one carrying two lanes of eastbound traffic and the other two lanes of westbound traffic. Both structures consist of prestressed concrete I-beam girders covered with precast concrete deck panels 102-mm thick x 2.4-m long (4-in. thick × 8-ft long), which in turn are covered with 89 mm (3 ½ in.) of cast-in-place concrete. The substructures consist of concrete columns, concrete bent caps, and concrete abutments at each end.

Figure 16. Photo. U.S. Route 67 Bridge (San Angelo, Texas). The eastbound bridge is in foreground, and the westbound bridge is in the background.

The eastbound structure is 290 m (950 ft) long and consists of eight spans. Spans 7 and 8 are skewed to accommodate the railroad tracks. The bridge decks at spans 1 through 4 are 11.6 m (38 ft) wide. The decks progressively widen in spans 5, 6, 7, and 8 to accommodate an exit-ramp at the eastern end of the eastbound bridge. Except for the girders in spans 6 through 8, HPC was used for all girders, deck panels, and cast-in-place concrete in the eastbound structure.

The westbound structure is 293 m (960 ft) long and consists of nine spans. Spans 7, 8, and 9 are skewed to accommodate the railroad tracks. The bridge deck at span 1 is 11.6 m (38 ft) wide, and the decks progressively widen in spans 2 through 9 to accommodate an on-ramp at the eastern end of the westbound bridge. At the time of the inspection, traffic lanes on the western half of the westbound bridge merged down to a single lane to accommodate original construction of the expressway west of the bridge. In the westbound structure, HPC was used only for the cast-in-place decks of spans 1 through 5.

With respect to the types of cracking, 204 transverse, 11 diagonal, and 125 longitudinal cracks were observed on the eastbound bridge. The total crack length for the transverse, diagonal, and longitudinal cracks was 384.8 m (1,261.6 ft), 11.0 m (36.2 ft), and 190.8 m (625.6 ft), respectively. This yielded crack densities of 1.005 m/m^2 (0.306 ft/ft^2) transverse, 0.029 m/m^2 (0.009 ft/ft^2) diagonal, and 0.498 m/m^2 (0.152 ft/ft^2) longitudinal. The crack density for the entire eastbound deck including all transverse, diagonal, and longitudinal cracks was 1.532 m/m^2 (0.467 ft/ft^2).

With respect to the types of cracking, 179 transverse, 12 diagonal, and 177 longitudinal cracks were observed on the westbound bridge. The total crack length for the transverse, diagonal, and longitudinal cracks was 645.0 m (2,114.8 ft), 13.5 m (44.4 ft), and 306.3 m (1,004.2 ft), respectively. This yielded crack densities of 1.625 m/m^2 (0.496 ft/ft^2) transverse, 0.034 m/m^2 (0.010 ft/ft^2) diagonal, and 0.772 m/m^2 (0.235 ft/ft^2) longitudinal. The crack density for the entire westbound deck including all transverse, diagonal, and longitudinal cracks was 2.431 m/m^2 (0.741 ft/ft^2).

The eastbound bridge where both the precast deck panels and the cast-in-place decks were constructed utilizing HPC has exhibited better cracking performance compared to the westbound bridge where HPC was used only in the cast-in-place decks of a limited number of spans (1 through 5). Comparing the cast-in-place deck of spans 1 through 5 of the westbound bridge with that of the eastbound bridge, the performance of the eastbound cast-in-place deck is found to be superior. This could be attributed to a better quality HPC used in the cast-in-place deck of the eastbound bridge. The class K (HPC) used in the cast-in-place deck of the eastbound bridge was reported to have a lower water-to-cementitious material ratio compared to the class S (HPC) used in the cast-in-place deck of the westbound bridge. The water-to-cementitious material ratio of class K (HPC) was specified as 0.31 compared to 0.42 of class S (HPC).

It is noted that a relatively large number of short-length transverse cracks were observed in spans 5 through 8 of the eastbound bridge. The eastbound bridge along the southern edge is also the side where the beams have a longer span and larger skew angle. These factors may have contributed to more cracking.

The rectangular pattern cracking particularly observed in spans 8 and 9 of the westbound bridge may be attributed to a combination of factors. These may include single pour construction of a number of spans, as indicated by Texas DOT, and the higher shrinkage of non-HPC mixture at these locations. The class S concrete used in the cast-in-place decks of spans 6 through 9 of the westbound bridge had a cement content of 363 kg/m^3 (611 lb/yd^3), without any pozzolans, and had a water-to-cement ratio of 0.42.

Route 40 Bridge over Falling River (Brookneal, Virginia)

The Route 40 Bridge over Falling River in Campbell County was constructed during the winter of 1995-1996 (see figure 17). The structure is 98 m (320 ft) long and 13.4 m (44 ft) wide. It carries one eastbound lane and one westbound lane of Virginia Route 40. The structure consists of 216-mm (8½-in.) -thick concrete deck with stay-in-place forms on four 24.4-m (80-ft) -long simple span prestressed concrete superstructure, on three concrete piers and two concrete abutments. The structure was built with a 20 degree skew at both abutments and all three piers. Five precast AASHTO Type IV girders, on 3.1 m (10 ft) centers support each span. The concrete stub abutments are separated from Falling River with loose riprap slope protection. The concrete piers are comprised of cast-in-place concrete hammerhead caps on cast-in-place pier stems.

Figure 17. Photo. Route 40 Bridge over Falling River (Brookneal, Virginia).

The abutments, piers, girders and deck were constructed with HPC. The factors that led to the use of HPC in this bridge included the use of fewer girders and a more durable structure. If HPC was not used, two more lines of girders would have been required.

Bridge inspection reports dated 5/3/96, 6/22/98, 5/3/00, and 4/29/02 were identified for this bridge. The 1996 inspection report documented that small horizontal cracks existed along the edges of steel plates in the beams at bearing areas; back corner of several beams cracked or delaminated slightly; and small hairline cracks existed on abutments and piers. The 1998 and 2000 inspection reports identified the same conditions. Cracks in the deck surface were first documented in the 2002 inspection report. Other defects included cracks in parapets, beam ends, abutment backwalls, and pier caps.

The visual inspection was performed on November 19 through 21, 2002. There were longitudinal, transverse, and diagonal corner cracks on the surface of the deck. According to ACI 201, these crack widths are classified as hairline cracks. Small spalls and fractured fins from deep grooving were also observed.

With respect to the types of cracking, 21 transverse, 9 diagonal, and 27 longitudinal cracks were observed on the bridge. The total crack length for the transverse, diagonal, and longitudinal cracks was 61.3 m (201.0 ft), 15.9 m (52.0 ft), and 141.8 m (465.0 ft), respectively. This yielded crack densities of 0.090 m/m^2 (0.027 ft/ft^2) transverse, 0.023 m/m^2 (0.007 ft/ft^2) diagonal, and 0.207 m/m^2 (0.063 ft/ft^2) longitudinal. The crack density for the entire deck including all transverse, diagonal, and longitudinal cracks was 0.320 m/m^2 (0.098 ft/ft^2).

In addition to the different types of cracking noted, a few isolated small defects were found on the deck. These defects included small spalls and D-spalls. Deep irregular grooving was found in some areas of the deck, and the grooves were deep enough to contribute to the fracturing of the fins. On the other hand, very shallow grooves were also found on the deck surface.

Virginia Avenue Bridge (Richlands, Virginia)

The Virginia Avenue Bridge over Clinch River, located in the Town of Richlands in Tazewell County, Virginia, was constructed in late 1997 (see figure 18). The structure is 45.1 m (148 ft) long and 12.2 m (40 ft) wide. It carries one northbound lane and one southbound lane of Virginia Avenue. The structure consists of 216-mm (8½-in.) -thick concrete deck with stay-in-place forms on five 22.6-m (74-ft) -long simple span prestressed concrete beams, on one concrete pier and two concrete abutments. The structure was built with no skew at either abutment or at the pier. Five precast AASHTO Type III girders on 2.7 m (8 ft 9 in.) and 2.8 m (9 ft 3 in.) centers support each span. The concrete stub abutments are separated from Clinch River with loose riprap slope protection. The concrete pier is comprised of cast-in-place concrete caps on cast-in-place pier stems.

Figure 18. Photo. Virginia Avenue Bridge (Richlands, Virginia).

The girders and deck were constructed with HPC. The factors that led to the use of HPC in this bridge included use of fewer girders and a more durable structure. If HPC was not used, two more lines of girders would have been required.

Previous inspection indicated cracks on the bridge deck. In the 2000 inspection, a total of 48.8 linear meters (160 linear feet) of deck cracks over the pier were documented, ranging in width from 0.40 to 0.76 mm (0.016 to 0.030 in.). The same cracks were noted again in the 2002 report, with random transverse cracks up to 0.76-mm (0.030-in.) -wide on the sidewalks.

The bridge deck was visually inspected on April 29 and 30, 2003. Defects in the top surface included longitudinal cracks, transverse cracks, one diagonal crack, and small gouges.

With respect to the types of cracking, 14 transverse, 1 diagonal, and 62 longitudinal cracks were observed. The total crack length for the transverse, diagonal, and longitudinal cracks was 32.6 m (107.0 ft), 1.2 m (4.0 ft), and 125.1 m (410.0 ft), respectively. This yielded crack densities of 0.079 m/m^2 (0.024 ft/ft^2) transverse, 0.003 m/m^2 (0.001 ft/ft^2) diagonal, and 0.303 m/m^2 (0.092 ft/ft^2) longitudinal. The crack density for the entire deck including all transverse, diagonal, and longitudinal cracks was 0.385 m/m^2 (0.117 ft/ft^2).

Other defects found on the deck included a small gouge in span B and transverse cracks in the sidewalks. The small gouge was located in span B, with a dimension of 25.4 mm (12 in.) long, 76.2 mm (3 in.) wide, and 12.7 mm (½ in.) deep. The west sidewalk had thirty-three 1.0-m (3.5-ft) -long cracks, while the east sidewalk had twenty-eight 1.4-m (4.5-ft) -long cracks. The cracks range from 0.25 mm (0.010 in.) to 0.50 mm (0.020 in.) in width, and were spaced on 0.6 to 1.2 m (2 to 4 ft) centers.

Eastbound SR18 over SR 516 (King County, Washington)

The eastbound SR18 / SR516 Over-crossing Bridge in King County, just north of Seattle, Washington was the first HPC Bridge built in Washington (see figure 19). It is a two-lane, three-span structure. HPC was used in all girders and decks. The bridge is 90.6 m (297 ft) long. Clear width of the bridge is 11.6 m (38 ft), and it consists of two 3.7-m (12-ft) -wide lanes, one 1.2-m (4-ft) -wide bike lane on the left side and one 3.1-m (10-ft) -wide shoulder on the right. The eastbound SR18 / SR 516 Over-crossing Bridge opened to traffic in March 1998.

The eastbound SR18 / SR 516 Over-crossing Bridge was designed for earthquake zone "C" (acceleration coefficient = 0.25g). Pretensioned concrete girders (WSDOT W74G) with a compressive strength of 69 MPa (10,000 psi) at 56 days were used in this HPC bridge construction project. The use of HPC improves construction economy by enabling longer spans, increased girder spacing, and shallower girders. WSDOT Class 4000D concrete mix design with a compressive strength of 28 MPa (4,000 psi) at 28 days was used in the construction of cast-in-place concrete deck. The concrete mixture contained fly ash and required continuous wet curing for 14 days.

Figure 19. Photo. Eastbound SR18 over SR 516 (King County, Washington).

The eastbound SR18 / SR 516 Over-crossing Bridge has three spans with lengths of 24.4, 41.8, and 24.4 m (80, 137, and 80 ft), respectively. The skew of the bridge is 40 degrees at both ends. Each span consists of five WSDOT W74G girders made of precast, prestressed HPC. The girders are evenly spaced at 2.4 m (8 ft) centers and support the cast-in-place concrete deck. The bridge decks are 191 mm (7.5 in.) thick. Longitudinal deck reinforcing steel was specified to have 63.5 mm (2½ in.) cover on the top and 25.4 mm (1 in.) cover on the bottom.

The visual inspection of the bridge decks was performed about 6 years after the bridge opened to traffic. The eastbound lanes are exhibiting transverse cracking, diagonal cracking, and longitudinal cracking. A total of 137 cracks were recorded on the bridge with a combined total crack length of 296.2 m (971 ft) over a bridge deck area of 1,050 m^2 (11,286 ft^2). The majority of these cracks were hairline cracks with a width less than 0.40 mm (0.016 in.). No major distress was observed in the bridge survey.

With respect to the types of cracking, 89 transverse, 46 diagonal, and 2 longitudinal cracks were observed. The total crack length for the transverse, diagonal, and longitudinal cracks was 32.6 m (757.5 ft), 63.0 m (206.5 ft), and 2.1 m (7.0 ft), respectively. This yielded crack densities of 0.220 m/m^2 (0.067 ft/ft^2) transverse, 0.060 m/m^2 (0.018 ft/ft^2) diagonal, and 0.002 m/m^2 (0.001 ft/ft^2) longitudinal. The crack density for the entire deck including all transverse, diagonal, and longitudinal cracks was 0.282 m/m^2 (0.086 ft/ft^2).

The total length of transverse cracks and number of cracks for span 2 are greater than those for other spans at the bridge. The crack density on eastbound span 2 is the largest.

Span 2 has the longest span length of 41.8 m (137 ft) compared to other spans of the bridge at 24.4 m (80 ft). This relatively flexible structural system might have contributed to the development and widening of some cracks in span 2.

It is also noted that relatively large numbers of short-length diagonal cracks were observed in span 3 near the span ends. The span ends have a 40 degree skew. Some of these cracks at span ends along the skew were exhibiting spalling due to breaking of the edges. A few fine-width cracks of 1 mm (0.039 in.) were observed. At span ends, the cast-in-place decks were skewed but the girder line supporting these deck panels had a straight geometry. The layout of the cast-in-place decks may partly be attributed to the development of these diagonal cracks.

In general, the work on the eastbound SR18 / SR 516 Over-crossing Bridge shows that HPC designs provide significantly higher strength that can lead to more efficient designs requiring fewer piers and, more important, improved durability.

Petrographic Analysis

Petrographic analysis was performed on cores from each bridge deck with the exception of the Colorado and New Mexico bridges. Petrographic analysis was performed at Turner-Fairbank Highway Research Center. These analyses investigated the material characteristics of the concrete core samples including the types of coarse and fine aggregate, as well as the maximum size of the coarse aggregate. The analyses indicated that all of the samples contained entrained air, however; the actual entrained air content of the hardened concrete was not determined. The bond between the aggregate and the cementitious material was investigated for the samples and there were no indications of a poor bond in the samples examined. The degree of cementitious material hydration was also estimated for the samples and all of the samples indicated a reasonable degree of hydration.

In many cases, ettringite crystals were observed in air voids of the samples. Often, ettringite filled part of a void, but voids fully filled with ettringite were also found in some of the concrete samples. There was no evidence of deterioration associated with the existence of the ettringite in the concrete.

The samples were also investigated for deleterious reactions such as alkali-silica reaction (ASR) and sulfate attack. There was no indication of secondary deposits from deleterious reactions in the samples investigated.

Information on water-to-cementitious material ratios, cementitious material contents, and hardened air contents were not investigated in the petrographic analyses performed.

CHAPTER 5. DISCUSSION OF RESULTS

CRACK DENSITIES

Inspections were performed on all of the bridge decks. These inspections included a crack survey that measured length of the transverse, diagonal, and longitudinal cracks on each deck. The widths of the cracks were also measured. The crack densities were calculated from these surveys. Table 3 presents the calculated crack densities for each bridge deck. The data include the transverse, diagonal, longitudinal, and total crack densities. Figure 20 is a graphical representation of the crack densities for all of the bridge decks, these data include the transverse, diagonal, longitudinal, and total crack densities for each deck.

For all of the bridge decks, the average transverse, diagonal, longitudinal, and total crack densities were 0.248, 0.027, 0.137, and 0.412 m/m^2 (0.073, 0.008, 0.042, and 0.123 ft/ft^2), respectively. The westbound lane of the San Angelo, Texas bridge deck exhibited the highest total crack density of 2.431 m/m^2 (0.741 ft/ft^2), while the New Hampshire Route 104 bridge deck exhibited the lowest crack density of 0.010 m/m^2 (0.003 ft/ft^2). It should be noted that the Colorado bridge deck had an asphalt overlay and no cracking was observed. Analysis was performed comparing the various structural systems, concrete constituent and material properties and the related crack densities to determine if there was a correlation.

Table 3. Transverse, Diagonal, Longitudinal, and Total Cracking.

State	**AL**	**CO**	**GA**	**LA**	**NE**	**NH**	**NH**	**NM**	**NC**	**OH**
Bridge Name	AL 199	Yale Ave.	S.R. 920	Charenton	120th St.	Route 104	Route 3A	Rio Puerco	U.S. 401	U.S. 22
Total Deck Area (m^2)	2969	0	2970	1494	1716	299	176	1299	2183	530
Transverse Cracking										
No. of Transverse Cracks	108	0	61	46	170	0	5	50	129	1
Length of Transverse Cracks, m	510.4	0.0	125.8	57.2	330.2	0.0	5.6	92.0	377.4	0.5
Transverse Crack Density, m/m^2	0.172	0.000	0.042	0.038	0.192	0.000	0.032	0.071	0.173	0.001
Diagonal Cracking										
No. of Diagonal Cracks	8	0	30	0	64	0	0	89	7	2
Length of Diagonal Cracks, m	9.2	0.0	65.2	0.0	106.4	0.0	0.0	79.3	5.2	1.5
Diagonal Crack Density, m/m^2	0.003	0.000	0.022	0.000	0.062	0.000	0.000	0.061	0.003	0.003
Longitudinal Cracking										
No. of Longitudinal Cracks	5	0	0	0	25	2	0	30	30	18
Length of Longitudinal Cracks, m	8.7	0.0	0.0	0.0	71.1	3.1	0.0	27.3	16.5	92.7
Longitudinal Crack Density, m/m^2	0.003	0.000	0.000	0.000	0.041	0.010	0.000	0.021	0.008	0.175
Total Cracking										
Total Number of Cracks	121	0	91	46	259	2	5	169	166	21
Total Length of Cracks, m	528.3	0.0	191.0	57.2	507.7	3.1	5.6	198.6	399.0	94.7
Total Crack Density, m/m^2	0.178	0.000	0.064	0.038	0.296	0.010	0.032	0.153	0.183	0.179

Table 3. Transverse, Diagonal, Longitudinal, and Total Cracking. - *Continued*

State	**SD**	**SD**	**TN**	**TN**	**TX (Louetta)**		**TX (San Angelo)**		**VA**	**VA**	**WA**
Bridge Name	I-29 NB	I-29 SB	Porter	Hickman	NB	SB	EB	WB	Route 40	VA Ave.	S.R. 18
Total Deck Area (m^2)	629	629	937	811	2872	3325	383	397	684	413	1050
Transverse Cracking											
No. of Transverse Cracks	101	75	90	10	228	282	204	179	21	14	89
Length of Transverse Cracks, m	217.3	266.0	248.6	25.6	291.3	444.3	384.8	645.0	61.3	32.6	231.0
Transverse Crack Density, m/m^2	0.346	0.423	0.265	0.032	0.101	0.134	1.005	1.625	0.090	0.079	0.220
Diagonal Cracking											
No. of Diagonal Cracks	30	42	10	6	41	58	11	12	9	1	46
Length of Diagonal Cracks, m	39.2	74.7	13.7	7.9	83.8	98.7	11.0	13.5	15.9	1.2	63.0
Diagonal Crack Density, m/m^2	0.062	0.119	0.015	0.010	0.029	0.030	0.029	0.034	0.023	0.003	0.060
Longitudinal Cracking											
No. of Longitudinal Cracks	12	2	0	0	576	518	125	177	27	62	2
Length of Longitudinal Cracks, m	15.1	1.2	0.0	0.0	1,232.0	1,234.9	190.8	306.3	141.8	125.1	2.1
Longitudinal Crack Density, m/m^2	0.024	0.002	0.000	0.000	0.429	0.371	0.498	0.772	0.207	0.303	0.002
Total Cracking											
Total Number of Cracks	143	119	100	16	845	858	340	368	57	77	137
Total Length of Cracks, m	271.6	341.9	262.3	33.6	1,607.1	1,777.9	586.6	964.8	219.0	158.9	296.2
Total Crack Density, m/m^2	0.432	0.544	0.280	0.041	0.560	0.535	1.532	2.431	0.320	0.385	0.282

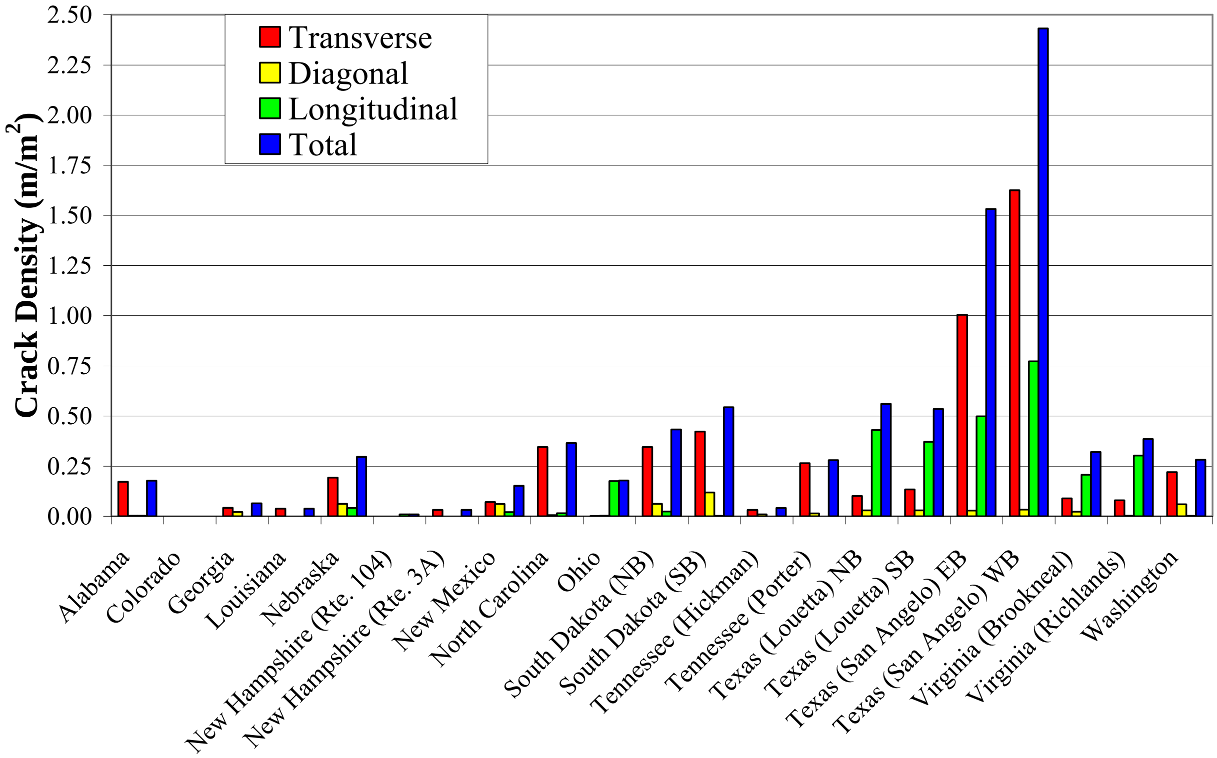

Figure 20. Chart. Crack Densities for All Bridge Decks.

STRUCTURAL SYSTEMS

The structural systems used in the 19 HPC bridges consisted of the following three types:

- Precast, prestressed concrete beams with a full depth cast-in-place concrete deck (14 bridges)
- Precast, prestressed concrete beams supporting precast, prestressed concrete deck panels with a partial depth composite cast-in-place concrete deck (3 bridges)
- Adjacent precast, prestressed concrete box beams with or without a cast-in-place concrete deck (2 bridges)

In general, each structural system exhibited a different pattern of cracks. The following discussion relates to the influence of the structural system on the pattern and density of cracks.

Bridges with Full Depth Cast-in-Place Concrete Decks

The 14 bridges that used precast, prestressed concrete beams with a full depth cast-in-place concrete deck exhibited a wide range of total crack densities. The bridges in Georgia, Louisiana, New Hampshire (Route 104), and Tennessee (Hickman) had relatively low total crack densities. In contrast, the bridges in Alabama, Nebraska, New Mexico, North Carolina, South Dakota, Tennessee (Porter), Virginia, and Washington had at least twice as much cracking. On average, the latter group of bridges had about eight times as much cracking as the former group.

For most bridges, the highest crack density occurred for cracks running in the transverse direction. The exceptions were the two bridges in Virginia as discussed later. The cracking densities in each span of each bridge were compared with span lengths, beam spacings, deck thickness, girder types, clear deck spans, and beam span-to-depth ratios in an attempt to identify any overall correlations. None were identified. However, some comparisons between crack densities on spans of individual bridges may be relevant.

The Georgia bridge is a four-span structure and exhibits an unusual pattern of deck cracking in that eastbound span 3 and westbound span 2 show very little cracking compared to westbound span 3 and eastbound span 2. Some diagonal cracking perpendicular to the skewed diaphragms at the end of the spans is present.

The Louisiana bridge is a five-span continuous structure that exhibits very little deck cracking. Most of the cracks that occur are located in the negative moment regions over the intermediate piers.

Each of the two bridges in North Carolina consists of two pairs of continuous spans. Most of the cracking is in the transverse direction and occurs in the half of each span adjacent to the continuity connection over the pier.

The two South Dakota bridges have a similar amount of cracking. The majority of the cracking is in the transverse direction with some diagonal cracking at the skewed abutments.

The two Virginia bridges are the only two bridges with a full cast-in-place deck on precast, prestressed concrete beams that have more longitudinal cracking than transverse cracking. The reason for this is unclear as the structural system for these bridges is very similar to that of the other 12 bridges with full depth cast-in-place concrete decks.

In summary, when the structural system of the bridge includes skewed supports, diagonal cracks are likely to occur near the supports. When the structural system of the bridge includes continuity over the supports, negative moment transverse cracks are likely to occur. Other transverse cracks and any longitudinal cracks appear to be unrelated to the structural system.

Bridges with Precast Deck Panels and Cast-in-Place Decks

Three bridges used precast, prestressed concrete panels supporting a composite cast-in-place concrete deck. For this type of bridge, the panels span between the flanges of the supporting beams and act as formwork for the cast-in-place concrete deck. The three bridges that included panels are the Route 3A Bridge in New Hampshire and the Louetta Road and San Angelo bridges in Texas.

The New Hampshire Route 3A Bridge uses four longitudinal NE 1000 girders supporting 90-mm (3.5-in) -thick precast panels and a 140-mm (5.5-in) -thick cast-in-place composite concrete deck. Girder spacing is 3.51 m (11.5 ft). The cracking in the main span of the bridge consisted of five cracks with a total length of 5.6 m (18.5 ft). This is a low amount of cracking and indicates that the use of precast concrete deck panels is not always a contributing factor in bridge deck cracking.

The Texas Louetta Road Bridge consists of separate northbound and southbound structures. Both structures use precast, prestressed concrete U-beams supporting 3.5-in (90-mm) -thick precast concrete deck panels and a 95-mm (3.75-in) -thick composite cast-in-place concrete deck. Beam spacing varies from 3.51 to 5.06-m (11.5 to 16.6 ft). The panels span between the two top flanges of individual beams as well as between the flanges of adjacent beams. The specified concrete strengths for the cast-in-place decks on the northbound and southbound structures were 28 MPa (4000 psi) at 28 days and 55 MPa (8000 psi) at 28 days, respectively. Measured compressive strengths were about 39 MPa (5700 psi) at 28 days for the northbound structure and about 63 MPa (9100 psi) at 28 days for the southbound structure.

Overall, both structures exhibited a similar and relatively high total cracking density with the northbound having less transverse cracking and more longitudinal cracking than the southbound bridge. Most of the longitudinal and transverse cracking appears to occur above the edges of the precast deck panels and occurs throughout the length of each span. Factors that contribute to this cracking could be shortening of the precast panels as a result of creep and difference in the coefficient of thermal expansion between the cast-in-place concrete and the precast panel concrete. It appears that the different concrete strengths used in the two bridges did not play a significant role in the amount of cracking in the decks.

The Texas San Angelo Bridge consists of separate eastbound and westbound structures. Both structures consist of precast, prestressed concrete I-beams supporting 100-mm (4.0-in) -thick precast concrete deck panels and a 90-mm (3.5-in) -thick composite cast-in-place concrete deck. AASHTO Type IV beams are used for most spans with Texas Type B beams for two short spans. Beam spacing varies from 1.65 to 3.35 m (5.4 to 11.0 ft). For the eastbound structure, high performance concrete was used for the beams, panels, and cast-in-place concrete deck except for the cast-in-place deck of spans 6 through 8. For the westbound structure, high performance concrete was used only for the cast-in-place deck of spans 1 through 5.

Overall, both structures exhibited the largest total crack density of the 19 bridges included in the investigation. However, the total crack density in the eastbound structure was about 60% of that in the westbound structure. In both structures, about 65% of the cracking occurred in the transverse direction. As for the Louetta Road Bridge, the presence of the precast concrete panels influenced the location of the cracks.

Of the 17 spans included in both structures, eastbound spans 1 through 4 exhibited the least total crack density. All four spans are rectangular in plan. Spans 1 through 3 have a constant beam length and spacing. Span 4 has a slightly variable beam length to accommodate a change in the roadway width and skew angle of the bents. Spans 2 through 4 are the longest three spans in the bridge. By contrast, eastbound spans 5 through 7 are shorter, have a larger change in beam spacing and length, and a large skew at the end of span 7. Most of the cracking, which is transverse cracking, occurs above the beams with the longer span lengths. In addition, span 7, although relatively short has a large skew at one end and exhibited the second highest total crack density in all of the bridge spans inspected. Westbound span 9, which has a square abutment on one end and skew bent at the other, had the highest total crack density. These observations indicate that bridge geometry influences the amount of concrete cracking particularly when the geometry results in torsional stresses.

Bridges with Adjacent Box Beams

Two bridges used adjacent precast, prestressed concrete box beams. The Ohio bridge consisted of twelve 1.07-m (42-in) -deep box beams with a 75-mm (3-in) -thick asphalt riding surface. With the exception of three short diagonal cracks, the entire crack pattern consisted of longitudinal cracks. This crack pattern is typical of that observed in adjacent box beam bridges [(40)]. The cracks occur above the edges of the adjacent boxes and are usually caused by a combination of temperature gradients and live load. They are more prevalent in bridges without a strong transverse connection between the box beams.

The Colorado bridge consisted of twenty-four 750-mm (29.5-in) -deep box beams with a 175-mm (6.9-in) -thick cast-in-place concrete deck with a 75- to 100-mm (3- to 4-in) -thick asphalt overlay. No visible leakage on the underside of the box beams was observed during the inspection. The lack of visible cracking above the edges of the box beams may be the result of using a 175-mm (6.9-in) -thick cast-in-place concrete deck that acts as a transverse tie. Most states use a cast-in-place deck thickness of 115 to 150 mm (4.5 to 6 in).

CONCRETE CONSTITUENT MATERIALS AND PROPERTIES

The following analyses were limited to the 14 full-depth, cast-in-place concrete decks and their associated approved mixture designs.

Mixture Proportions

Table’s 4A and 4B presents the approved mixture proportions for all the cast-in-place concrete decks included in the investigation.

Table 4A. Concrete Mix Proportions for Cast-in-Place Concrete Decks (SI Units).[1]

State	**AL**	**CO**	**GA**	**LA**	**NE**	**NH[2]**	**NH[2]**	**NM**	**NC**	**OH[3]**
Bridge Name	AL 199	Yale Ave.	S.R. 920	Charenton	120th St.	Route 104	Route 3A	Rio Puerco	U.S. 401	U.S. 22
w/cm ratio	0.37	0.38	0.34	0.39	0.31	0.38	0.38	0.32	0.33	0.22
Cement Type	II	-	I	IS	IP	II	Blended	I/II	I/II	I
Cement Quantity, kilograms per cubic meter (kg/m^3)	390	418	386	182	445	360	361	408	348	476
Fly Ash Type	C	-	-	-	C	-	-	F	F	F
Fly Ash Quantity, kg/m^3	98	-	-	-	45	-	-	102	104	41
Silica Fume, kg/m^3	-	-	7	-	-	31	31	-	-	52
Ground Granulated Blast-Furnace Slag, kg/m^3	-	-	-	181	-	-	-	-	-	-
Total Cementitious Material, kg/m^3	488	418	393	363	490	391	392	510	452	569
Fine Aggregate, kg/m^3	618	821	821	697	830	706	706	765	593	515
Coarse Aggregate Maximum Size, mm	25	19	19	25	12.5	19	19	12.5	25	9.5
Coarse Aggregate Quantity, kg/m^3	1,103	882	1,008	1,127	830	1,076	1,076	830	1,082	1,021
Water, kg/m^3	180	158	133	141	151	150	150	163	148	125
Air Entrainment, liters per cubic meter (L/m^3)	1.24	0.13	0.63	0.15	0.19	0.23	0.17	0.33	-	1.22
Water Reducer, L/m^3	0.97	-	0.75	-	1.16	0.77	0.77	-	-	-
Retarder, L/m^3	-	-	-	1.42	-	-	-	-	-	-
High-Range Water Reducer, L/m^3	3.79	0.73	5.53	-	5.22	3.05	4.08	2.17	-	7.41

Table 4A. Concrete Mix Proportions for Cast-in-Place Concrete Decks (SI Units).[1] - *Continued*

State	**SD**	**SD**	**TN**	**TN**	**TX (Louetta)**		**TX (San Angelo)**		**VA**	**VA**	**WA**
Bridge Name	I-29 NB	I-29 SB	Porter	Hickman	NB	SB	EB	WB	Route 40	VA Ave.	S.R. 18
w/cm ratio	0.39	0.36	0.36	0.36	0.43	0.35	0.31	0.42	0.40	0.45	0.39
Cement Type	II	II	I	I	I	I	II	II	II	I	I
Cement Quantity, kilograms per cubic meter (kg/m^3)	303	350	293	293	227	281	291	253	195	332	392
Fly Ash Type	-	F	C	C	C	C	C	C	-	F	C
Fly Ash Quantity, kg/m^3	70	74	91	91	88	131	125	110	-	83	45
Silica Fume, kg/m^3	33	-	30	30	-	-	-	-	-	-	-
Ground Granulated Blast-Furnace Slag, kg/m^3	-	-	-	-	-	-	-	-	195	-	-
Total Cementitious Material, kg/m^3	406	424	414	414	315	412	416	363	390	415	436
Fine Aggregate, kg/m^3	652	725	661	661	737	773	809	735	696	595	652
Coarse Aggregate Maximum Size, mm	-	-	25	25	38	25	32	32	25	25	12.5
Coarse Aggregate Quantity, kg/m^3	1,023	969	1,073	1,703	1,101	1,074	1,127	1,101	1,051	1,022	1,008
Water, kg/m^3	157	151	149	149	136	145	130	153	156	187	172
Air Entrainment, liters per cubic meter (L/m^3)	-	-	-	-	0.08	-	0.12	0.12	0.33	0.19	-
Water Reducer, L/m^3	1.58	0.85	-	-	-	-	-	-	2.55	-	0.23
Retarder, L/m^3	-	-	-	-	1.74	0.85	1.08	1.01	-	0.81	-
High-Range Water Reducer, L/m^3	-	-	-	-	-	4.72	6.03	-	0.50-0.77	-	-

[1] Based on approved concrete mix proportions.
[2] Also includes 19.80 liters per cubic meter (l/m^3) of corrosion inhibitor.
[3] Ohio bridge did not have a separate concrete deck. Values are for the abutments.

Table 4B. Concrete Mix Proportions for Cast-in-Place Concrete Decks (English Units).[1]

State	AL	CO	GA	LA	NE	NH[2]	NH[2]	NM	NC	OH[3]
Bridge Name	AL 199	Yale Ave.	S.R. 920	Charenton	120th St.	Route 104	Route 3A	Rio Puerco	U.S. 401	U.S. 22
w/cm ratio	0.37	0.38	0.34	0.39	0.31	0.38	0.38	0.32	0.33	0.22
Cement Type	II	-	I	IS	IP	II	Blended	I/II	I/II	I
Cement Quantity, pounds per cubic yard (lb/yd^3)	658	705	651	306	750	607	608	687	587	803
Fly Ash Type	C	-	-	-	C	-	-	F	F	F
Fly Ash Quantity, lb/yd^3	165	-	-	-	75	-	-	172	175	68.5
Silica Fume, lb/yd^3	-	-	12	-	-	53	52	-	-	87.5
Ground Granulated Blast-Furnace Slag, lb/yd^3	-	-	-	305	-	-	-	-	-	-
Total Cementitious Material, lb/yd^3	823	705	663	611	825	660	660	859	762	959
Fine Aggregate, lb/yd^3	1,042	1,384	1,385	1,176	1,400	1,190	1,190	1,290	1,000	868
Coarse Aggregate Maximum Size, in	1	0.75	0.75	1	0.5	0.75	0.75	0.5	1	0.375
Coarse Aggregate Quantity, lb/yd^3	1,860	1,488	1,700	1,900	1,400	1,815	1,815	1,400	1,825	1,721
Water, lb/yd^3	304	266	225	238	255	253	253	275	250	210
Air Entrainment, fluid ounces per cubic yard (fl oz/yd^3)	32	3.4	16.2	4.0	5	6	4.5	8.6	-	31.6
Water Reducer, fl oz/yd^3	25	-	19.5	-	30	20	19.8	-	-	-
Retarder, fl oz/yd^3	-	-	-	36.7	-	-	-	-	-	-
High-Range Water Reducer, fl oz/yd^3	98	19	143	-	135	79	105.6	56.3	-	191.8

Table 4B. Concrete Mix Proportions for Cast-in-Place Concrete Decks (English Units).[1] - *Continued*

State	SD	SD	TN	TN	TX (Louetta)		TX (San Angelo)		VA	VA	WA
Bridge Name	I-29 NB	I-29 SB	Porter	Hickman	NB	SB	EB	WB	Route 40	VA Ave.	S.R. 18
w/cm ratio	0.39	0.36	0.36	0.36	0.43	0.35	0.31	0.42	0.40	0.45	0.39
Cement Type	II	II	I	I	I	I	II	II	II	I	I
Cement Quantity, pounds per cubic yard (lb/yd^3)	511	590	494	494	383	474	490	427	329	560	660
Fly Ash Type	-	F	C	C	C	C	C	C	-	F	C
Fly Ash Quantity, lb/yd^3	118	124	153	153	148	221	210	185	-	140	75
Silica Fume, lb/yd^3	55	-	50	50	-	-	-	-	-	-	-
Ground Granulated Blast-Furnace Slag, lb/yd^3	-	-	-	-	-	-	-	-	329	-	-
Total Cementitious Material, lb/yd^3	684	714	697	697	531	695	700	612	658	700	735
Fine Aggregate, lb/yd^3	1,100	1,222	1,115	1,115	1,243	1,303	1,365	1,240	1,173	1,004	1,100
Coarse Aggregate Maximum Size, in	-	-	1	1	1.5	1	1.25	1.25	1	1	0.5
Coarse Aggregate Quantity, lb/yd^3	1,725	1,634	1,810	1,810	1,856	1,811	1,900	1,856	1,773	1,724	1,700
Water, lb/yd^3	264	255	251	251	229	244	219	258	263	315	290
Air Entrainment, fluid ounces per cubic yard ($fl\ oz/yd^3$)	-	-	-	-	2.1	-	3.1	3.1	8.5	5	-
Water Reducer, $fl\ oz/yd^3$	41	22	-	-	-	-	-	-	66	-	6
Retarder, $fl\ oz/yd^3$	-	-	-	-	45	22	28	26	-	21	-
High-Range Water Reducer, $fl\ oz/yd^3$	-	-	-	-	-	122	156	-	13-20	-	-

[1] Based on approved concrete mix proportions.
[2] Also includes 4.0 gallons per cubic yard (gal/yd^3) of corrosion inhibitor.
[3] Ohio bridge did not have a separate concrete deck. Values are for the abutments.

Water-to-Cementitious Materials Ratio

Analysis was performed comparing the w/cm ratio to crack densities for the bridge decks. Figure 21 presents the w/cm ratios for the individual bridge decks and the corresponding crack densities.

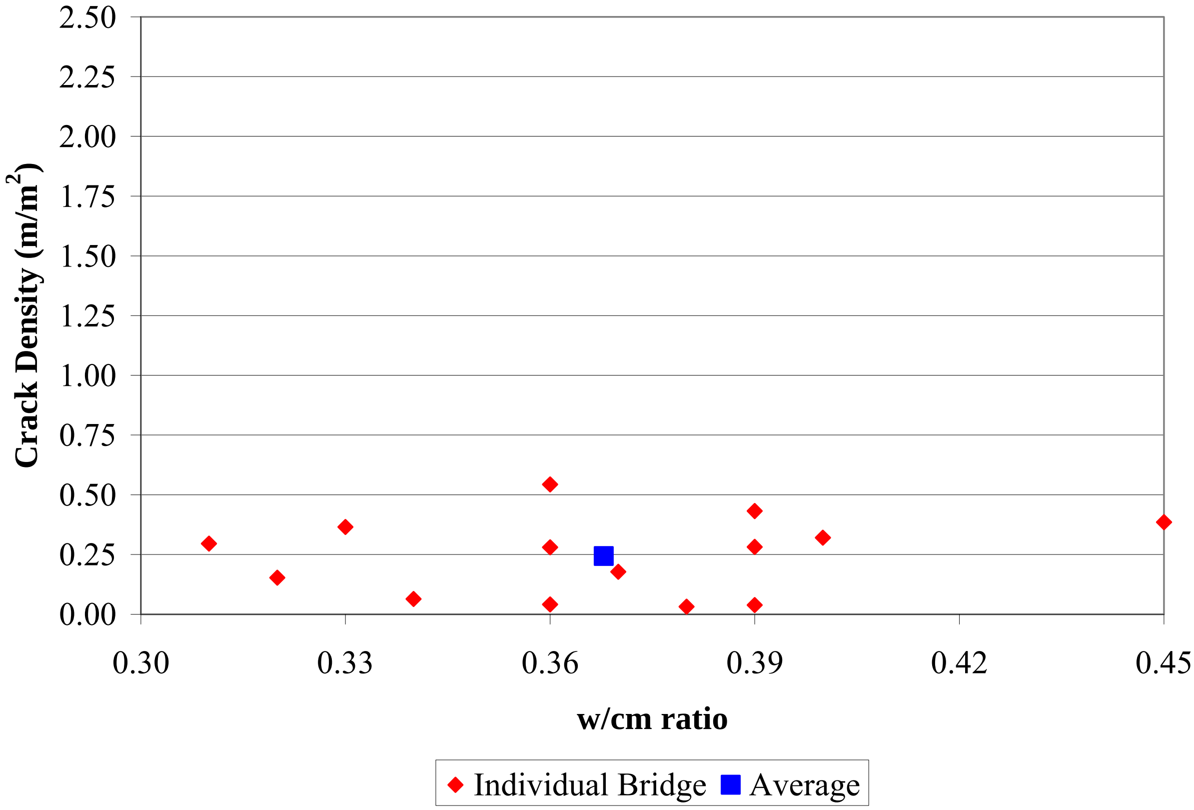

Figure 21. Chart. Crack Density vs. w/cm ratio for all Bridge Decks.

The average crack density for all of the bridge decks combined was 0.244 m/m^2 (0.074 ft/ft^2). There does not appear to be a correlation between the w/cm ratio and crack densities for the entire set of bridge decks. Generally, a reasonable w/cm ratio for bridge decks is in the range of 0.37 to 0.45. The w/cm ratios for this study ranged from 0.31 to 0.45. The bridge decks were then divided into groups based on the w/cm ratio to determine if there was a correlation between w/cm ratio ranges and crack densities. Also, to observe if there was a certain range of w/cm ratios that performed better related to cracking. The bridge decks were divided into the following groups:

Group 1: w/cm ratio between 0.30 and 0.35,
Group 2: w/cm ratio between 0.35 and 0.40,
Group 3: w/cm ratio between 0.40 and 0.45.

The bridge deck with a w/cm ratio of 0.40 was included in Group 3.

Figure 22 presents the w/cm ratio versus crack densities for Group 1.

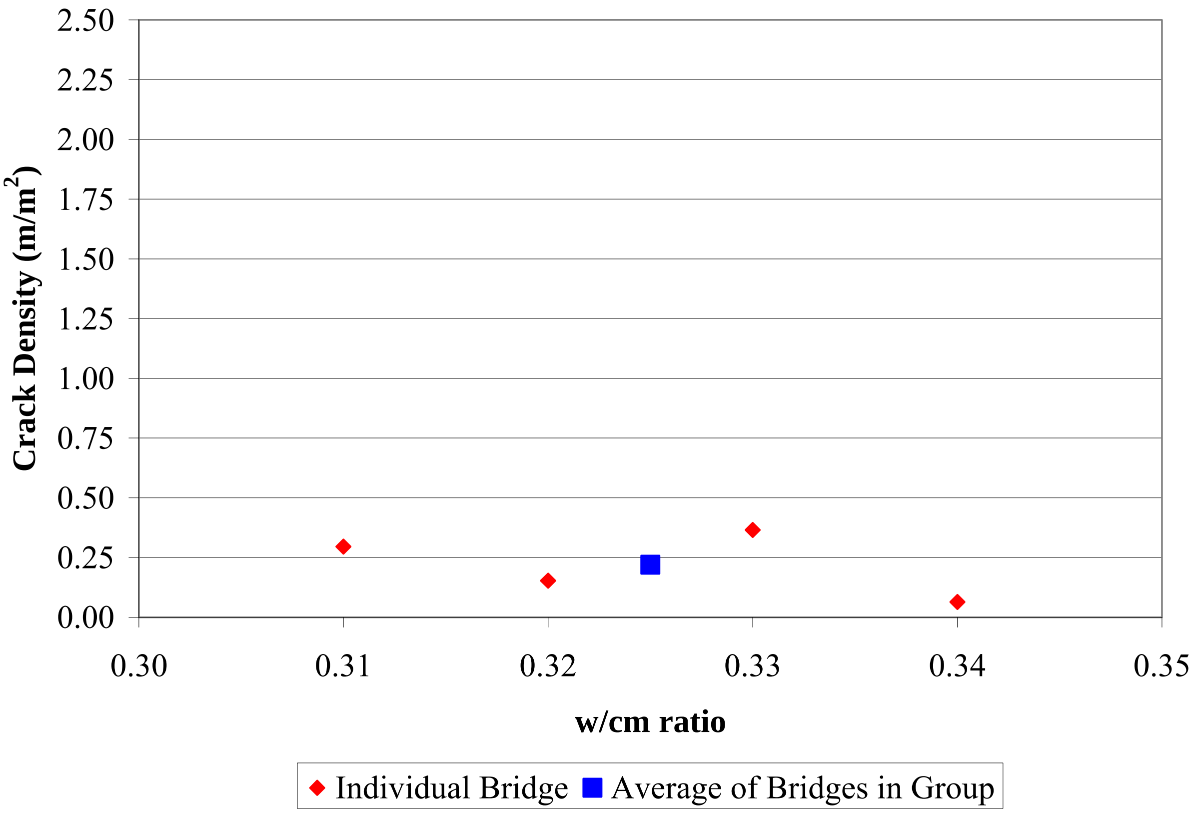

Figure 22. Chart. Crack Densities for w/cm ratio 0.30 to 0.35.

The range of crack densities for Group 1 was 0.064 to 0.365 m/m^2 (0.020 to 0.111 ft/ft^2), while the average for the group was 0.220 m/m^2 (0.067 ft/ft^2). The average crack density for Group 1 was lower than the overall average for all of the bridge decks.

Figure 23 presents the w/cm ratio versus crack densities for Group 2.

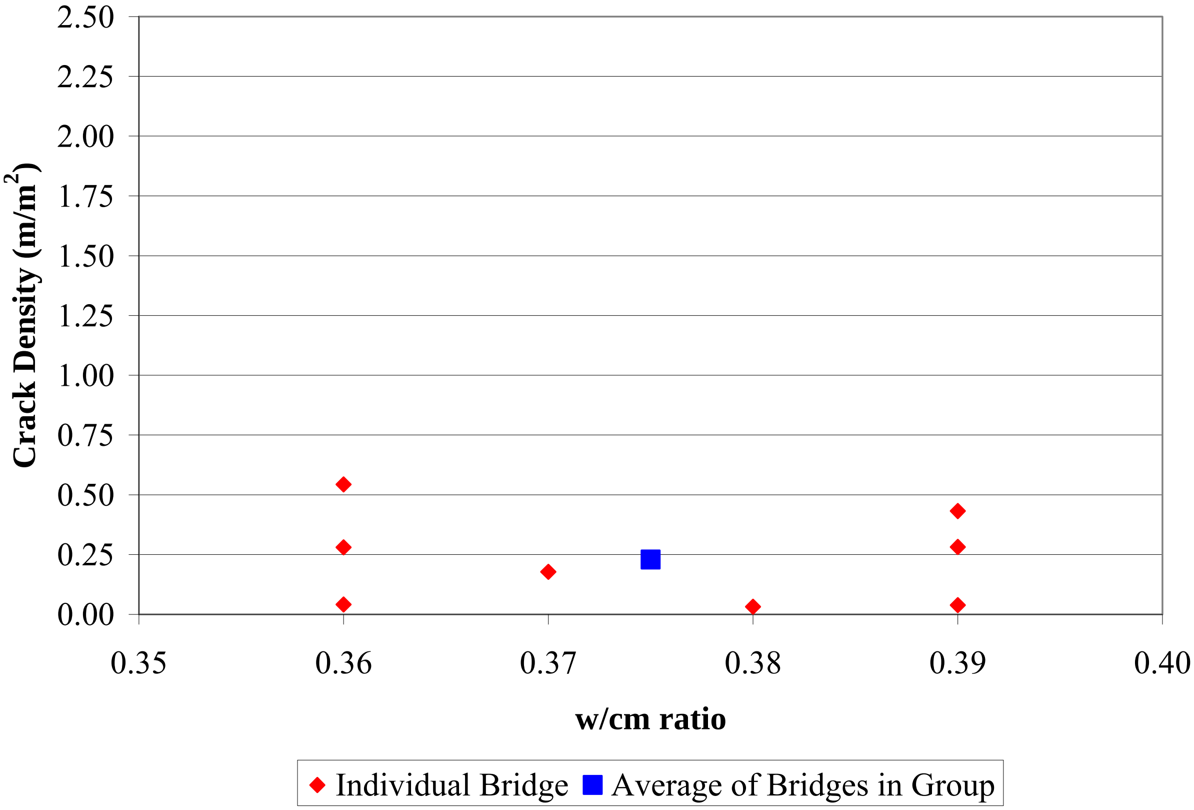

Figure 23. Chart. Crack Densities for w/cm ratio 0.35 to 0.40.

The range of crack densities for Group 2 was 0.032 to 0.544 m/m^2 (0.010 to 0.166 ft/ft^2), while the average for the group was 0.228 m/m^2 (0.069 ft/ft^2). The average crack density for Group 2 was lower than the overall average for all of the bridge decks.

Figure 24 presents the w/cm ratio versus crack densities for Group 3.

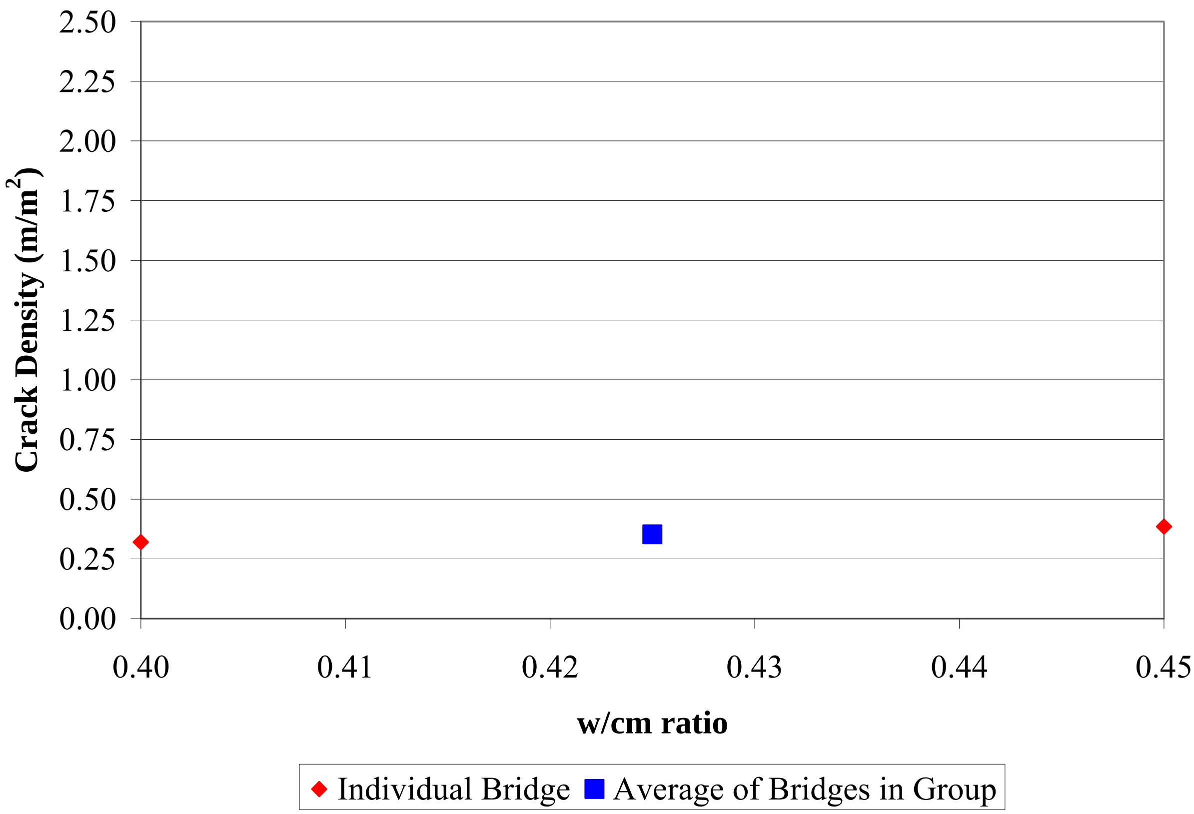

Figure 24. Chart. Crack Densities for w/cm ratio 0.40 to 0.45.

The range of crack densities for Group 3 was 0.320 to 0.385 m/m^2 (0.098 to 0.117 ft/ft^2), while the average for the group was 0.352 m/m^2 (0.107 ft/ft^2). The average crack density for Group 3 was significantly higher than the overall average for all of the bridge decks.

In summary, there does not appear to be a strong correlation between the w/cm ratio and the crack densities observed on all of the bridge decks. However, when the bridge decks are divided into groups based on w/cm ratio ranges, there are certain ranges that perform better than others as related to crack densities. Groups 1 and 2 exhibited lower average crack densities than Group 3. Although the Group 1 bridge decks had an average crack density similar to the overall average for all of the bridge decks, the w/cm ratio was relatively low for most of them and in some cases the cementitious materials contents were considered in the high range. The Group 3 bridge decks had the higher w/cm ratio range and exhibited the greatest average crack densities.

Cementitious Material Content

Analysis was performed comparing the cementitious material contents to crack densities for all of the bridge decks. Figure 25 presents the cementitious material contents for the individual bridge decks and the corresponding crack densities.

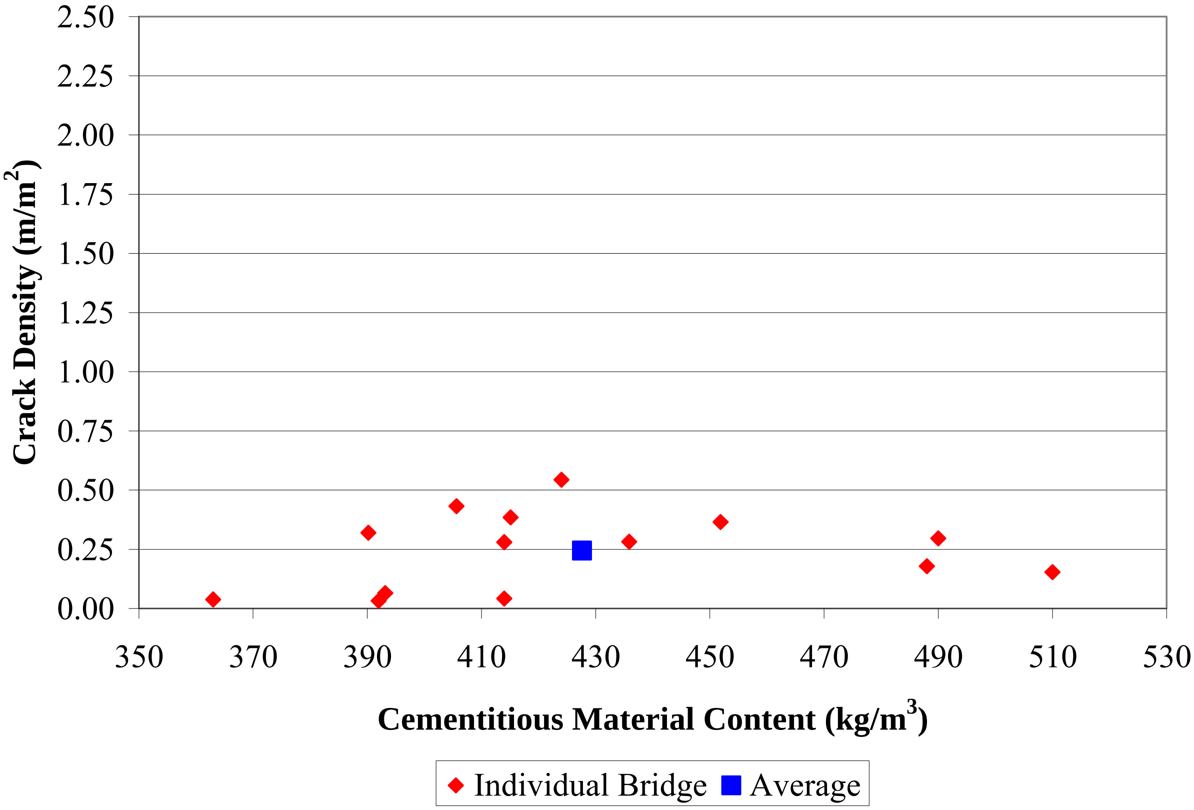

Figure 25. Chart. Crack Density vs. Cementitious Material Content for all Bridge Decks.

The average cementitious material content for all of the bridge decks was 428 kg/m^3 (722 lb/yd^3). As with the w/cm ratio analysis, there does not appear to be a strong correlation between the cementitious material content and crack densities for the entire set of bridge decks. The bridge decks were again divided into groups according to the cementitious material content of the individual decks. The range of cementitious material contents was between 363 and 510 kg/m^3 (611 lb/yd^3 859 lb/yd^3). The decks were divided into the following groups:

Group 1: cementitious material contents between 356 and 415 kg/m^3 (600 and 700 lb/yd^3),
Group 2: cementitious material contents between 415 and 475 kg/m^3 (700 and 800 lb/yd^3),
Group 3: cementitious material contents greater than 475 kg/m^3 (800 lb/yd^3).

The bridge deck with a cementitious materials content of 415 kg/m^3 (700 lb/yd^3) was included in Group 2.

Figure 26 presents the cementitious material content versus crack densities for Group 1.

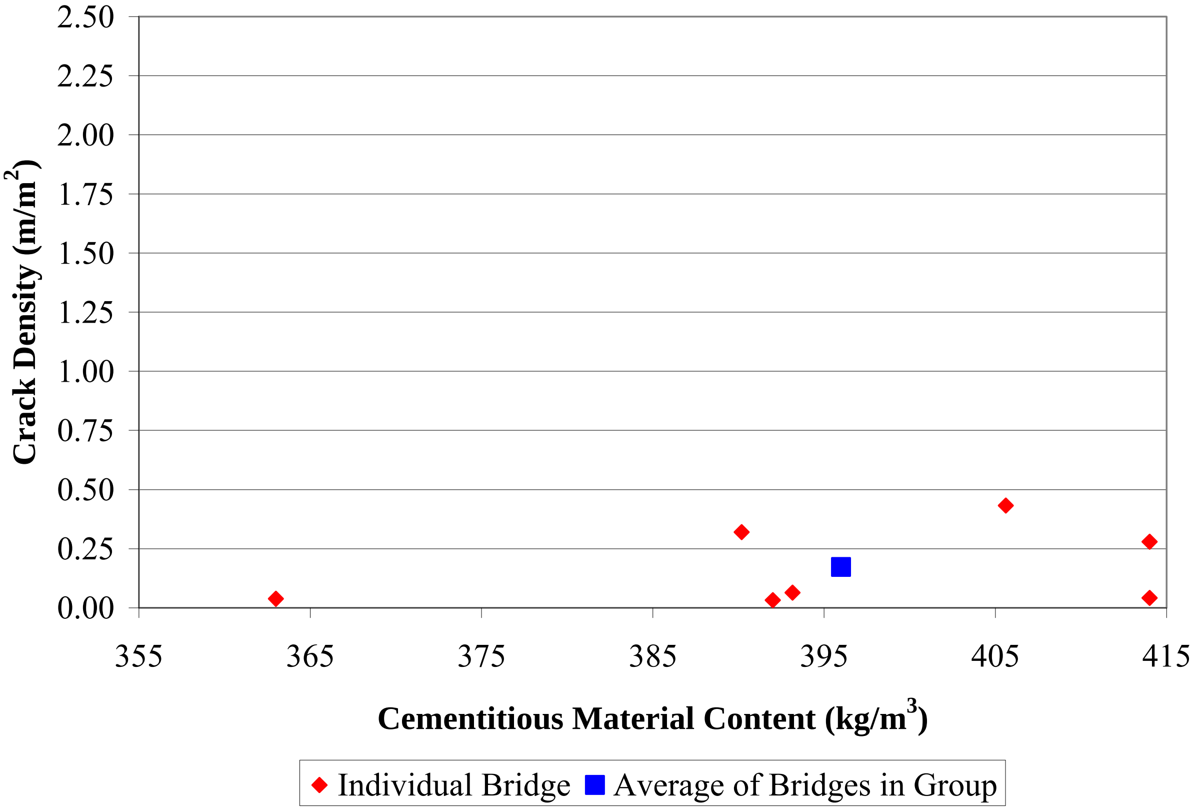

Figure 26. Chart. Crack Densities for Cementitious Material Contents between 356 and 415 kg/m^3 (600 and 700 lb/yd^3).

The range of crack densities for the bridge decks in Group 1 was between 0.032 and 0.432 m/m^2 (0.010 and 0.132 ft/ft^2), while the average crack density for Group 1 was 0.173 m/m^2 (0.053 ft/ft^2). The overall average crack density for the bridge decks in Group 1 was lower than the average crack densities for all of the bridge decks combined.

Figure 27 presents the cementitious material content versus crack densities for Group 2.

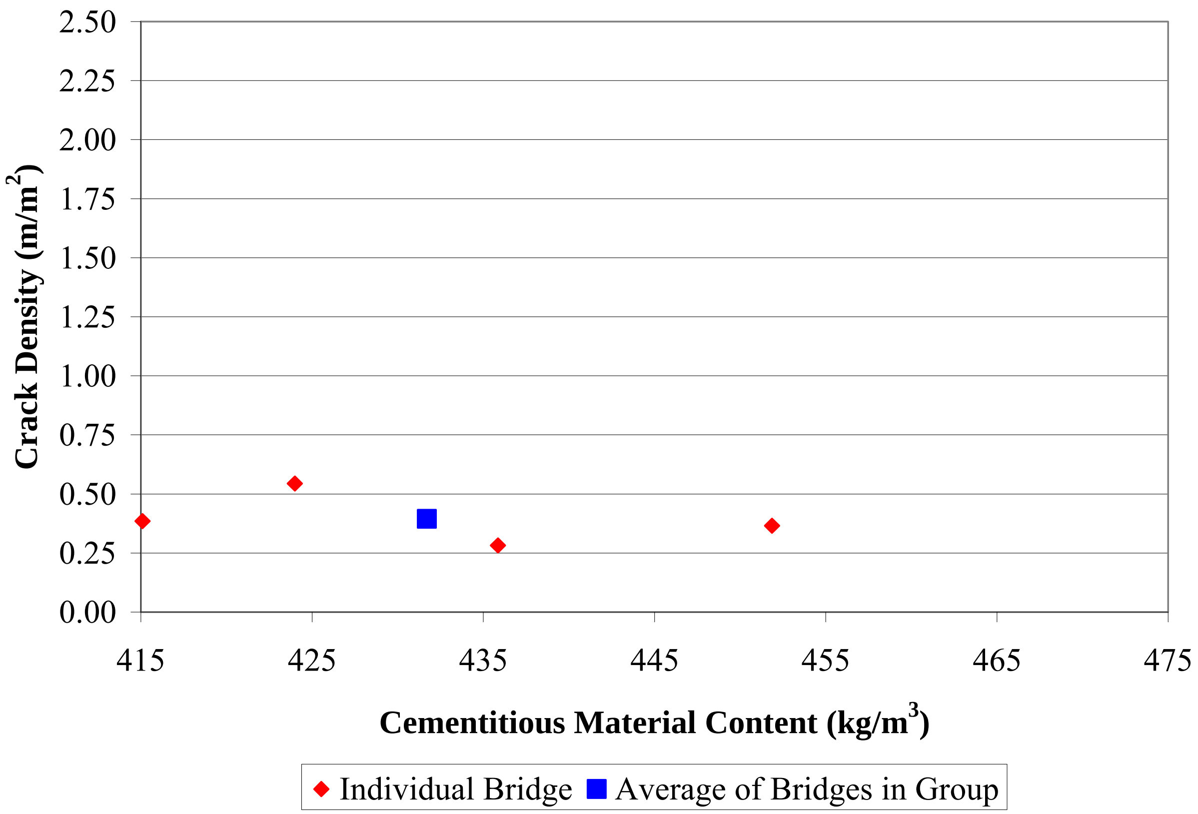

Figure 27. Chart. Crack Densities for Cementitious Material Contents between 415 and 475 kg/m^3 (700 and 800 lb/yd^3).

The range of crack densities for the bridge decks in Group 2 was between 0.282 and 0.544 m/m^2 (0.086 and 0.166 ft/ft^2), while the average crack density for Group 2 was 0.394 m/m^2 (0.120 ft/ft^2). The overall average crack density for the bridge decks in Group 2 was higher than the average crack densities for all of the bridge decks combined.

Figure 28 presents the cementitious material content versus crack densities for Group 3.

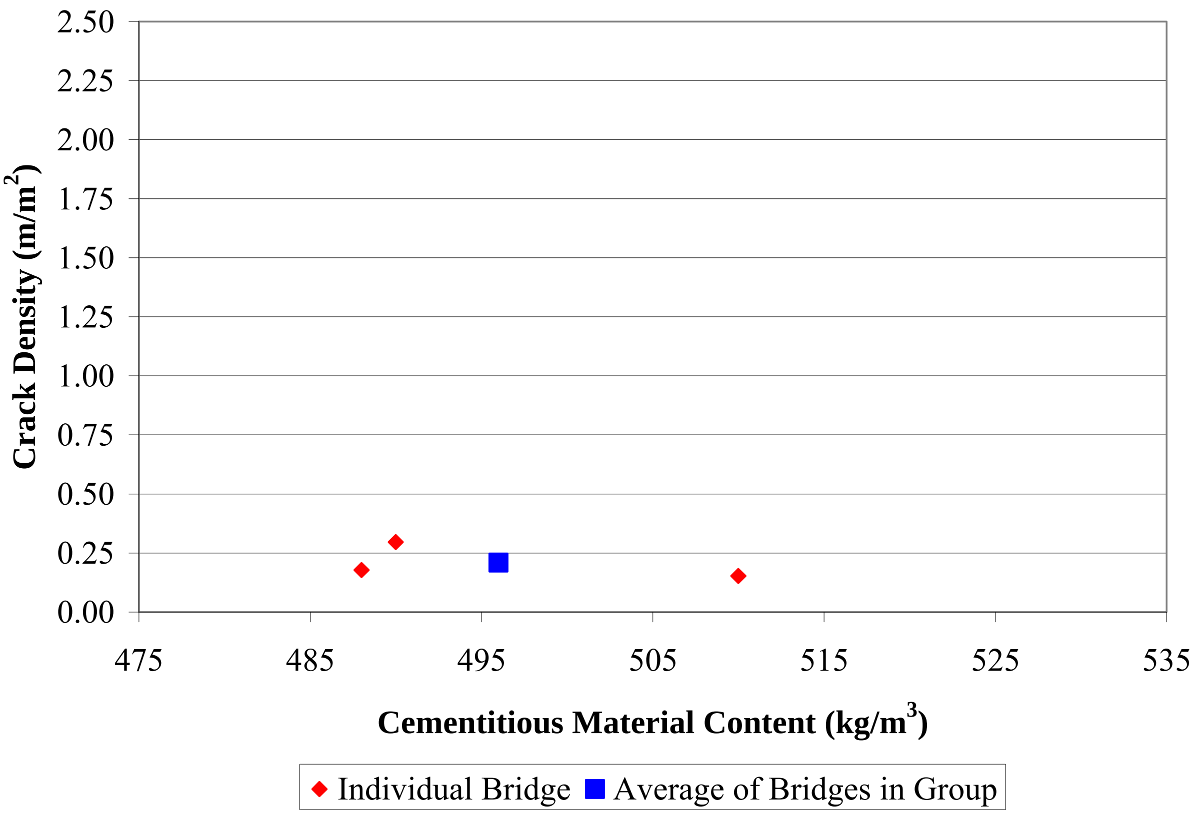

Figure 28. Chart. Crack Densities for Cementitious Material Contents Greater than 475 kg/m^3 (800 lb/yd^3).

The range of crack densities for the bridge decks in Group 3 was between 0.153 and 0.296 m/m^2 (0.047^2 and 0.090 ft/ft^2), while the average crack density for Group 3 was 0.209 m/m^2 (0.064 ft/ft^2). The overall average crack density for the bridge decks in Group 3 was lower than the average crack densities for all of the bridge decks combined. The cementitious material contents in this group were extremely high. The range was between 488 and 510 kg/m^3 (823 and 859 lb/yd^3). Although they exhibited low average crack densities, the excessive cementitious material content could be cost prohibitive in some cases.

From these data, there again does not appear to be a strong correlation between cementitious material content and average crack density. However, when the bridge decks are divided into groups based on cementitious material content, there are some groups that perform better than others. Typical concrete bridge deck mixtures have a cementitious material content of between 356 and 415 kg/m^3 (600 and 700 lb/yd^3). The Group 1 bridge decks, which had cementitious material contents between 356 and 415 kg/m^3 (600 and 700 lb/yd^3) performed better than the other two groups with higher cementitious material contents as it related to crack densities.

Summary of Water-to-Cementitious Materials Ratio and Cementitious Materials Content

The analyses performed relating w/cm ratio and cementitious material content to average crack densities revealed that by dividing the bridge decks into groups, some of the groups performed better than others related to average crack densities. From these data, it appeared that the average crack densities were low for the bridge decks with a w/cm ratio between 0.35 and 0.40 and cementitious material contents between 356 and 415 kg/m^3 (600 and 700 lb/yd^3). An analysis was then performed using only those bridge decks that were within the ranges of both of these parameters.

Six bridge decks had w/cm ratio and cementitious material contents that were within the above ranges. Table 5 presents the data for the six bridge decks.

Table 5. Bridge Decks with w/cm ratio 0.35 to 0.40 and Cementitious Material Content between 356 and 415 kg/m^3 (600^3 and 700 lb/yd^3).

Bridge	**w/cm ratio**	**Cementitious Material Content, kg/m^3 (lb/yd^3)**	**Average Crack Density, m/m^2(ft/ft^2)**
New Hampshire (3A)	0.38	392 (660)	0.032 (0.010)
Louisiana	0.39	363 (611)	0.038 (0.012)
Tennessee (Hickman)	0.36	414 (697)	0.041 (0.012)
Tennessee (Porter)	0.36	414 (697)	0.280 (0.085)
Virginia (Brookneal)	0.40	390 (658)	0.320 (0.098)
South Dakota (NB)	0.39	406 (685)	0.432 (0.132)
Average	**0.38**	**396 (668)**	**0.191 (0.058)**

From the data presented in Table 5, the average crack density of the six bridge decks was 0.191 m/m^2 (0.058 ft/ft^2). This value is significantly lower than the average crack density of 0.412 m/m^2 (0.123 ft/ft^2) for all of the bridge decks in the study and less than the average crack density of 0.244 m/m^2 (0.074 ft/ft^2) for all bridges with a full-depth, cast-in-place concrete deck. Also, the average cementitious material content for the six bridge decks was 396 kg/m^3 (668 lb/yd^3), which is in the typical range of bridge deck concrete mixtures.

The average w/cm ratio and cementitious material content for the above example are reasonable, and are readily producible. These data show that if these types of concrete mixtures are fabricated, placed, and cured properly, they can aid in reducing the incidence of cracking in bridge decks.

Pozzolans and Slag Cement

Analysis was performed looking at the use of pozzolans and slag cement and the associated crack densities. There were seven bridges that used portland cement and fly ash, the fly ash replacement ranged between 9 and 23 percent. The average fly ash replacement was 17 percent.
The average crack density for the bridge decks using fly ash was 0.315 m/m^2 (0.096 ft/ft^2), the range of crack densities for these bridge decks was between 0.153 and 0.544 m/m^2 (0.047 and 0.166 ft/ft^2).

Two bridge decks used portland cement and silica fume, the silica fume replacements were 2 and 8 percent for an average replacement of 5 percent. The average crack density was 0.048 m/m^2 (0.015 ft/ft^2).

Two bridge deck used slag cement; the replacement was 50 percent for both bridges. The average crack density was 0.179 m/m^2 (0.055 ft/ft^2). Three bridge decks used ternary mixtures using portland cement, fly ash, and silica fume. The average replacements were 20 percent fly ash and 7 percent silica fume. The average crack densities for these bridge decks were 0.251 m/m^2 (0.076 ft/ft^2).

Figure 29 presents the crack density data for the mixtures containing pozzolans and slag cement.

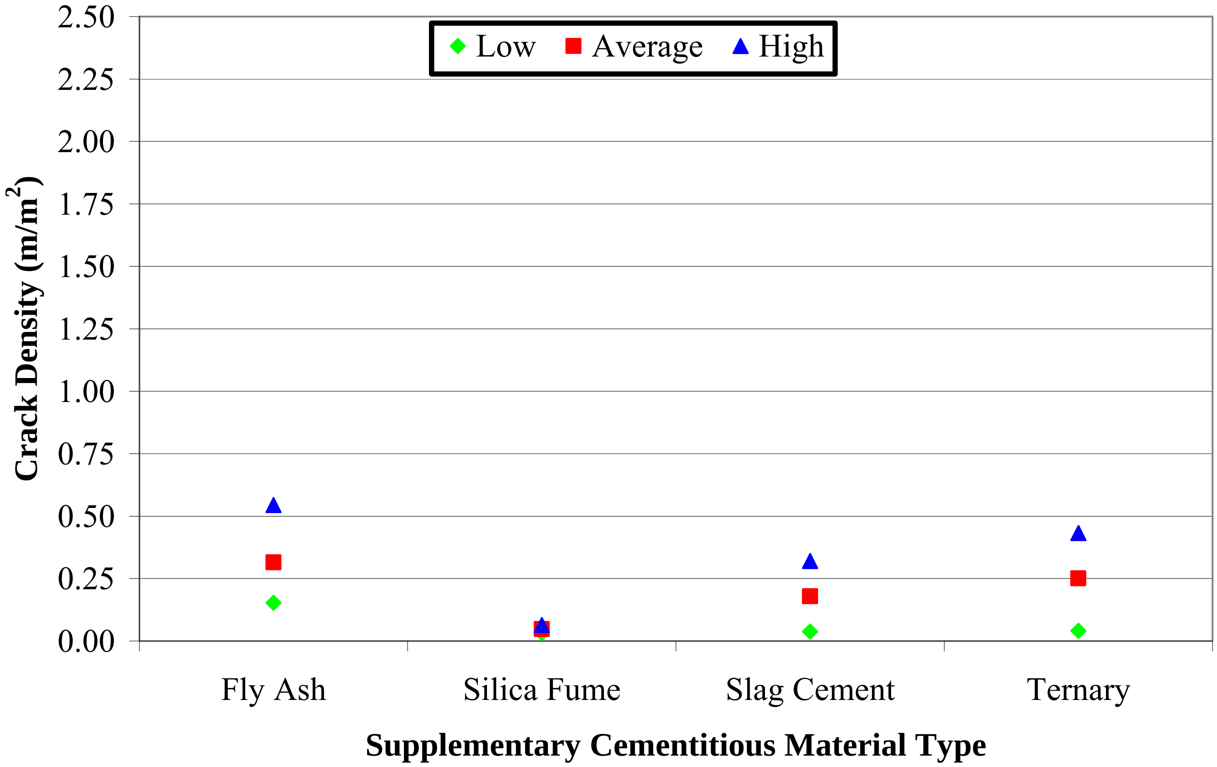

Figure 29. Chart. Crack Density for Supplementary Cementitious Materials.

In summary, the bridge decks using silica fume had a lower crack density than the bridge decks using fly ash, slag cement, and ternary mixtures. The bridge decks using fly ash exhibited the highest crack densities as a group.

Measured Concrete Properties

Tables 6A and 6B present the measured plastic and hardened concrete properties for the cast-in-place decks from the actual production concrete.

Table 6A. Measured Structural Concrete Properties for Cast-in-Place Concrete Decks (SI Units).[1]

State	**AL**	**CO**	**GA**	**LA**	**NE**	**NH**	**NH**	**NM**	**NC**	**OH[2]**
Bridge Name	AL 199	Yale Ave.	S.R. 920	Charenton	120th St.	Route 104	Route 3A	Rio Puerco	U.S. 401	U.S. 22
Slump, mm	147	97	102-178	-	-	76-127	135	64-216	102-127	114
Air Content, %	4.7	5.5	3.2-6.5	-	-	4-5.8	6	4.5-8.3	5.7-6.8	6.5
Unit Weight, kg/m^3	-	2297	2390	-	-	2307-2355	2361	2146-2403	-	2259
Curing Type	Wet	Wet	Wet	Wet	Wet	Wet	Wet	Wet	Wet	Wet
Curing Duration, days	7	5	7	7	-	5.7	7	14	7	7
Compressive Strength										
7 Days, MPa	41.4	30.2	34.7	21.7	50.0	47.5	48.9	35.3	-	-
28 Days, MPa	50.8	36.6	42.9	37.8	66.2	62.1	62.0	42.4	49.3 NB	59.9
56 Days, MPa	-	41.0	53.3	49.9	71.9	-	62.8	51.7	-	-

State	**SD**	**SD**	**TN**	**TN**	**TX (Louetta)**		**TX (San Angelo)**		**VA**	**VA**	**WA**
Bridge Name	I-29 NB	I-29 SB	Porter	Hickman	NB	SB	EB	WB	Route 40	VA Ave.	S.R. 18
Slump, mm	-	64	-	-	102	178	188	-	145	91	84
Air Content, %	-	6.7	-	-	3.8	0	6.3	4.7	7.0	5.8	5.6
Unit Weight, kg/m^3	-	2291	-	-	2291	2403	-	2323	-	-	-
Curing Type	Wet	Wet	Wet	Wet	Wet	Wet	Wet	Wet	Wet	Wet	Wet
Curing Duration, days	7	7	7	7	10	10	10	10	7	7	14
Compressive Strength											
7 Days, MPa	35.3	33.9	34.1	29.6	-	-	41.7	-	37.1	29.4	-
28 Days, MPa	48.7	42.5	57.0	44.5	39.3	62.7	50.6	42.2	45.5	37.2	37.8
56 Days, MPa	-	-	60.0	49.6	39.3	67.1	-	-	-	46.2	-

[1] Based on production concrete.
[2] Ohio bridge did not have a separate concrete deck. Values are for the abutments.

Table 6B. Measured Structural Concrete Properties for Cast-in-Place Concrete Decks (English Units).[1]

State	AL	CO	GA	LA	NE	NH	NH	NM	NC	OH[2]
Bridge Name	AL 199	Yale Ave.	S.R. 920	Charenton	120th St.	Route 104	Route 3A	Rio Puerco	U.S. 401	U.S. 22
Slump, inches	5.8	3.8	4-7	-	-	3-5	5.3	2.5-8.5	4-5	4.5
Air Content, %	4.7	5.5	3.2-6.5	-	-	4-5.8	6	4.5-8.3	5.7-6.8	6.5
Unit Weight, lb/ft^3	-	143.4	149.2	-	-	144-147	147.4	134-150	-	141
Curing Type	Wet	Wet	Wet	Wet	Wet	Wet	Wet	Wet	Wet	Wet
Curing Duration, days	7	5	7	7	-	5.7	7	14	7	7
Compressive Strength										
7 Days, psi	6,010	4,390	5,040	3,150	7,250	6,890	7,100	5,120	-	-
28 Days, psi	7,370	5,310	6,220	5,490	9,610	9,020	9,000	6,160	7,150 NB	8,690
56 Days, psi	-	5,950	7,740	5,790	10,430	-	9,120	7,500	-	-

State	SD	SD	TN	TN	TX (Louetta)		TX (San Angelo)		VA	VA	WA
Bridge Name	I-29 NB	I-29 SB	Porter	Hickman	NB	SB	EB	WB	Route 40	VA Ave.	S.R. 18
Slump, inches	-	2.5	-	-	4	7	7.4	-	5.7	3.6	3.3
Air Content, %	-	6.7	-	-	3.8	0	6.3	4.7	7.0	5.8	5.6
Unit Weight, lb/ft^3	-	143	-	-	143	150	-	145	-	-	-
Curing Type	Wet	Wet	Wet	Wet	Wet	Wet	Wet	Wet	Wet	Wet	Wet
Curing Duration, days	7	7	7	7	10	10	10	10	7	7	14
Compressive Strength											
7 Days, psi	5,120	4,920	4,950	4,290	-	-	6,050	-	5,390	4,260	-
28 Days, psi	7,070	6,170	8,270	6,460	5,700	9,100	7,350	6,120	6,600	5,400	5,490
56 Days, psi	-	-	8,710	7,200	5,700	9,740	-	-	-	6,710	-

[1] Based on production concrete.
[2] Ohio bridge did not have a separate concrete deck. Values are for the abutments.

For the cast-in-place decks, the specified design compressive strengths ranged from 28 to 55 MPa (4,000 to 8,000 psi) at 28 days. The actual compressive strengths all exceeded the specified design compressive strengths.

The average 28-day compressive strength for the cast-in-place decks was 47.5 MPa (6,890 psi). The range for all of the decks was 37.2 to 66.2 MPa (5,400 to 9,610 psi) at 28 days. An analysis comparing ranges of compressive strengths and crack densities was performed in which the range of compressive strengths was divided into four groups: 34–41 MPa (5,000 – 6,000 psi), 41 – 48 MPa (6,000 – 7,000 psi), 48 – 55 MPa (7,000 – 8,000 psi), and 55+ MPa (8,000+ psi). The average crack densities for each group were 0.235, 0.224, 0.325, and 0.203 m/m^2 (0.072, 0.068, 0.099, and 0.062 ft/ft^2), respectively. The average crack densities are similar regardless of compressive strength. In general, there does not appear to be a correlation between the 28-day compressive strengths and the crack densities for the bridge decks.

Durability Properties

Table 7 presents the measured durability properties for the cast-in-place concrete decks. Figure 30 presents a comparison of the total crack densities and the rapid chloride permeability values for the cast-in-place concrete decks.

Table 7. Measured Durability Properties for Cast-in-Place Concrete Decks .[1]

State	AL	CO	GA	LA	NE	NH	NH	NM	NC	OH[2]
Bridge Name	AL 199	Yale Ave.	S.R. 920	Charenton	120th St.	Route 104	Route 3A	Rio Puerco	U.S. 401	U.S. 22
Cast-in-Place Concrete Decks										
Air Content, %	4.7	5.5	3.2-6.5	-	-	4.0-5.8	6.0o	4.5-8.2	5.7-6.8	-
Chloride Permeability, C	2,870	5,597	3,963	1,390	589	753	1,060	-	-	-
Age, days	56	-	56	56	-	56	56	-	-	-
Freeze-Thaw Resistance, %	92	-	-	-	-	97[2]	-	-	-	-
Scaling Resistance	-	-	-	-	-	0-1	-	-	-	-
Abrasion Resistance, mm (inches)	-	-	-	-	-	-	-	-	-	-

State	SD	SD	TN	TN	TX (Louetta)		TX (San Angelo)		VA	VA	WA
Bridge Name	I-29 NB	I-29 SB	Porter	Hickman	NB	SB	EB	WB	Route 40	VA Ave.	S.R. 18
Cast-in-Place Concrete Decks											
Air Content, %	-	6.8	-	-	3.8	0	6.3	4.7	7.0	5.8	5.6
Chloride Permeability, C	461	1,058	1,297	317[3]	1,730[4]	900[4]	703[4]	2,573[4]	778	1,457	2,645
Age, days	90	-	56	28	56	56	56	56	28	28[3]	>210
Freeze-Thaw Resistance, %	-	-	-	-	-	-	97.9	97.3	-	-	-
Scaling Resistance	-	-	-	-	-	-	2-3	0	-	0-1	-
Abrasion Resistance, mm (inches)	-	-	-	-	-	-	1.0 (0.04)	1.8 (0.07)	-	-	[5]

[1] Based on production concrete.
[2] ASTM C666 procedure A or AASHTO T 161 procedure A.
[3] Includes 21 days at 100 ºF.
[4] ASTM standard cure.
[5] Reported as a weight loss of 3.33 grams.

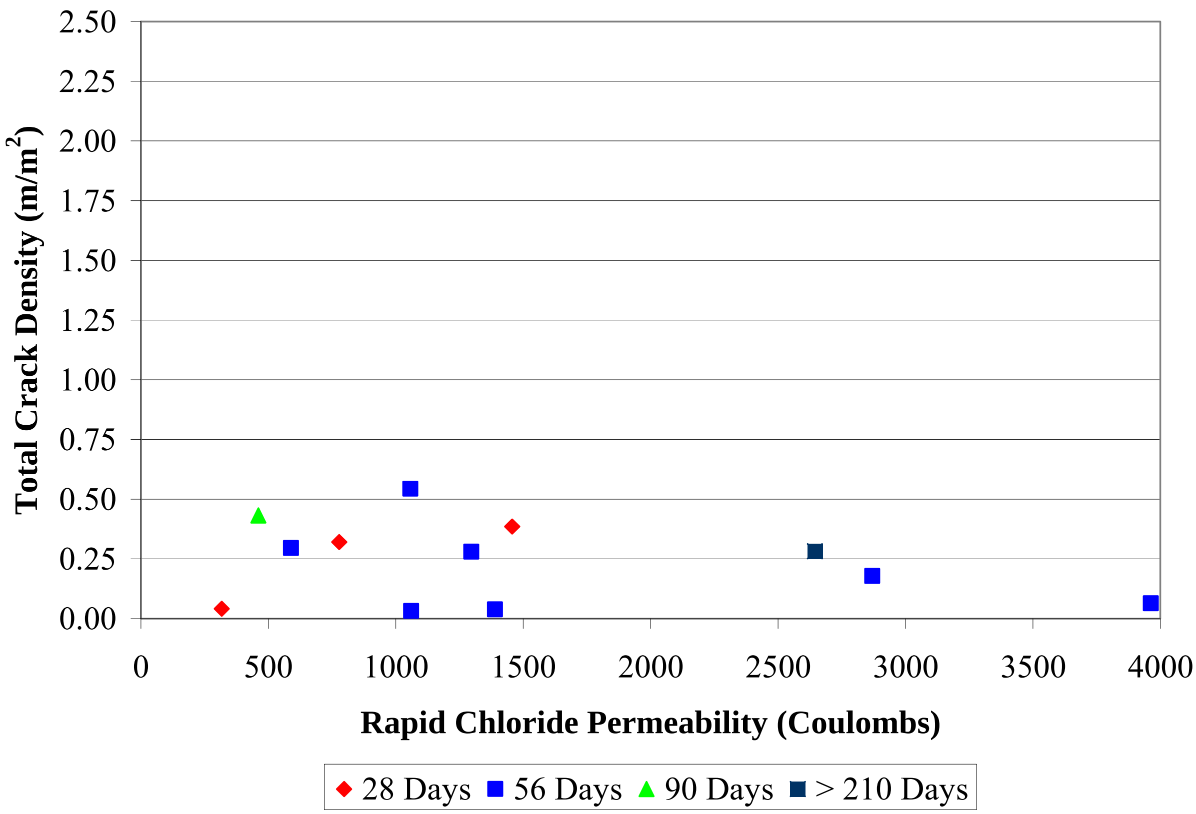

Figure 30. Total Crack Density (m/m^2) versus Rapid Chloride Permeability (Coulombs).

From the data presented in Table 7, most of the durability measurements are within specified ranges. For the cast-in-place decks, the permeability values were mostly within specified ranges. Analysis was performed comparing the range of coulomb values to the average crack densities of the bridge decks. Table 8 presents the classification of chloride ion permeability according to ASTM C1202 (AASHTO T277).

Table 8. Classification of Chloride Ion Permeability.

Charge Passed (Coulombs)	Chloride Permeability	Typical of
> 4000	High	High water-cement ratio, conventional (> 0.6) PCC
2000 – 4000	Moderate	Moderate water-cement ratio, conventional (0.4-0.5) PCC
1000 – 2000	Low	Latex-modified concrete Internally sealed concrete
100 – 1000	Very Low	Polymer impregnated concrete
< 100	Negligible	Polymer concrete

The rapid chloride permeability testing was performed using various techniques. Some of the samples were tested at 28 days after an accelerated cure, while others were tested at 56 days without an accelerated cure. For this analysis, the samples tested at 56 days without accelerated curing were considered. For coulomb values less than 1,000, the average crack density was 0.296 m/m^2 (0.090 ft/ft^2). For coulomb values between 1,000 and 2,000, the average crack density was 0.224 m/m^2 (0.068 ft/ft^2), and for values greater than 2,000, the average crack density was 0.121 m/m^2 (0.037 ft/ft^2). From these data, the bridge decks with coulomb values less than 2,000 at 56 days appear to have similar average crack densities. The bridge decks with the coulomb values above 2,000 had a lower average crack density. However, the crack density for the South Dakota (SB) bridge deck was significantly higher than the other crack densities for the group with coulomb values between 1,000 and 2,000. If the South Dakota (SB) crack density is not considered for the group, the average crack density is 0.117 m/m^2 (0.036 ft/ft^2). In general, the values were in the very low to moderate range. This suggests that the concrete material is considered to be durable and the associated cracking is probably not due to permeability considerations.

ENVIRONMENTAL CONDITIONS

The range of environmental conditions varied for the bridges as they were spread out over different regions across the country. Some of the bridges were in an environment that had negligible freezing and thawing, such as Alabama, Louisiana, and Texas. Other bridges were in areas that had moderate to significant freezing and thawing cycles. In some cases, such as the Texas bridges, the environmental conditions are negligible related to freezing and thawing cycles, yet there was significant cracking in the decks. The other bridge decks in similar environmental conditions exhibited significantly less cracking. There were also some bridge decks that were in more severe environments that exhibited relatively low crack densities. There did not appear to be a correlation between the environmental conditions and the cracking observed in the bridge decks.

CONSTRUCTION PRACTICES

The Alabama bridge consists of seven equal simple spans of 33.9 m (114 ft). Almost 100 percent of the cracks occur in the transverse direction and are located in the quarter-span lengths at the end of each span. The middle half of each span was relatively free of cracks. The center portion of each bridge was cast first and the quarter lengths were cast several days later. The measured properties of the concrete used in the quarter and center lengths were similar. The casting of the quarter lengths after the center would induce compressive stresses in the deck of the center portion and may explain why the center portion is relatively crack-free. Placement sequence can have an influence on bridge deck cracking.

The two bridges in Tennessee have similar span lengths and structural systems and were constructed with similar materials using similar specifications. Measured compressive strengths of the deck concrete were 60 and 50 MPa (8,700 and 7,200 psi) at 56 days for the Porter Road and Hickman Road bridges, respectively. The Porter Road Bridge deck was cast in January 2000 when the ambient temperature at time of placement was 2-4 °C (35-40 °F). Heaters were used on the Porter Road Bridge. The Hickman Road Bridge was cast in May 2000 when the ambient temperature at time of placement was 21 °C (70 °F). The Porter Road Bridge deck had almost seven times the amount of cracking as the Hickman Road Bridge deck. Most of the cracks were transverse and on a line along the middle of the deck. Some diagonal cracks were present at the skewed abutments. It is possible that the cracks were the result of differential temperatures between the heated deck and the cooler beams. In general, there was no correlation between the total crack density and the time of year when the decks were cast.

The specifications for all of the bridge decks except Ohio required wet or moist curing with a curing period that ranged from 4 to 14 days. Interestingly, the shortest curing period of 4 days was specified for the New Hampshire Route 104 Bridge, which had a low amount of cracking. For all bridges, it is unknown how well the actual curing was performed and how long the concrete was actually cured. The specification on both of Virginia's bridges required a 7-day moist cure. In addition, the specification for the Route 40 Bridge required application of a curing compound after removal of the plastic sheeting and burlap. The total crack density on the Route 40 Bridge was about 60 percent of that on the Virginia Avenue Bridge.

CHAPTER 6. OVERALL CONCLUSIONS

From the information gathered from this investigation, the following conclusions can be made:

1. In general, it appears that the concrete material has performed well. There were no indications of alkali-silica reaction (ASR), sulfate attack, or other deleterious reactions. There was also no significant spalling or delamination observed on the bridge decks. There was some spalling along the edges of some cracks.

2. When the structural system of the bridge included skewed supports, diagonal cracks were likely to occur near the supports.

3. When the structural system of the bridge included continuity over the supports, negative moment transverse cracks were likely to occur.

4. Observations from the Texas bridges indicated that bridge geometry influences the amount of concrete cracking particularly when the geometry results in torsional stresses.

5. Observations from the Ohio bridge showed that longitudinal cracks occur above the edges of the adjacent boxes in box beam bridges.

6. A w/cm ratio between 0.35 and 0.40 provided a lower average crack density for the bridge decks in this study.

7. Cementitious materials contents between 356 and 415 kg/m^3 (600 and 700 lb/yd^3) provided the lowest average crack density for the bridge decks in this study.

8. For the cast-in-place decks, the specified design compressive strengths ranged from 28 to 55 MPa (4,000 to 8,000 psi) at 28 days. The actual compressive strengths all exceeded the specified design compressive strengths.

9. For the cast-in-place decks, the specified permeability values ranged from 1,000 to 2,500 coulombs. The actual permeability values were generally less than the specified values and ranged from very low to moderate.

10. As evidenced by the Alabama Bridge, placement sequence can have an influence on bridge deck cracking.

11. These projects show that HPC decks can be produced with relatively few cracks.

CHAPTER 7. RECOMMENDATIONS

1. From the data obtained from this study, a high performance concrete mixture with a w/cm ratio between 0.35 and 0.40, cementitious material content between 356 and 415 kg/m^3 (600 and 700 lb/yd^3), and appropriate construction practices, is expected to have a lower crack density. The associated rapid chloride permeability is expected to be in the low to moderate range for these mixtures.

CHAPTER 8. RECOMMENDATIONS FOR FUTURE WORK

1. The petrographic analysis provided limited information. The analysis performed in this study could be enhanced by a detailed petrographic analysis containing the following:
    - Chloride content at three different depths,
    - Air void parameters,
    - Estimated water-to-cementitious materials ratio,
    - Estimated paste content,
    - Identification of constituent materials.

2. The structures included in this study are getting close to having 10-15 years of service life. Any new information that could be obtained regarding the current condition of these structures would be useful for future reference. Data could be collected through a second set of inspections.

3. The concrete material has performed well for this study. It has been shown that the material can be designed and fabricated to meet specifications. The individual reports for the bridge decks investigated in this study provide a database that can be made available for future design and optimization of high performance concrete bridges.

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APPENDIX A

Uphapee Creek Bridge, Alabama

HIGH PERFORMANCE CONCRETE BRIDGE DECK INVESTIGATION

The Uphapee Creek Bridge - Alabama SR-199
Macon County, Alabama

I. BACKGROUND

The Uphapee Creek Bridge on Alabama Highway 199 in Macon County, Alabama, is one of the first High Performance Concrete (HPC) bridges built in Alabama (see photos 1 through 3). It replaced a bridge built in the 1940's that had suffered from streambed scour resulting from sand and gravel mining downstream. The bridge carries heavily loaded gravel and loading trucks traffic. After the completion of the HPC bridge project, the Uphapee Creek Bridge opened to traffic in April 2000.

The Uphapee Creek Bridge has 7 spans on both northbound and southbound lanes. The overall length of the bridge is 798 ft. The clear width of the bridge is 40 ft, carrying four lanes of traffic with shoulders. The overall length of each span is 114 ft, and the length between the centerlines of the bearing is 112.25 ft. Plan view of the bridge is shown in Figure 1.

The Uphapee Creek Bridge has a deck thickness of 7 in. HPC was used on all girders and the cast-in-place deck in the Uphapee Creek Bridge. On the same project, within one mile the Uphapee Creek Relief Bridge was constructed utilizing HPC only for the cast-in-place concrete.

There are five AASHTO BT-54 girders per span spaced at 8.75 ft in the Uphapee Creek Bridge. Typical bridge girders designed by Alabama Department of Transportation (ALDOT) are based on 4000 psi release and 5000 psi at 28 days, with the prestressing force provided by 0.5 in. diameter 7-wire strand. The HPC girders utilize the 0.6 in. diameter strand, which allows a higher prestressing force to be applied. The #7 crushed limestone was allowed in the prestressed concrete girders for the first time in ALDOT projects. Compressive strength of the girder was specified as 8000 psi at release and 10,000 psi at 28 days. The use of HPC enabled the bridge to be designed with one less line of girders than would be required if regular concrete was used.

The specified compressive strength of the cast-in-place concrete was 6,000 psi. Design consideration for the concrete members was based on a compressive strength of 4000 psi. While the higher strength of the cast-in-place concrete was not fully utilized, HPC was specified to provide enhanced performance and durability. ALDOT conducted the Uphapee Creek Bridge project in cooperation with Auburn University.

II. SCOPE OF SERVICES

Professional Service Industries Inc. (PSI) is under contract with the Federal Highway Administration (FHWA) to conduct bridge deck inspections for HPC bridges. Our scope of services on this bridge included a series of tasks and sub-tasks, which are described as follows:

- Collect all available information relevant to the construction of each bridge, including;
    - Deck Concrete Properties
    - Specified Deck Concrete Construction Procedures
    - Approved Concrete Mix Proportions
    - Measured Properties from QC
    - Other Measured Properties
    - Actual Method of Deck Placement
    - Average Daily Traffic (ADT)
    - Exposure Condition of the Bridge
    - Any Performed Maintenance
    - Any Inspection Reports

- Visually inspect the bridge, and obtain the following:
    - General condition of the deck top surface
    - Determination of the maximum crack width
    - General condition of the deck underside
    - General condition of the girders
    - Photograph areas of significant deterioration
    - Prepare drawings locating defects
    - Extract 7 concrete core samples

III. COMPILATION OF BRIDGE CONSTRUCTION INFORMATION

Information sources used for this report include bridge drawings, inspection reports, bridge summary data sheets, and technical information contained in FHWA's "Compilation and Evaluation of Results from High Performance Concrete Bridge Projects" version 3.

Deck Concrete Properties

HPC was used on all girders and the cast-in-place deck in the Uphapee Creek Bridge. The bridge deck had a specified concrete compressive strength of 6000 psi at 28 days. Maximum water-to-cementitious materials ratio (w/cm) of 0.40 was specified for cast-in-place deck concrete. Table 1 lists the specified concrete properties used in the cast-in-place deck.

TABLE 1: Specified Concrete Properties

Property	Deck Class AA (HPC)
Max. Water/Cementitious Materials Ratio:	0.40
Min. Percentage of Class C Fly Ash:	20
Max. Percentage of Class C Fly Ash:	30
Min. Percentage of Class F Fly Ash:	15
Max. Percentage of Class F Fly Ash:	25
Min. Percentage of Silica Fume:	7
Max. Percentage of Silica Fume:	15
Slump:	≤125 mm for Superstructures ≤200 mm for Substructures
Air Content:	3.5~6.0%
Compressive Strength - Design:	6000 psi. @ 28 days
Maximum Temperature of Fresh Concrete:	95ºF

Specified Deck Concrete Construction Procedures

General requirements for the materials, concrete, sampling, and testing of HPC used in the Uphapee Creek Bridge were in a special provision to the ALDOT Standard Specifications. Wet curing for the bridge decks was specified. As part of the HPC project specifications, a slab (20-ft×20-ft×3½-in) test pour was required before any superstructure concrete could be placed. Three specimens at each age were required from three different deck pours made during the construction of the bridge.

The span lengths at the Uphapee Creek Bridge were sufficiently long that multiple pours were required to cast the deck slab for each span. First the center half of each span was cast. Then pours were repeated until all deck slabs were completed. To reduce the chance of plastic shrinkage cracking, ALDOT required the use of fogging spray to keep the evaporation at acceptable levels (0.1 lb/sq ft/hr).The deck should be immediately covered with water soaked burlap after initial set.

Approved Concrete Mix Proportions

Deck

Class AA HPC was used in the cast-in-place deck of the Uphapee Creek Bridge. The approved proportions for cast-in-place deck are shown in Table 2.

TABLE 2: Approved Mix Proportions for Uphapee Creek Bridge

Mix Parameters	Cast-in-Place Superstructure	Cast-in-Place Substructure
Cement Brand:	Blue Circle	Holman
Cement Type:	II	II
Cement Quantity:	658 lb/yd^3	640 lb/yd^3
Fly Ash Type:	C	--
Fly Ash Quantity:	165 lb/yd^3	160 lb/yd^3
Fine Aggregate Type:	Natural Sand	Natural Sand
Fine Aggregate Quantity:	1042 lb/yd^3	990 lb/yd^3
Coarse Aggregate, Max. Size:	1-in.	--
Coarse Aggregate Type:	Crushed Limestone	Crushed Limestone
Coarse Aggregate Quantity:	1860 lb/yd^3	1950 lb/yd^3
Water:	288 lb/yd^3	300 lb/yd^3
Water Reducer Brand:	MB Pozzolith 100-XR	--
Water Reducer Type:	B and D	--
Water Reducer Quantity:	25 fl oz $/yd^3$	25 fl oz $/yd^3$
High Range Water Reducer Brand:	Polyheed 977	--
High Range Water Reducer Type:	A and F	--
High Range Water Reducer Quantity:	98 fl oz $/yd^3$	96 fl oz $/yd^3$
Air Entrainment Brand:	MB AE90	--
Air Entrainment Type:	Anionic Surfactant	--
Air Entrainment Quantity	32 fl oz $/yd^3$	32 fl oz $/yd^3$
Water/Cementitious Materials Ratio:	0.37	0.38

Measured Properties from QC Tests of Production Concrete

Cast-in-Place Deck Panels

Measured properties of concrete mix for the cast-in-place deck are summarized in Table 3.

TABLE 3: Measured Properties of Approved Concrete Mixes for Cast-in-Place Deck and Precast Girders

Date Cast	Span	Location	Air Content, %	Slump, in.	Compressive Strength, psi	
					7 days	28 days
11/15/99	5	Center	4.2	5	6180	7500
			4.0	6	5690	7000
11/16/99	6	Center	4.9	5-½	6430	7840
			5.7	5-¾	5700	7010
11/18/99	7	Center	4.9	6-½	5000	6000
			3.8	6	5730	6700
11/19/99	5	North Quarter	4.5	7	6010	7100
11/22/99	6	South Quarter	4.2	5	5780	6840
11/23/99	6	North Quarter	5.0	5	5860	6970
11/24/99	7	North Quarter	4.5	5-¼	5620	7140
11/29/99	7	South Quarter	5.1	6-¾	5120	6510
11/30/99	5	South Quarter	4.9	5-½	5860	7360
12/2/99	4	Center	5.9	6	5720	6900
			4.8	5-¼	5470	7300
12/6/99	3	Center	4.0	5-½	5760	8280
			5.1	6	6560	7760
12/7/99	4	North Quarter	5.3	5-½	5640	7100
12/8/99	4	South Quarter	5.6	5-½	5820	7070
12/14/99	2	Center	4.6	5	6600	8000
			4.6	6	6640	7950
12/15/99	1	Center	4.6	5-½	6570	8180
			4.3	6	6830	7540
12/16/99	3	North Quarter	4.9	6	6600	8140
12/16/99	3	South Quarter	3.7	5-½	6670	8200
12/20/99	2	South Quarter	5.3	6	6320	7560
12/22/99	2	North Quarter	6.2	6	5920	6960
12/23/99	1	South Quarter	5.0	5-½	6240	7700
12/23/99	1	North Quarter	3.25	5-½	6060	7790
Average			4.7	5-¾	6010	7370

<u>Measured Properties from Research Tests of Production Concrete in Cast-in-Place Deck</u>

The slump, air content, compressive strength, modulus of elasticity, freeze-thaw durability, and shrinkage of HPC production concrete used in the Uphapee Creek Bridge deck are shown in Tables 4 through 7.

Slump and Air Content

TABLE 4: Measured Slump and Air Content of Production Concrete Used in the Cast-in-Place Deck

Span	Slump, in.	Air Content, %
Span 3	4	3.9
Span 4	4-½	4.7
Span 6	5-¼	3.9

Compressive Strength, modulus of elasticity, and chloride permeability

TABLE 5: Measured Compressive Strength, Modulus of Elasticity, and Chloride Permeability of Production Concrete Used in the Cast-in-Place Deck

Span	Age, days			
	7	28	56	91
Compressive Strength (1), psi				
4	5810	7440	8220	8630
6	5280	7220	7440	7870
3	5170	6450	6940	7370
Modulus of Elasticity (2), ksi				
4	4650	6500	6600	7300
6	4050	5750	5350	6600
3	4800	4950	5050	6050
Splitting Tensile Strength (3), psi				
4	440	530	520	490
6	410	530	490	560
3	350	470	490	430
Chloride Permeability (4), coulombs				
4	--	--	2835	1995
6	--	--	2765	1960
3	--	--	3020	2085

NOTES:

(1) Test follows AASHTO T 22. Result based on the average of three 6 × 12-in. cylinders stored on site for 24 hours and then placed in a moist room and tested with neoprene caps.

(2) Test follows ASTM C 469. Result based on one 6 × 12-in. cylinders stored on site for 24 hours and then placed in a moist room.

(3) Test follows AASHTO T 198. Result based on the average of two 6 × 12-in. cylinders stored on site for 24 hours and then placed in a moist room.

(4) Test follows AASHTO T 277. Result based on the average of two 2-in. thick slices cut from 4 × 8-in. cylinders stored on site for 24 hours and then placed in a moist room and tested with neoprene caps.

The modulus of elasticity illustrated a clear increase with age. At the age of 91 days, the average of three test results was 6,650 ksi. This value is used as the reported modulus of elasticity of the deck concrete.

Freeze-Thaw Durability

TABLE 6: Measured Freeze-Thaw Durability of Production Concrete Used in the Cast-in-Place Deck

Spans	Initial Frequency, Hz	Final Frequency, Hz	Durability Factor, %
3	2320	2230	92.4
	2320	2220	91.6
	2330	2220	90.8
Average			91.6
4	2340	2230	90.8
	2350	2170	85.3
	2350	2250	91.7
Average			89.3
6	2270	2170	91.4
	2270	2160	90.5
	2260	2200	94.8
Average			92.2

NOTES: Test follows AASHTO T 161 procedure A. 3 × 4 ×16-in. prisms stored in a lime water bath for 14 days prior to start of the test. Tests were conducted for 300 freeze thaw cycles with three samples per mix.

Free Shrinkage

TABLE 7: Shrinkage Measurements of Production Concrete Used in the Deck

Span	Curing Period, days	Shrinkage, millionths
3	7	250
	28	300
4	7	470
	28	280
6	7	480
	28	330

NOTES: Test follows ASTM C157 using 3 × 3 ×12-in. prisms. Zero length was measured when the specimens were stripped from the molds at 1 day and before immersing in lime water. Values are reported for a concrete age of 90 days.

In addition, following ASTM C 944, abrasion resistance of production concrete used in the Uphapee Creek Bridge was tested. The measured value was 0.00195 oz/in^2. For this test, 6x16x2-in. specimens were water cured for 7 days followed by curing in air until a concrete age of 56 days. Test used a 22 lb. force at 200 rpm for a two-minute abrasion period. The test result is mass loss per unit area.

Actual Method of Deck Placement

Construction of the deck occurred in the late fall of 1999, with the concrete for the deck delivered by truck and dumped to the deck surface using a one cubic yard bottom dump bucket. Formwork was prepared on site in advance with no reinforcement in the formed area. The bucket was lifted by crane to the location where the concrete was placed into the formwork. Concrete was moved manually with shovels and consolidated with a mechanical immersion vibrator. Following the consolidation of the deck concrete, the steel screed was used for initial leveling. A longitudinal screeding process was used on the bridge decks. Confilm® was applied in front of the screed that typically required three passes of the screed before hand finishing began. A final troweled finish was applied followed by tinning for enhanced skid resistance.

Fogging of the concrete deck started when the concrete was in the plastic state. The deck was cured using water soaked burlap covered with white plastic for seven days. The wet burlaps was kept moist. Curing of the slab involved brooming the slab areas finished by the bull float and fogging the entire slab with water vapor to help retard evaporation. Water soaked burlap was applied over the entire slab as a curing membrane after the slab had set for approximately 3-½ hours. The burlap was then covered with a plastic tarp, and soaker hoses were placed under the burlap to continuously wet cure the slab for seven days. This construction practice proved to be particularly important for HPC with low w/cm ratios.

Average Daily Traffic (ADT)

Average daily traffic for both eastbound and westbound lanes was calculated based on a count of all vehicles crossing the bridge during a 15 minutes period beginning at 1310 hrs on April 7, 2004. These vehicle counts gave an ADT of 2688. The estimation of traffic flow was made while the PSI inspection crew was on-site.

Exposure Condition of the Bridge

The Uphapee Creek Bridge on Alabama Highway 199 in Macon County, Alabama carries heavily loaded gravel and loading trucks traffic. The area surrounding the bridge is developed with mixed residential and farm use. The National Weather Service reports that the normal maximum temperature varies between 92°F in July and 57°F in January. The normal minimum temperature varies between 72°F in July and 37°F in January. The normal precipitation varies between 6.1 inches per month in March and 2.5 inch per month in October. Very few days per year does the temperature drop below 32°F. Based on this information, the bridge has minimal annual exposure to wet/dry and freeze/thaw cycles.

Performed Maintenance

No documents were found that would indicate any maintenance had been performed since the bridge was constructed in 1999.

Inspection Reports

As part of the project, bridge instrumentation and bridge monitoring are being performed by Auburn University Highway Research Center. The researchers have developed an instrumentation program to monitor the structural performance of the bridge and its components as described in "High Performance Bridge Concrete".

IV. BRIDGE DECK INSPECTION

PSI personnel performed a visual inspection of the bridge decks during the week of April 7, 2004. The results of the inspection are summarized as follows.

General Condition of the Deck Top Surface

Figure 1 shows the general layout of the decks for the Uphapee Creek Bridge. Results of visual inspection of the decks are shown in Figures 2a and 2b. Surface defects observed and documented during visual inspection primarily included transverse cracks, longitudinal cracks, and diagonal cracks (see photos 8 through 10). Other defects observed and documented included dog footprints on Span 1, which was believed to have occurred at an early age when concrete was placed and cured; small spalls at joints, and cracks and broken tinned edges (see photo 12). However, apparent signs of other serious damages such as freeze-thaw, D-cracking, alkali-silica reaction, and alkali-aggregate reaction were not observed. Both longitudinal and diagonal cracks observed on the top surface of cast-in-place deck were marginal. Efflorescence was observed on the concrete barrier wall along the bridge.

A total of 121 cracks (108 traverse cracks, 5 longitudinal cracks, and 8 diagonal cracks) were recorded during visual survey of the bridge decks (see Figure 2). The sum of crack lengths was 1732 ft over a bridge deck area of 31,920 ft^2. Crack density (total crack length / deck area) for the eastbound and westbound bridges combined was calculated to be 0.054 ft/ft^2.

It is noted that the number of transverse crack counts among all spans appear to be similar. The total crack length is the longest for Span 1 (316 ft) and the shortest for Span 6 (154 ft). All measured cracks are hairline crack with a width of less than 0.031 in. Typical transverse crack, longitudinal crack, and diagonal crack on the bridge decks are shown in photos 8, 9 and 10, respectively.

Cracks were typically limited at span ends. Small surface spalls, which either occurred due to breaking of tined edges or the crack edges, were observed. Figures 2a and 2b also illustrate the locations of drilled cores.

The number, length and density of cracks for entire bridge decks in both directions are shown in Tables 8 through 11, and described below according to the crack type.

Transverse Cracks: Figure 2 illustrates the transverse cracks that were identified on the surface of the bridge decks. Table 8 provides the detailed information regarding transverse cracks identified on the bridge decks. The crack densities (crack length per deck area) range from 0.0338 to 0.0690 ft/ft^2 for the 7 spans investigated.

TABLE 8: Measured Transverse Cracks on the Bridge Decks

Eastbound Traverse Cracks	Count	Length Range (feet)	Mean Length of Cracks (feet)	Median Length of Cracks (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span 1	19	6 to 40	16.3	11.5	316.0	4560	0.0690
Span 2	15	5 to 40	32.2	18	289.5	4560	0.0635
Span 3	16	4 to 40	14.3	10.5	228.0	4560	0.0500
Span 4	14	5 to 5.5	17.3	10.5	241.5	4560	0.0530
Span 5	20	4 to 40	11.8	8.3	240	4560	0.0526
Span 6	12	3 to 40	17.0	11.5	204.5	4560	0.0448
Span 7	12	4 to 40	12.8	8	154	4560	0.0338

NOTE: Transverse cracks include cracks oriented parallel to skewed joints.

Diagonal Cracks: The diagonal crack densities (crack length per deck area) range from 0 to 0.0007 ft./ft.^{2} for the 7 spans investigated. Diagonal cracks in the bridge decks were typically present near the joints.

TABLE 9: Measured Diagonal Cracks on the Bridge Decks

Eastbound Traverse Cracks	Count	Length Range (feet)	Mean Length of Cracks (feet)	Median Length of Cracks (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span 1	1	4.5	4.5	4.5	4.5	4560	0.0010
Span 2	1	3.0	3.0	3.0	3.0	4560	0.0007
Span 3	1	4	4	4	4	4560	0.0009
Span 4	2	2 to 4.5	3.3	3.3	6.5	4560	0.0014
Span 5	2	3 to 4	3.5	3.5	7	4560	0.0015
Span 6	1	5	5	5	5	4560	0.0011
Span 7	NA	NA	NA	NA	NA	4560	NA

Longitudinal Cracks: The number and length of longitudinal cracks were minimal. Several of the longitudinal cracks were along the beams and at the boundaries of the precast deck panels.

TABLE 10: Measured Longitudinal Cracks on the Bridge Decks

Eastbound Traverse Cracks	Count	Length Range (feet)	Mean Length of Cracks (feet)	Median Length of Cracks (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span 1	NA	NA	NA	NA	NA	4560	NA
Span 2	NA	NA	NA	NA	NA	4560	NA
Span 3	3	1 to 8	5.2	6.5	15.5	4560	0.0033
Span 4	NA	NA	NA	NA	NA	4560	NA
Span 5	2	6 to 7	6.5	6.5	13.0	4560	0.0029
Span 6	NA	NA	NA	NA	NA	4560	NA
Span 7	NA	NA	NA	NA	NA	4560	NA

Maximum Crack Width

The maximum width of transverse cracks was measured to be 0.016 in. According to ACI 201, these crack widths are classified as hairline cracks. The fine width cracks were generally located at span ends and some exhibited spalling due to the breaking of the edges.

General Condition of the Deck Underside

The underside of the deck exhibits no signs of distress. At very limited locations, efflorescence was observed. Photos 4 through 6 show a general view of the underside of the deck.

General Condition of the Girders

The girders were inspected from a motor boat, without the aide of any access equipment. No visible signs of distress were noted on any of the girders.

Concrete Core Samples

Seven cores 3-¾ inches in diameter were retrieved from the decks. The core sample locations are shown on Figure 2a and 2b. The locations were evenly distributed along each shoulder of the bridge. The cores were labeled AL-1 through AL-7 and were transferred to FHWA for further analysis.

TABLE 17: Core Dimensions

Sample	AL-1	AL-2	AL-3	AL-4	AL-5	AL-6	AL-7
Diameter (in.)	3¾	3¾	3¾	3¾	3¾	3¾	3¾
Length (in.)	2	3¼	3¼	3½	3½	2½	3 ½

Preliminary Conclusions

The construction of the Uphapee Creek Bridge was completed in 2000. Researchers from Auburn University performed the material testing, bridge instrumentation, and bridge monitoring throughout this project.

The visual inspection of the bridge decks as part of our study was performed about two years after the bridge opened to traffic. A total of 121 transverse, longitudinal, and diagonal cracks were recorded on the bridge with a combined total crack length of 1732 ft over a bridge deck area of 31,920 ft^2. All cracks on the bridge were hairline cracks with a width of less than 0.031 in. No major distresses were observed in our bridge survey.

With respect to cracking type, transverse cracking exhibited the greatest density with a total value of 0.3667 ft/ft^2. Furthermore, the transverse crack density was greatest in Span 1 and least in Span 7. It should be noted that the structural system of the Uphapee Creek Bridge is flexible compared to conventional bridges considering the wide beam spacing, large span, and relatively thin deck used. This relatively flexible structural system combined with the heavy ADT on the bridge might have contributed to the development and widening of some cracks.

In general, the work on the Uphapee Creek Bridge shows that HPC designs provide significantly higher strength that can lead to more efficient designs and improved durability.

Petrographic examination was performed on seven concrete cores samples that were retrieved from the decks of the northbound and southbound bridges of the Uphapee Creek Bridge. The seven concrete cores ranged from 2- to 3-in. long, with a 3-3/4-in. diameter. The identification on the cores was as follows: AL-1, AL-2, AL-3, AL-4, AL-5, AL-6 and AL-7.

Visual inspection of the concrete cores revealed that three cores (AL-2, AL-5, and AL-6) have cracked along the length, and one core (AL-3) was split longitudinally. It was speculated that shrinkage may be the cause of the cracking. There was no evidence of any material related deterioration in the concrete. Core AL-5 showed that the concrete at the rebar level (about 1-1/8 in. below the surface) and below was poorly consolidated, leaving honeycombing in the concrete. No defects were observed visually in the other cores.

Rebar and rebar impressions were found in 5 cores. The distance from the top of the rebar to the exposed concrete surface varies in the cores, as illustrated in Table 18.

TABLE 18: Rebar Clear Cover Depth

Core	Clear Cover Depth (in.)
AL-1	1-¾
AL-2	2-¾
AL-3	2-¾
AL-5	2-1/8
AL-6	2-3/8

The coarse aggregate in the concrete was crushed stone of a carbonate rock. Coarse aggregate particles were mostly angular, and the maximum size, measured from the prepared concrete samples, was about 1 inch. Preferential orientation of coarse aggregate particles was not observed. The natural sand fine aggregate was mainly composed of quartz. The fine aggregate particles appeared rounded to angular.

The cement was reasonably hydrated with respect to the age of the concrete. The cement paste contained some unhydrated cement particles. Fly ash particles were also present in the cement paste matrix.

The concrete was air entrained, and small, spherical air voids were observed in the concrete. Entrained air voids were well distributed in the concrete. Entrapped air voids were also present in the concrete.

The cement/aggregate interface was dense and fairly strong.

Ettringite was observed in some air voids in the concrete. There was no evidence of deterioration associated with the existence of the ettringite in the concrete. It is very common to see ettringite as secondary deposit in concrete.

Respectfully Submitted,
Professional Service Industries, Inc.
Structural Investigation and Petrography Department

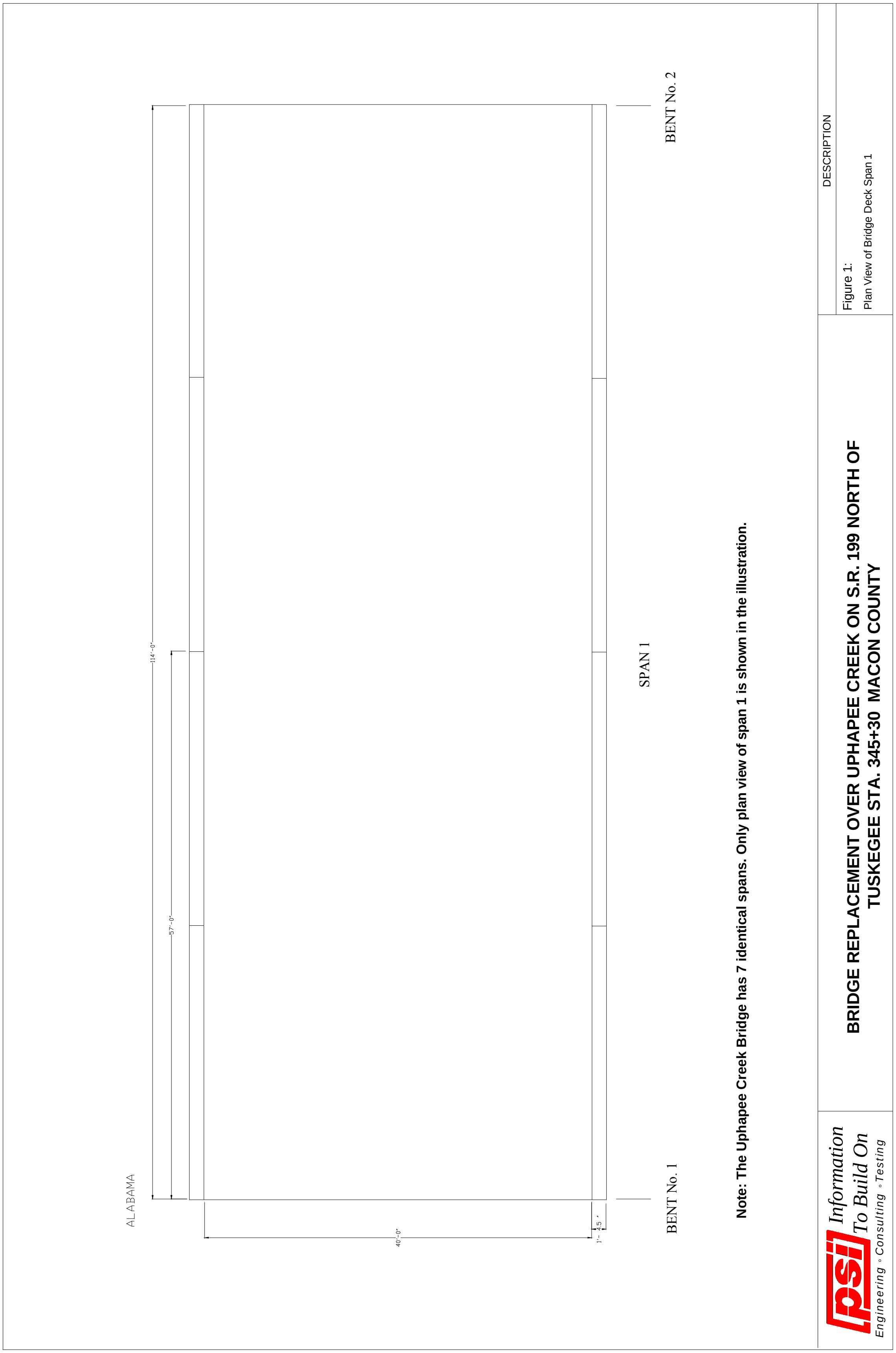

ALABAMA
114'-0"
57'-0"
40'-0"
BENT No. 1
SPAN 1
BENT No. 2
Note: The Uphapee Creek Bridge has 7 identical spans. Only plan view of span 1 is shown in the illustration.
psi
Information
To Build On
Engineering • Consulting • Testing
BRIDGE REPLACEMENT OVER UPHAPEE CREEK ON S.R. 199 NORTH OF
TUSKEGEE STA. 345+30 MACON COUNTY
DESCRIPTION
Figure 1:
Plan View of Bridge Deck Span 1

Figure 2a:
Plan View (Span No.1 through No.4) Core and Crack Locations

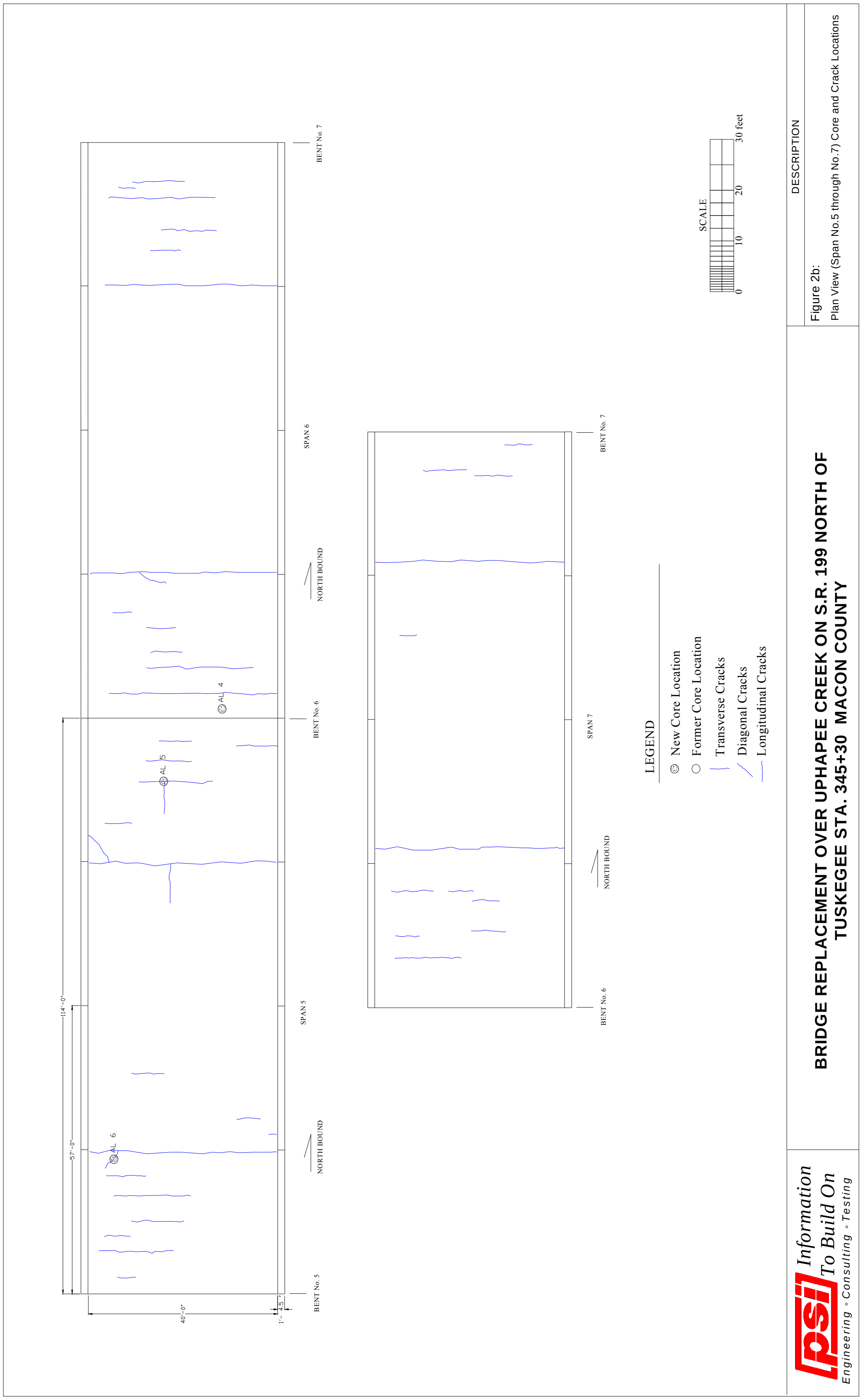


Figure 2b:
Plan View (Span No.5 through No.7) Core and Crack Locations

Photographic Documentation

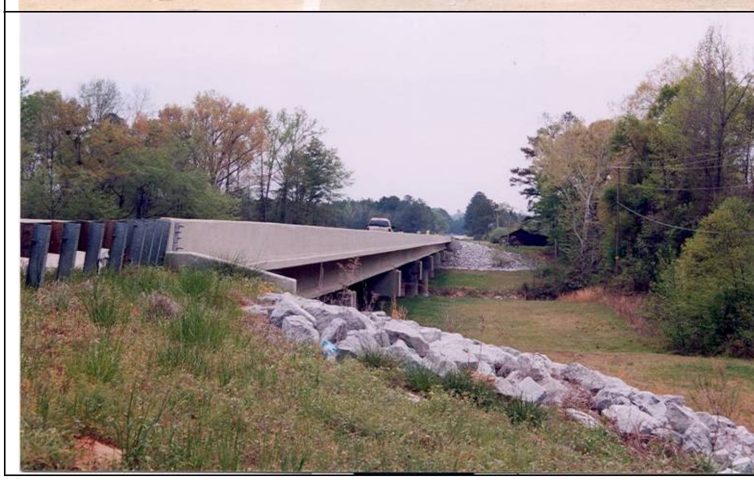
	Photo 1: View looking south from the northern end of west facing bridge.
	Photo 2: View looking north from the southern end of west facing bridge.
	Photo 3: View looking south from the northern end of east facing bridge.

	Photo 4: View of the underside of bridge.
	Photo 5: View of the girders from underside of bridge.
	Photo 6: General condition of the piles at Span 2 east facing bridge. Accumulation of debris observed near the pile.

	Photo 7: Close up view of bridge deck underside inspection using a snooper.
	Photo 8: Close up view of typical transverse crack at deck joints.
	Photo 9: Close up view of typical longitudinal crack.

Photo 10: Close up view of typical diagonal crack.

Photo 11: Unsealed gaps in between adjacent spans.

Photo 12: Close up view of the dog footprints during the early-age curing.

	Photo 13: View of the drilled cores on bridge deck.

APPENDIX A – Supplement 1

Uphapee Creek Bridge, Alabama
Petrographic Report

PETROGRAPHIC EXAMINATION OF CONCRETE CORES FROM AN ALABAMA BRIDGE (AL)

A report submitted to the Structures Laboratory at TFHRC

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Petrographic Laboratory
FHWA TFHRC
(Reviewed by Richard Meininger, PE; Concrete Laboratory; printed in color 9-6-2006)

May 12, 2006

1. Abstract

Seven concrete cores were received from the Structures Laboratory of the Turner-Fairbank Highway Research Center for petrographic examination. Reportedly, the concrete cores were collected from a concrete bridge in Alabama.

Petrographic examination was performed on samples using optical microscopes. Thin sections, as well as polished slabs, were made from the concrete cores for microscopic examination.

Visual inspection of the concrete cores revealed that three cores (AL-2, AL-5, and AL-6) have cracked along the length, and one core (AL-3) was split longitudinally. Core AL-5 shows that the concrete at the rebar level (about 1-1/8 inch below the surface) and below was poorly consolidated, leaving honeycombing in the concrete. No defects were observed in the other cores. The findings from microscopic examination indicate that the concrete has entrained air voids. The hydration of the cement was reasonable. The presence of some unhydrated cement particles was also observed in the cement paste. Ettringite as secondary deposit formed in air voids.

2. Introduction

The Petrographic Laboratory of the FHWA Turner-Fairbank Highway Research Center was asked by the Structures Laboratory to examine a set of concrete cores retrieved from a bridge in Alabama. Seven concrete cores of 3-3/4-in. diameter, 2- to 3-in. long were received by the Petrographic Laboratory. The identification on the cores was as follows: AL-1, AL-2, AL-3, AL-4, AL-5, AL-6 and AL-7

3. Laboratory Procedures

Petrographic examination of the concrete samples was performed in accordance with ASTM C 856, “Standard Practice for Petrographic Examination of Hardened Concrete.”

Sections were polished and examined using a stereomicroscope at magnifications up to 350×. Small rectangular blocks were cut from concrete samples. One surface of each block was polished, dried, placed on petrographic slide with low−viscosity epoxy resin, and reduced to a thickness of approximately 25 micrometers (0.001 in.). These thin sections were examined using a polarized−light microscope at magnification up to 400×, to determine aggregate mineralogy, paste characteristics, and microstructure.

Two ¾ inch thick slabs were sawn from the concrete cores. The slabs were ground and polished to obtain a smooth and plane surface. They were examined using a stereomicroscope at magnifications up to 200×.

4. Findings

Eight (8) thin section samples taken from the received cores were examined using the polarized light microscope. Two polished sections were examined using the stereomicroscope. The findings are summarized as following:

Aggregates

The coarse aggregate in the concrete is crushed marble. Coarse aggregate particles are mostly angular, and the maximum size is about 1 inch. Preferential orientation of coarse aggregate particles is not observed in this concrete.

The fine aggregate fraction is mainly composed of quartz. The fine aggregate is from natural sand and the particles appear rounded to angular.

Cement Paste

The cement is reasonably hydrated and the paste contains some unhydrated cement particles as seen under the microscope (Figure A1-1). Fly ash particles are present in the cement matrix (Figure A1-2).

Air Voids

Small, spherical air voids are observed in the concrete (Figure A1-3), hence the concrete was air entrained. Entrained air voids are well distributed in the concrete. Entrapped air voids, and occasionally water voids, are also present in the concrete.

Cement-Aggregate Bonding

The cement/aggregate interface is dense and fairly strong, as shown in Figure A1-4 and Figure A1-5.

Secondary Deposit

Ettringite is observed in some air voids in the concrete. Very often, ettringite crystals filled up a portion of a void, as shown in Figure A1-6 and Figure A1-7. Occasionally, voids fully filled with ettringite are also found in the concrete (Figure A1-8).

Honeycombing

Honeycombing, as shown in Figure A1-9, is found in core AL-5. Core AL-5 shows that the concrete at the rebar level (about 1-1/8 in. below the surface) and below was poorly consolidated, leaving honeycombing in the concrete.

Cracking

The cause of the cracking in three of the cores is uncertain. It is speculated that shrinkage may be the cause of the cracking. There is no evidence of any material related deterioration in the concrete.

Rebar Clear Cover Depth:

Rebar and rebar impressions were found in 5 cores. The distance from the top of the rebar to the exposed concrete surface varies in the cores, as shown in Table A1-1:

TABLE A1-1. Rebar Clear Cover Depth

Core	Clear Cover Depth (in.)
AL-1	1-¾
AL-2	2-¾
AL-3	2-¾
AL-5	2-1/8
AL-6	2-3/8

5. Summary

The concrete was air entrained, and the entrained air voids were well distributed in the concrete. The bond between the aggregate and the paste appears fairly strong.

The cause of the cracking in three of the cores is uncertain. It is speculated that shrinkage may be the cause of the cracking. There is no evidence of any material related deterioration in the concrete.

The presence of honeycombing in one of the cores suggests poor consolidation in some sections of the concrete.

Ettringite crystals formed in air voids. Often, ettringite filled part of a void. But voids fully filled with ettringite are also found in the concrete. There was no evidence of deterioration associated with the existence of the ettringite in the concrete. It is very common to see ettringite as secondary deposit in concrete.

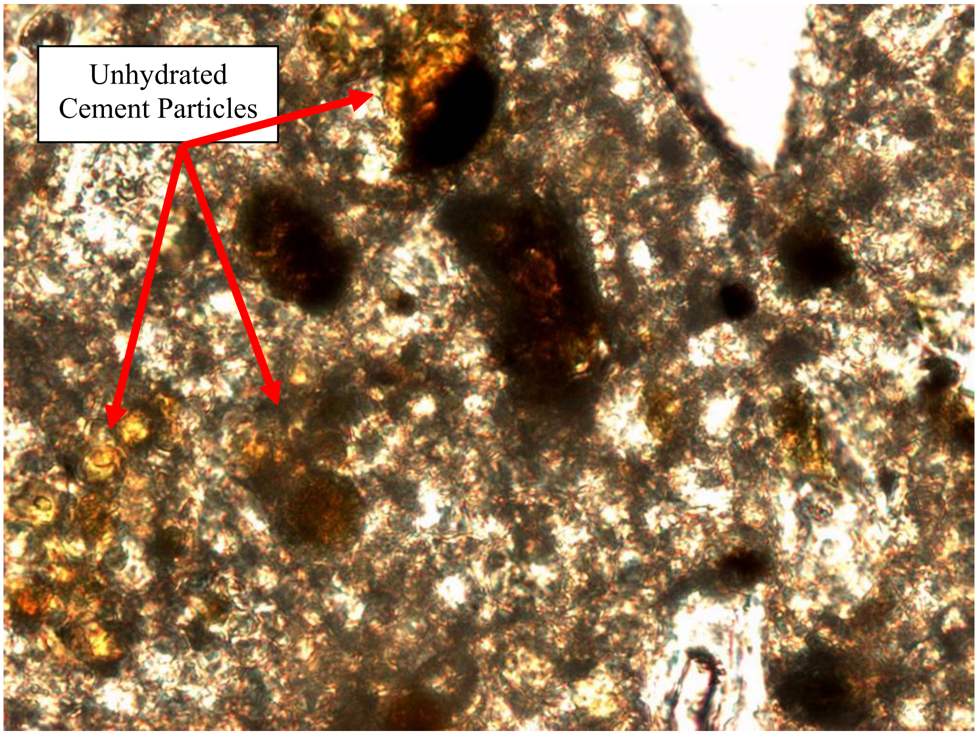


Figure A1-1: Unhydrated cement particles in paste. Width of field is 0.165 mm. Thin section image.

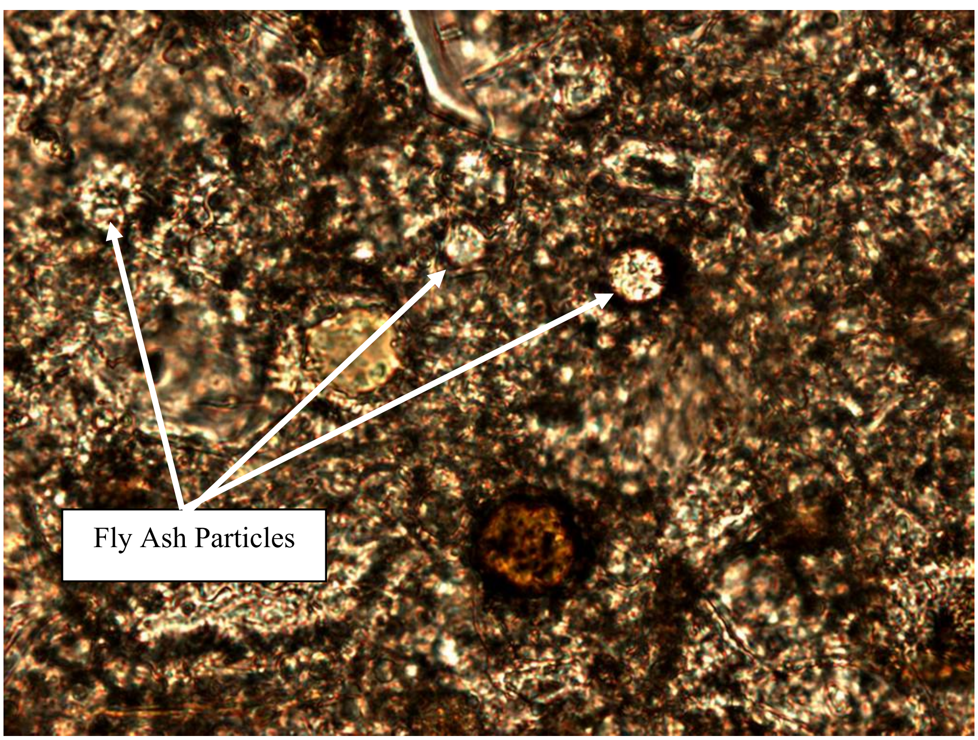


Figure A1-2: Fly ash particles in the cement matrix. Width of field is 0.165 mm. Thin section image.

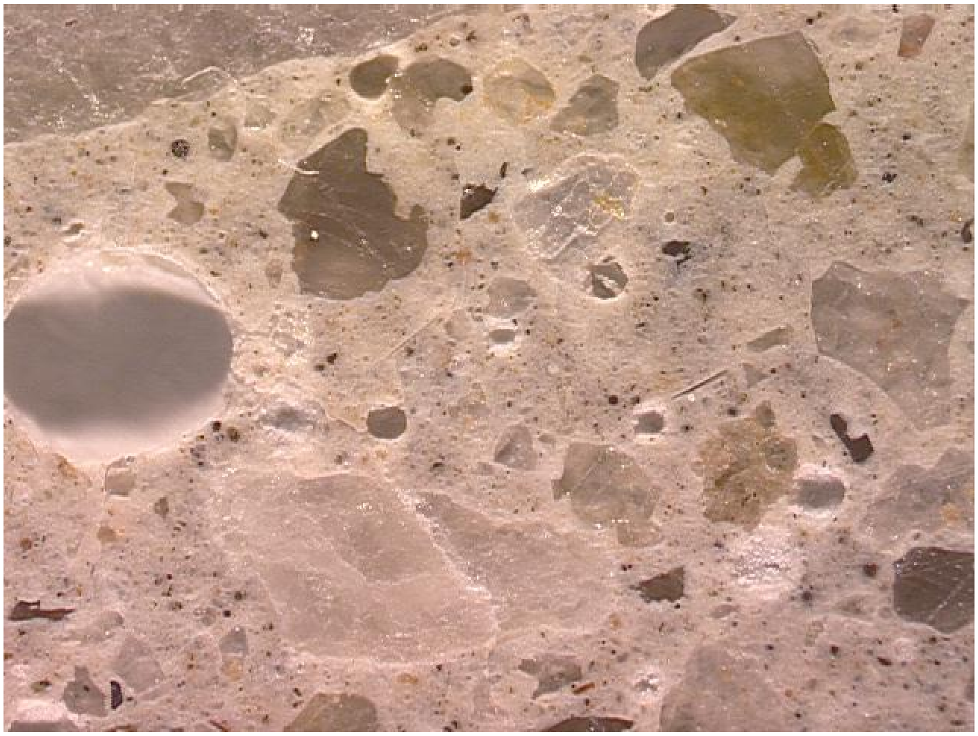


Figure A1-3: Entrained air voids in the concrete. Width of field is 4.0 mm. Polished surface image.

Figure A1-4: The bonding between aggregate and cement paste is strong. Width of field is 4.0 mm. Polished surface image.

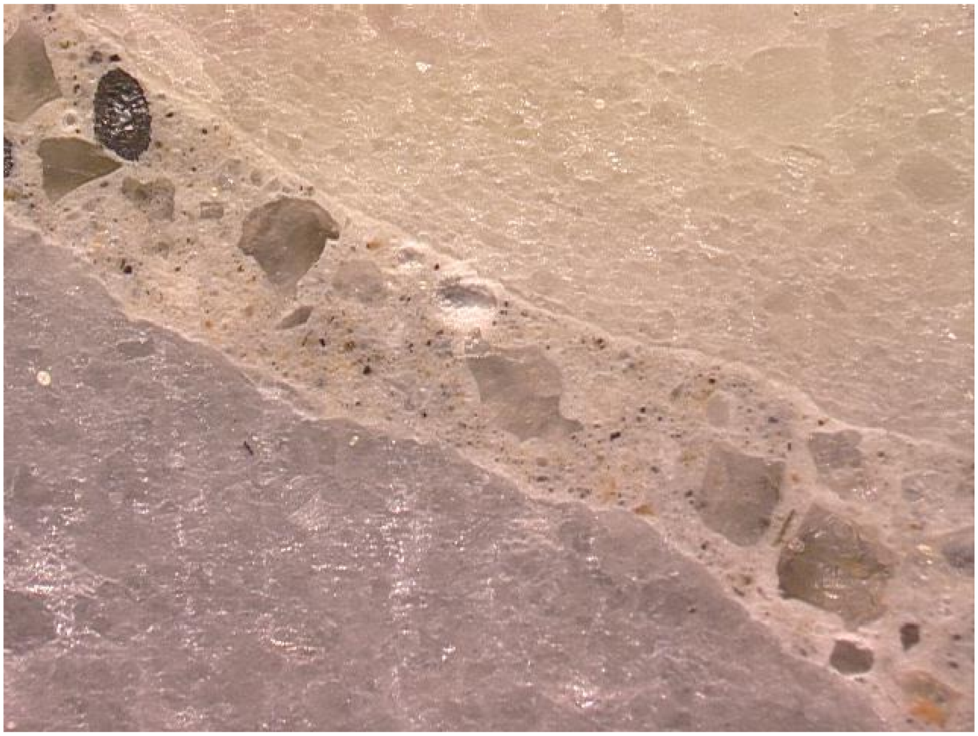


Figure A1-5: Another image showing the aggregate-paste interface is dense and strong. Width of field is 4.0 mm. Polished surface image.

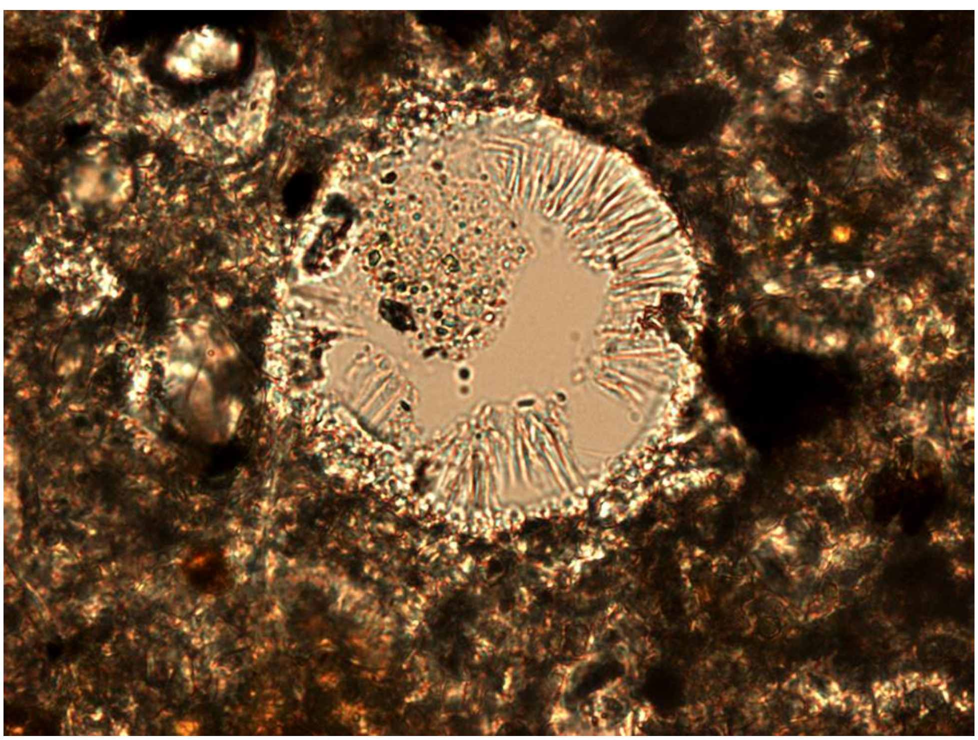


Figure A1-6: Ettringite in an air void. Width of field is 0.165 mm. Thin section image.

Figure A1-7: Another image of ettringite in an air void. Width of field is 0.33 mm. Thin section image.

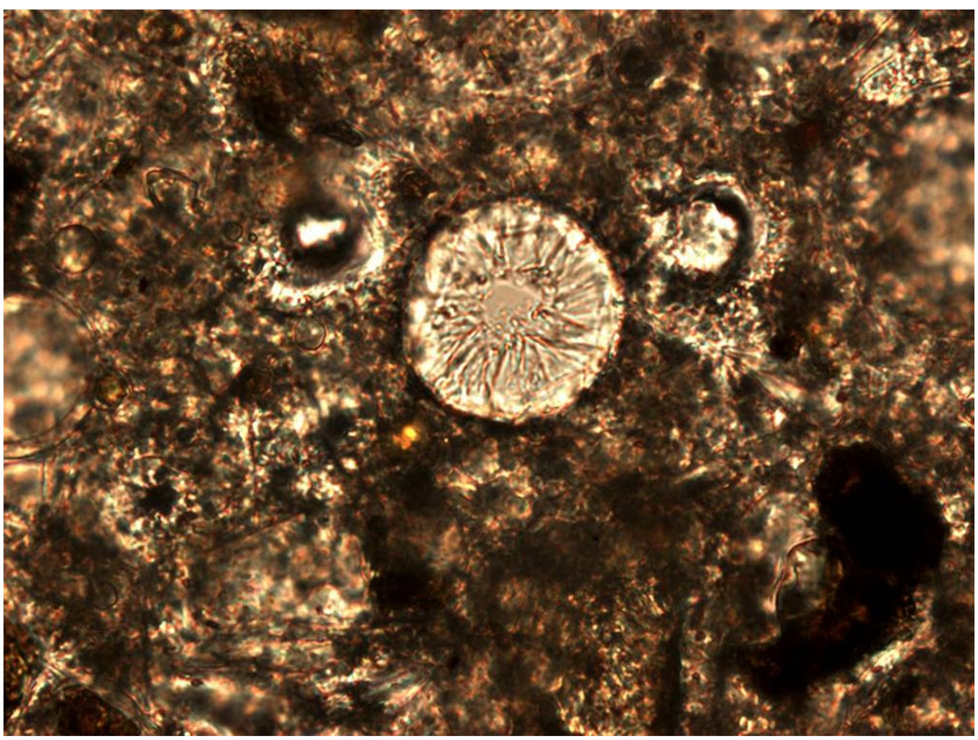


Figure A1-8: An air void fully filled with ettringite. Width of field is 0.165 mm. Thin section image.

Figure A1-9: Honeycombing in core AL-5.

APPENDIX A – Supplement 2

Uphapee Creek Bridge, Alabama
Survey Checklist

Checklist

The following checklist is adapted from 201.1R-92, and provided to facilitate a thorough survey. The definition of terms and associated photographs in 201.1 R-92, are utilized to standardize the reporting of the condition of the concrete in the structures.

1. Description of structure or pavement
   - 1.1 Name, location, type, and size:
     The Uphapee Creek Bridge on Alabama Highway 199 in Macon County, Alabama is 798 ft long. The clear width of the bridge is 40-ft (12.2 m), carrying four lanes of traffic with shoulders.
   - 1.2 Owner, project engineer, contractor, when built:
     Owner: North Carolina Department of Transportation. This bridge is part of a demonstration project for HPC in bridge structures which were co-sponsored by the Federal Highway Administration (FHWA) and the North Carolina Department of Transportation (NCDOT). The bridge was opened to traffic in April 2000.
   - 1.3 Design
     - 1.3.1 Architect and/or engineer: North Carolina Department of Transportation (NCDOT)
     - 1.3.2 Intended use and history of use: To replace a bridge built in 1940's that had suffered from streambed scour resulting from sand and gravel mining downstream. It carries heavily loaded gravel and loading trucks traffic on Alabama Highway 199.
     - 1.3.3 Special features: Bridge consists of seven spans (798-ft in total). AASHTO BT-54 girders were used. Compressive strength of the girder was specified as 8000 psi at release and 10,000 psi at 28 days.
   - 1.4 Construction
     - 1.4.1 Contractor-general: Clark Construction Company
     - 1.4.2 Subcontractors concrete placement: Sherman Prestressed Concrete
     - 1.4.3 Concrete supplier: Blue circle Williams
     - 1.4.4 Agency responsible for testing: North Carolina State University
     - 1.4.5 Other subcontractors: N/A
   - 1.5 Photographs
     - 1.5.1 General view — Photos 1 through 3
     - 1.5.2 Detailed close up of condition of area — Photos 4 through 12
   - 1.6 Sketch map-orientation showing sunny and shady and well and poorly drained regions: N/A
2. Present condition of structure — Date of Evaluation: The week of April 7, 2004
   - 2.1 Overall alignment of structure — No signs of misalignment
     - 2.1.1 Settlement
     - 2.1.2 Deflection
     - 2.1.3 Expansion
     - 2.1.4 Contraction

2.2 Portions showing distress (beams, columns, pavement, walls, etc., subjected to strains and pressures): Efflorescence at the underside of the bridge

2.3 Surface condition of concrete

2.3.1 General (good, satisfactory, poor, dusting, chalking, blisters) Good

2.3.2 Cracks Transverse and longitudinal

2.3.2.1 Location and frequency See Figure 2a and Figure 2b

2.3.2.2 Type and size (see Definitions) See Figure 2a and Figure 2b

Transverse At the beam diaphragm and panel boundaries

Width (from Crack comparator): Majority less than 0.03 in.

Hairline	(Less than 1/32 in.)
Fine	(1/32 in. - 1/16 in.)
Medium	(1/16 – 1/8 in.)
Wide	(Greater than 1/8 in.)

Craze N/A

Width (from Crack comparator)

Hairline	(Less than 1/32 in.)
Fine	(1/32 in. - 1/16 in.)
Medium	(1/16 – 1/8 in.)
Wide	(Greater than 1/8 in.)

Map N/A

Width (from Crack comparator)

Hairline	(Less than 1/32 in.)
Fine	(1/32 in. - 1/16 in.)
Medium	(1/16 – 1/8 in.)
Wide	(Greater than 1/8 in.)

D-Cracking N/A

Width (from Crack comparator)

Hairline	(Less than 1/32 in.)
Fine	(1/32 in. - 1/16 in.)
Medium	(1/16 – 1/8 in.)
Wide	(Greater than 1/8 in.)

Diagonal N/A

Width (from Crack comparator) N/A

Hairline	(Less than 1/32 in.)
Fine	(1/32 in. - 1/16 in.)
Medium	(1/16 – 1/8 in.)
Wide	(Greater than 1/8 in.)

2.3.2.3 Leaching, stalactites N/A

2.3.3 Scaling N/A

2.3.3.1 Area, depth

2.3.3.2 Type (see Definitions)

Light (Less than 1/8 in.)
Medium (1/8 in. – 3/8 in.)
Severe (3/8 in. – 3/4 in.)
Very Severe (Greater than 3/4 in.)

2.3.4 Spalls and popouts: Not significant. Along cracks may have spalls

2.3.4.1 Number, size, and depth N/A
2.3.4.2 Type (see Definitions) N/A
Spalls
Small (Less than 3/4 in. depth)
Large (Greater than 3/4 in. depth)
Popouts
Small (Less than 3/8 in. diameter)
Medium (3/8 in. – 2 in. diameter)
Large (Greater than 2 in. diameter)

2.3.5 Extent of corrosion or chemical attack, abrasion, impact, cavitation N/A

2.3.6 Stains, efflorescence Efflorescence at a few locations on the underside of the bridge

2.3.7 Exposed reinforcement: None

2.3.8 Curling and warping N/A

2.3.9 Previous patching or other repair N/A

2.3.10 Surface coatings N/A

2.3.10.1 Type and thickness N/A
2.3.10.2 Bond to concrete N/A
2.3.10.3 Condition N/A

2.3.11 Abrasion N/A

2.3.12 Penetrating sealers

2.3.12.1 Type N/A
2.3.12.2 Effectiveness N/A
2.3.12.3 Discoloration N/A

2.4 Interior condition of concrete (in situ and samples) N/A

2.4.1 Strength of cores
2.4.2 Density of cores
2.4.3 Moisture content
2.4.4 Evidence of alkali-aggregate or other reactions N/A
2.4.5 Bond to aggregate, reinforcing steel, joints N/A
2.4.6 Pulse velocity
2.4.7 Volume change
2.4.8 Air content and distribution
2.4.9 Chloride-ion content
2.4.10 Cover over reinforcing steel
2.4.11 Half-cell potential to reinforcing steel.
2.4.12 Evidence of reinforcement corrosion
2.4.13 Evidence of corrosion of dissimilar metals
2.4.14 Delaminations N/A

2.4.14.1 Location N/A
2.4.14.2 Number, and size N/A
2.4.15 Depth of carbonation
2.4.16 Freezing and thawing distress (frost damage)
2.4.17 Extent of deterioration
2.4.18 Aggregate proportioning, and distribution

3. Nature of loading and detrimental elements
3.1 Exposure
3.1.1 Environment (arid, subtropical, marine, freshwater, industrial, etc.) N/A
3.1.2 Weather-(July and January mean temperatures, 92ºF and 57ºF
mean annual rainfall and 4.4
months in which 60 percent of it occurs) March
3.1.3 Freezing and thawing negligible
3.1.4 Wetting and drying Minimal annual exposure
3.1.5 Drying under dry atmosphere N/A
3.1.6 Chemical attack-sulfates, acids, chloride N/A
3.1.7 Abrasion, erosion, cavitation, impact N/A
3.1.8 Electric currents N/A
3.1.9 Deicing chemicals which contain chloride ions N/A
3.1.10 Heat from adjacent sources N/A
3.2 Drainage N/A
3.2.1 Flashing
3.2.2 Weepholes
3.2.3 Contour
3.2.4 Elevation of drains
3.3 Loading Research Test Data Available in Compilation CD Version 3
3.3.1 Dead
3.3.2 Live
3.3.3 Impact
3.3.4 Vibration
3.3.5 Traffic index
3.3.6 Other
3.4 Soils (foundation conditions)
3.4.1 Compressibility
3.4.2 Expansive soil
3.4.3 Settlement
3.4.4 Resistivity
3.4.5 Evidence of pumping
3.4.6 Water table (level and fluctuations)

4. Original condition of structure Good
4.1 Condition of formed and finished surfaces Good
4.1.1 Smoothness
4.1.2 Air pockets ("bugholes")

4.1.3 Sand streaks
4.1.4 Honeycomb
4.1.5 Soft areas (retarded hydration)
4.1.6 Cold joints
4.1.7 Staining
4.1.8 Sand pockets

4.2 Defects N/A

4.2.1 Cracking
4.2.1.1 Plastic shrinkage
4.2.1.2 Thermal shrinkage
4.2.1.3 Drying shrinkage
4.2.2 Curling

5. Materials of Construction See Table 2

6. Construction Practices See Report pg. 3 and 8

APPENDIX B

I-25 over East Yale Avenue, Denver, Colorado

HIGH PERFORMANCE CONCRETE BRIDGE DECK INVESTIGATION

I-25 over East Yale Avenue
Denver, Colorado

I. BACKGROUND

The I-25 Bridge over East Yale Avenue in Denver, Colorado is a two-span bridge that carries Interstate 25 over Yale Avenue. HPC was used in the construction of box beams, bridge deck, and substructure. The new two-span HPC Bridge replaced a four-span bridge. The total length of the bridge is 65.5 m (215 ft) and the two spans are 34.5-m (112-ft) and 30-m (97-ft-7-in.) long, respectively. The 42-m (138-ft) wide bridge was built in phases to permit traffic flow in both directions during construction. The bridge has a 175-mm (7-in.) thick cast-in-place concrete deck. HPC was used in the construction of the precast prestressed side-by-side box girders that were used in the new bridge. The HPC, with specified compressive strength of 69 MPa (10,000 psi), enabled the superstructure to attain a high span-to-depth ratio. This allowed longer spans while maintaining a shallow superstructure depth. Three prestressed box girders were tested to evaluate the transfer and development length. The test results indicated that the AASHTO specifications on transfer and development length were conservative for the box girders.

The previous structure was a four-span, cast-in-place T-girder bridge with piers located in the median of Yale Avenue and at each side of the roadway. The new HPC bridge consisted of two spans in place of the original four spans. The new bridge improved clearances over Yale Avenue without a significant change in the grade of I-25.

The prestressed concrete box girders are 1700-mm (67-in.) wide and 750-mm (30-in.) deep. Prestressing strand, 15.2 mm (0.6 in.) in diameter with 51-mm (2-in.) center-to-center spacing, was used in the girders. The University of Colorado performed testing on strand pull-out strength, transfer length, and development length. The results are documented in Report No. CDOT-DTDR-98-7, Colorado Study on Transfer and Development Length of Prestressing Strand in High-Performance Concrete Box Girders.

The completed bridge was instrumented to measure temperature and strain variations. This was combined with deformation measurements to determine how the bridge behaves in response to creep, shrinkage, temperature changes, dead load, and live load. The Colorado Department of Transportation (CDOT) conducted the project in cooperation with the University of Colorado at Boulder. The first girder camber measurement occurred at prestress transfer and then at each stage of girder loading until the bridge construction was complete.

The replacement bridge for Interstate 25 over Yale Avenue in Denver, Colorado, is an excellent example of using high performance concrete (HPC) to meet the demands of urban bridge replacement. Construction on this project began in November 1996 and was completed in June 1998.

II. SCOPE OF SERVICES

Professional Service Industries Inc. (PSI) is under contract with the Federal Highway Administration (FHWA) to conduct bridge deck inspections for HPC bridges. Our scope of services on this bridge included a series of tasks and sub-tasks, which are described as follows:

- Collect all available information relevant to the construction of each bridge, including:
  - Deck Concrete Properties
  - Specified Deck Concrete Construction Procedures
  - Approved Concrete Mixture Proportions
  - Measured Properties from QC Tests of Production Concrete
  - Measured Properties from Research Tests of Production Concrete
  - Actual Method of Deck Placement
  - Average Daily Traffic (ADT)
  - Average Daily Truck Traffic (ADTT)
  - Exposure Condition of the Bridge
  - Any Performed Maintenance
  - Any Inspection Reports
- Visually inspect the bridge, and obtain the following:
  - General condition of the deck top surface
  - Determination of the maximum crack width
  - General condition of the deck underside
  - General condition of the girders
  - Photograph areas of significant deterioration
  - Prepare drawings locating defects
  - Extract concrete core samples

III. COMPILATION OF BRIDGE CONSTRUCTION INFORMATION

Information sources used for this report included bridge drawings, field inspection results, bridge summary data sheets, and technical information contained in FHWA's "Compilation and Evaluation of Results from High Performance Concrete Bridge Projects" version 3.

Deck Concrete Properties

At the time of the project, the CDOT specified that the deck concrete should achieve 28-day strength of 31 MPa (4500 psi), with approval based on 28-day strength of 35 MPa (5080 psi). The deck concrete for this bridge required 28-day strength of 35 MPa (5080 psi) for mix approval. No fly ash or silica fume was added to the concrete mix. The concrete mixture had a specified maximum w/cm of 0.44. The air content was 5-8%. The slump was not to be more than 1.5 in. greater than slump of the approved mix design. Table 1 lists the specified properties for the concrete used in the bridge deck.

TABLE 1: Specified Concrete Properties for Bridge Girders and Decks

Property	Girders	Decks
Min. Cementitious Materials Content:	660 lb/yd^3	660 lb/yd^3
Max. Water/Cementitious Material Ratio:	---	0.44
Max. Aggregate Size:	¾ in.	¾ in.
Max. Percentage of Fly Ash:	20%	10%
Slump:	---	---
Air Content:	---	5 - 8%
Designed Compressive Strength Release of Strands: Design:	 6500 psi 10,000 psi @ 56 days	 5076 psi @ 28 days
Chloride Permeability (AASHTO T 277):	---	

Specified Deck Concrete Construction Procedures

The procedures specified for deck concrete construction were in conformance with the current CDOT specifications for construction of bridge decks. The wind and low humidity in Colorado are problems and can contribute to deck cracking. A membrane-forming curing compound was placed immediately upon finishing and a moist cure was started when the deck concrete could be walked on without damage.

Approved Concrete Mixture Proportions for Production Concrete

The I-25 Bridge over East Yale Avenue has bridge deck consisting of 113-mm (4-1/2-in.) thick precast beam top flange and 175-mm (7-in.) thick cast-in-place concrete. Table 2 provides the approved mixture proportions for the HPC used in the bridge girders and decks.

TABLE 2: Approved Mixture Proportions for Precast Girders and Cast-in-Place Decks

	Precast Girders	**Cast-in-Place Decks**
Cement Brand:	Southwestern	Dacotah
Cement Type:	III	
Cement Quantity:	730 lb/yd^3	418 kg/m^3
Fine Aggregate Quantity:	1363 lb/yd^3	524 kg/m^3
Intermediate Aggregate Quality:	---	297 kg/m^3
Coarse Aggregate, Max. Size:	3/8 in.	¾ in.
Coarse Aggregate Type:	---	No. 67
Coarse Aggregate Quantity:	1775 lb/yd^3	883 kg/m^3
Water:	219 lb/yd^3	158 kg/m^3
Water-Reducer Brand:	WRDA 64	---
Water-Reducer Type:	A and D	---
Water-Reducer Quantity:	15-58 fl oz/ yd^3	---
High-Range Water-Reducer Brand:	WRDA 19	Master Builders
High-Range Water-Reducer Type:	A and F	---
High-Range Water-Reducer Quantity:	44-131 fl oz/yd^3	730 ml/m^3
Air Entrainment Brand:	---	Master Builders
Air Entrainment Quantity:	---	133 fl oz/yd^3
Water/Cementitious Materials Ratio:	0.29	0.38

Measured properties of the approved concrete mixtures for precast girders and cast-in-place deck are summarized in Table 3.

TABLE 3: Measured Properties of Approved Mixture for Precast Girders and Cast-in-Place Deck

Property	Precast Girders	Cast-in-Place Deck
Slump	4 – 9 in.	76 mm
Air Content, %	0-1.6%	6.6%
Unit Weight	150 – 152 lb/ft^3	2244 kg/m^3
Initial Set	4 hours	---

The measured properties of approved mix used in precast girders and cast-in-place deck are shown in Table 4.

TABLE 4: Properties of Approved Concrete Mixture used in the Construction of Precast Girders and Cast-in-Place Deck

Property	Precast Girders	Cast-in-Place Deck
Compressive Strength:	7500 psi at release 8000 psi at 7 days 9600 psi at 28 days 9900 psi at 56 days 10,100 psi at 90 days	46.9 MPa at 28 days
Flexural Strength:	765 psi at release 825 psi at 7 days 813 psi at 14 days 1150 psi at 28 days 1156 psi at 56 days	
Splitting Tensile Strength:	556 psi at 7 days 602 psi at 14 days 631 psi at 28 days 667 psi at 56 days	
Modulus of Elasticity:	4210 ksi at release	---
Shrinkage:	444 millionths at 88 days	---
Creep:	0.437 millionths at 88 days	---
Chloride Permeability (AASHTO T 277)	---	---

<u>Measured Properties from QC Tests of Production Concrete</u>

Precast Girders

For precast girders, the measured properties included the maximum girder temperature and the compressive strength. The maximum girder temperature was 158 °F. Compressive strength was tested at release and then at 56 days. The measured compressive strength ranged from 5600 to 10,900 psi at release, and at 56 days it varied from 7800 to 14,000 psi.

Cast-in-Place Deck

Table 5 summarizes the measured properties from QC tests of production concrete used in the cast-in-place deck of the bridge. The concrete deck and the cylinders were moist cured for 5 days.

TABLE 5: Measured Properties of Production Concrete for Cast-in-Place Deck

Sample No.(1)	Slump(2), mm	Air Content(3), %	Unit Weight(4), kg/m³	Compressive Strength(5), MPa			
				7 days	28 days	56 days	90 days
1	108	5.2	2286	28.9	35.3	39.1	40.7
2	89	5.3	2294	32.1	37.7	42.9	42.8
3	95	6.0	2310	29.8	36.7	41.0	41.9
Average	97	5.5	2297	30.3	36.6	41.0	41.8

NOTES: AASHTO (1) T 141, (2) T 119, (3) T 152, (4) T 121, (5) T 22.

Measured Properties from Research Tests of Production Concrete

Precast Girders

Two girders, identified as Girder 1 and Girder 2, were tested for research purposes. Measured properties included compressive strength, modulus of elasticity, and splitting tensile strength. The measured properties from research tests of production concrete used in the precast girders are shown in Table 6.

TABLE 6: Measured Properties from Research Tests of Production Concrete Used in the Precast Girders

Curing (1)	Girder No.	Age, days				
		7	14	28	56	90
Compressive Strength, psi (2)						
Air	1	7980	8130	8910 (3)	8630	9010 (3)
	2	7650	9980	9210	9820	10,720 (3)
Moist	1	8650	10,010	10,180	10,750	9690 (3)
	2	7890	10,410	10,060	10,220	10,970
Modulus of Elasticity, ksi (2)						
Air	1	5000	5000	6000 (3)	5000	5000 (3)
	2	5000	5500	6500	5000	5000 (3)
Moist	1	5000	5500	5000	6000	6000 (3)
	2	5000	6000	5500	6000	6000
Modulus of Rupture, (ASTM C 78), psi (4)						
Air	1	775	860	975	1085	—
	2	835	835	—	—	—
Moist	1	—	735	1185	1115	—
	2	—	815	1290	1285	—
Splitting Tensile Strength (ASTM C 496), psi (5)						
Air	1	495	530	525	565	—
	2	525	515	485	550	—
Moist	1	620	680	755	750	—
	2	600	710	755	815	—

NOTES: Test results are the average of two specimens except as noted.
(1) Air-cured specimens were steam cured with the girders followed by air curing in the laboratory. Moist cured specimens were cured in a fog room from the beginning.
(2) 4x8-in cylinders. (3) Single test result. (4) 3x3x11.5-in beams. (5) 6x12-in cylinders.

Creep and shrinkage tests were performed on concrete mixes that were used for the two girders fabricated for research purposes. The specimens were 4x8-in. cylinders, and they were steam cured with girders followed by curing at 73 ^{o}F at 50% RH. The creep data for production concrete used in the precast girders is shown in Figure 1, and shrinkage data for production concrete used in the precast girders is shown in Figure 2.

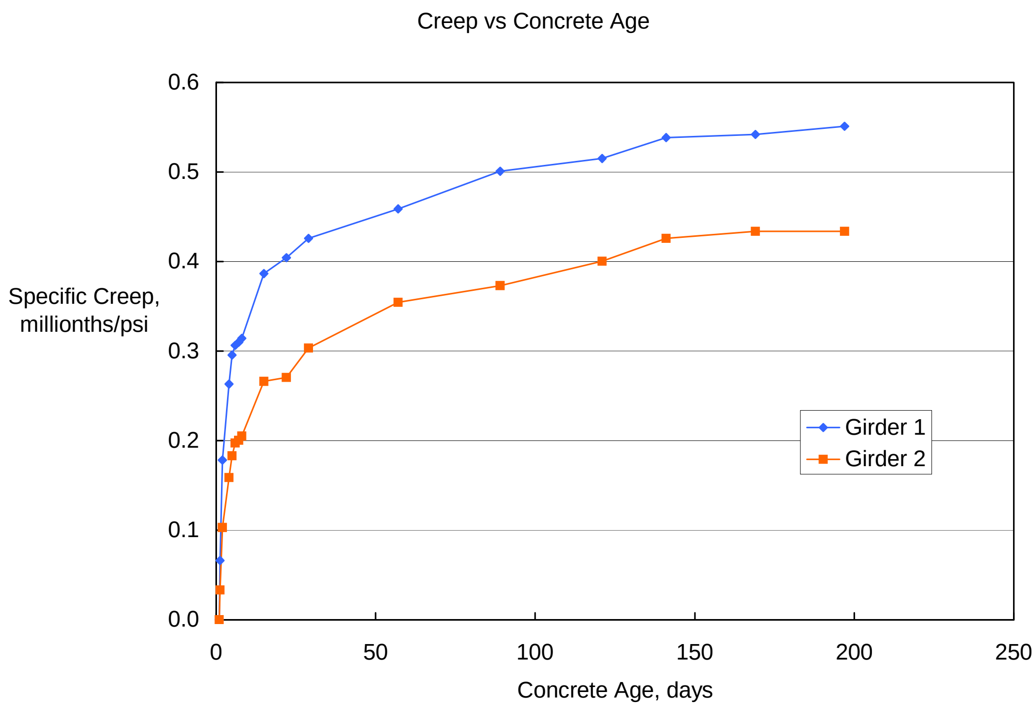


Figure 1. The creep data for production concrete used in the precast girders.

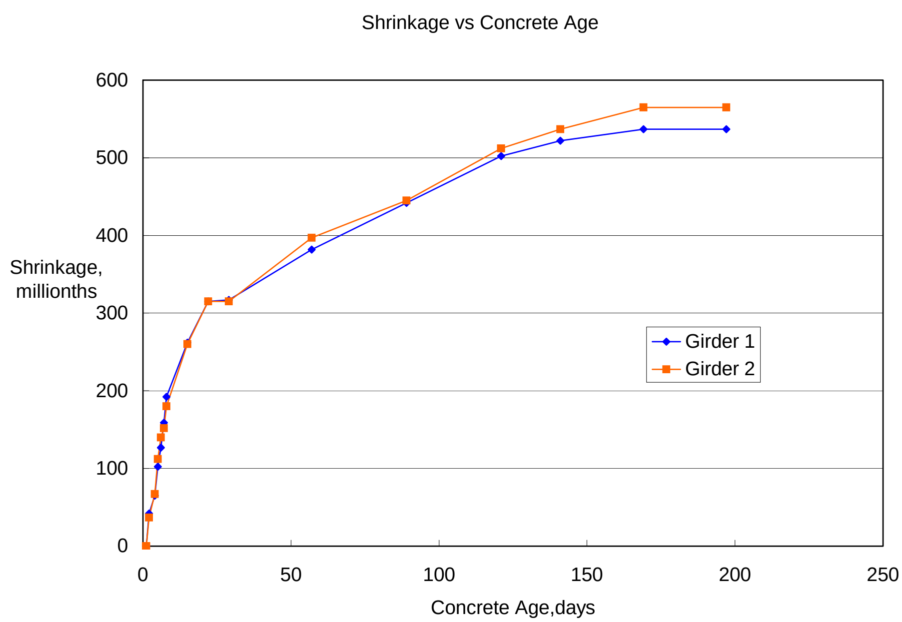


Figure 2. Shrinkage data for production concrete used in the precast girders.

Cast-in-Place Deck

The measured properties from research tests of production concrete for the bridge deck included modulus of elasticity, chloride permeability and abrasion resistance. The results from modulus of elasticity test (ASTM C 469) are listed in Table 7. Chloride permeability test (AASHTO T 277) was conducted in two laboratories, and the results are present in Table 8. The result from abrasion resistance tests (ASTM C 779, Procedure A) are shown in Table 9.

TABLE 7: Modulus of Elasticity of Production Concrete for the Cast-in-Deck

Sample No.	Age, days	Comp. Strength, MPa	Modulus of Elasticity, MPa
1	28	38.4	27,400
2	28	38.8	28,000

NOTE: Measured on 152x305-mm cylinders.

TABLE 8: Chloride Permeability (AASHTO T 277) of Production Concrete for the Cast-in-Place Deck

Laboratory	Permeability, coulombs		Test Date
	Individual	Average	
CTL	6211, 5334, 5246	5597	6/20/97
FHWA	4264, 4617	4440	11/25/97
CU, Boulder	3797, 3904	3850	12/25/97
	2945, 3005	2975	1/4/98

TABLE 9: Abrasion Resistance of Production Concrete for the Cast-in-Place Deck

Sample No.	Depth of Wear, mm	
	30 min.	60 min.
1	0.66	1.19
2	0.66	1.22
3	0.61	1.14
Average	0.64	1.14

NOTES: Test procedure followed ASTM C 779 - Procedure A. Tests were performed at a concrete age of 42 days.

Actual Method of Deck Placement

The wind and low humidity in Colorado are a problem and can contribute to deck cracking. A membrane-forming curing compound was placed immediately upon finishing and a moist cure was started when the deck concrete could be walked on without damage. The moist cure was continued for 5 days. Construction on this project began in November 1996 and was completed in June 1998.

Average Daily Traffic (ADT) and Average Daily Truck Traffic (ADTT)

Average daily traffic for northbound lanes was calculated based on a count of all vehicles crossing the bridge during a 10 minute period beginning at 1350 hrs on December 19, 2007. These vehicle counts gave at an ADT of 138,960 and an ADTT of 8,496.

Average daily traffic for southbound lanes was calculated based on a count of all vehicles crossing the bridge during a 10 minute period beginning at 1330 hrs on December 19, 2007. These vehicle counts gave an ADT of 164,880 and an ADTT of 6,624.

Exposure Condition of the Bridge

The I-25 Bridge over Yale Avenue is on I-25 just south of Denver, Colorado. The new bridge improved the clearances over Yale Avenue. The National Weather Service reports that the average high temperature is 93.0°F in July and the average low temperature is 38°F in February. The minimum temperature varies between 60.8°F in July and 16.5°F in February. The normal precipitation varies between 3.95 inches per month in June to 0.03 inches per month in January. Based on the National Weather Service record there is annual exposure to wet/dry and freeze/thaw cycles. The normal annual snowfall is 61.7 inches.

Performed Maintenance

No documents were found which would indicate any maintenance had been performed since bridge construction in 1998.

Inspection Reports

No previous inspection reports for this bridge were located.

IV. Bridge Deck Inspection

PSI personnel performed a close visual inspection of the bridge deck during the week of December 17, 2007. The results of that inspection are summarized as follows.

General Condition of the Deck Top Surface

Figure 5 shows the general layout of the decks for the I-25 Bridge over Yale Avenue. Core locations and the pothole on the bridge deck are shown in Figure 5. No apparent sign of abrasion damage, freeze-thaw damage, D-cracking, pop-out, and alkali aggregate reaction (AAR) was observed. Surface defects observed and documented during visual inspection primarily included a pothole. The concrete bridge deck was not visible since it was covered by asphalt overlay. No cracks were observed on the asphalt overlay surface.

General Condition of the Bridge Underside

The underside of the bridge was inspected from the ground without the aide of any access equipment. The bottom of the box girders was visible. The underside of the bridge was generally in good condition.

General Condition of the Girders

The girders were inspected from the ground without the aide of any access equipment. Visible cracks were observed in only one girder. A crack approximately 32-inches long was observed on the 16th girder, located about 15 feet from the south abutment (see photos 10 and 11). The crack was about 1/16- to 1/8-in. wide. This crack was believed to be the cause of impaction during transportation or construction, since there was an impacting mark on the side of the box girder.

Concrete Core Samples

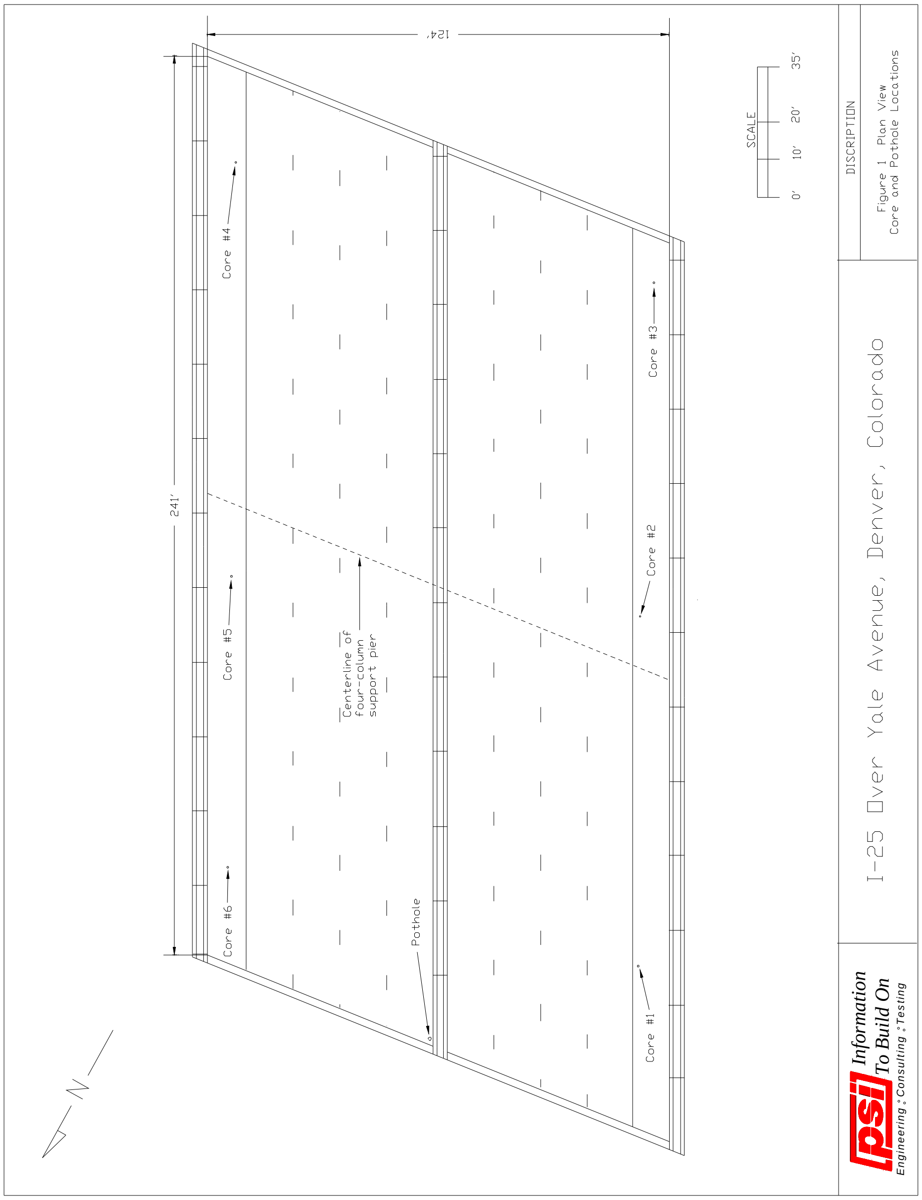
Six core samples were retrieved from the decks during the inspection. Core sample locations are shown in Figure 5. The cores were 3 in diameter and were labeled as 1 through 6. Petrographic analysis was not performed on the samples.

Preliminary Conclusions

The construction of the I-25 Bridge over Yale Avenue was completed in 1998. Researchers from the University of Colorado performed testing on standard pull-out strength, transfer length, and development length on the prestressed concrete box beams.

The visual inspection of the bridge decks as part of our study was performed about nine years after the construction. The bridge deck was covered by an asphalt overlay about 3- to 4-inches thick. No cracks were visible on the asphalt pavement. There was a pothole in the inner northbound lane close to the parapet, adjacent to the expansion joint (Figure 5). There were some hair-size cracks on the parapet, but no significant damage was observed.

Respectfully Submitted,
Professional Service Industries, Inc.
Structural Investigation & Petrography Department


124'
241'
Core #1
Core #2
Core #3
Core #4
Core #5
Core #6
Pothole
Centerline of four-column support pier
N
SCALE
0' 10' 20' 35'
DISCRIPTION
Figure 1 Plan View
Core and Pothole Locations
I-25 Over Yale Avenue, Denver, Colorado
psi
Information To Build On
Engineering • Consulting • Testing

Photographic Documentation

	Photo 1: View from the east side of the bridge.
	Photo 2: View from the west side of the bridge.

Photo 3: View looking south at the northbound lanes.

Photo 4: View showing bridge substructure consisting of box girders and columns.

	Photo 5: View looking south at the southbound lanes.
	Photo 6: View looking at the east side of the northbound lanes.

Photo 7: View showing typical cracks on the bridge parapet. Crack width is less than 1/32 in. and is categorized as hairline.

Photo 8: View showing cracks on the abutment at span ends.

Photo 9: View showing underside of bridge at span end.

Photo 10: View showing the impact crack in the 16th girder.

	Photo 11: Close-up view showing of the impact crack in the 16th girder.

APPENDIX C

Jonesboro Road Bridge, Atlanta, Georgia

HIGH PERFORMANCE CONCRETE BRIDGE DECK INVESTIGATION

Jonesboro Road Bridge over Interstate 75 Atlanta, Georgia

I. BACKGROUND

The Jonesboro Road Bridge over I-75 on State Route 920, located in Henry County, south of Atlanta, is the first High Performance Concrete (HPC) Bridge built in Georgia (see photos 1 through 3). It replaced a steel girder bridge carrying Jonesboro Road, a route connecting Lovejoy, Georgia to the west and McDonough, Georgia to the east. All girders and the cast-in-place deck were fabricated using high performance concrete (HPC). The bridge, which has 4 spans on both eastbound and westbound lanes, measured 54.4, 127.1, 127.2, and 41.7 ft (16.25, 38.75, 38.75, and 13.75 m), respectively. The clear width of the bridge is 90 ft (27.4 m), carrying five lanes of traffic with bike lanes and shoulders. The bridge has a skew that varies from 27 to 31 degrees to accommodate a horizontal curve. The Jonesboro Road Bridge was constructed in two stages to handle traffic during construction. After the completion of the first stage, the bridge opened to traffic in February 2002.

The Jonesboro Road Bridge was designed in accordance with the AASHTO Standard Specifications for Highway Bridges (1996) using MS 18 and/or military design live load. Each of the four spans was simply-supported with 13 HPC girders made with design strengths of 10,280 psi (70 MPa). AASHTO Type IV girders were used for the 127-ft (38.75-m) long spans and AASHTO Type II girders were used for the 53-ft and 45-ft shorter spans. Concrete diaphragms were used at mid-span locations for Spans 1 and 4. Spans 2 and 3 had diaphragms at 1/3 span lengths. The use of 127-ft (38.75-m) long AASHTO Type IV beams minimized the overall depth of the superstructure. Beam spacing is 7.60 ft (2.31 m) (see photo 4). Grade 60 ASTM #5 rebar was used in the cast-in-place deck top layer.

The Jonesboro Road Bridge has a composite bridge deck. The cast-in-place deck is 8-in. thick. The deck was formed with stay-in-place (SIP) galvanized steel deck forms that were connected to the girders with welded shear connectors. The prestressed girder strands are 0.6 in. (15.2 mm) in diameter. The cast-in-place concrete overlay was reinforced with epoxy coated rebar. The top reinforcing mat had a specified cover of 2.75 in. (70 mm) while the bottom mat had a specified cover of 1 in. (25.4 mm) above the metal decking. A cast-in-place normal strength (3500 psi or 24 MPa) concrete barrier was constructed on each side of the bridge. The specified compressive strength for the deck concrete was 7250 psi (50 MPa) at 56 days. The deck was constructed simultaneously using similar construction techniques by the same personnel.

The Jonesboro Road Bridge is part of a demonstration project for the use of HPC in bridge structures, which are co-sponsored by the Federal Highway Administration

(FHWA) and the Georgia Department of Transportation (GDOT). Georgia Institute of Technology investigated the advantages of using HPC in bridges. The project is expected to demonstrate that HPC girders provide greater economy in highway bridge construction by permitting smaller depth girders to be used for longer spans while also allowing wider girder spacing. Furthermore, the HPC deck will provide greater durability with reduced long-term maintenance. With construction of this bridge, designs of precast, prestressed girders using HPC with compressive strengths up to 10,000 psi (70 MPa) were approved by GDOT.

II. SCOPE OF SERVICES

Professional Service Industries (PSI) is under contract with the Federal Highway Administration (FHWA) to conduct bridge deck inspections for HPC bridges. Our scope of services on this bridge included a series of tasks and sub-tasks, which are described as follows:

- Collect all available information relevant to the construction of each bridge, including:
    - Deck Concrete Properties
    - Specified Deck Concrete Construction Procedures
    - Approved Concrete Mix Proportions
    - Measured Properties from QC
    - Other Measured Properties
    - Actual Method of Deck Placement
    - Average Daily Traffic (ADT)
    - Exposure Condition of the Bridge
    - Any Performed Maintenance
    - Any Inspection Reports
- Visually inspect the bridge, and obtain the following:
    - General condition of the deck top surface
    - Determination of the maximum crack width
    - General condition of the deck underside
    - General condition of the girders
    - Photograph areas of significant deterioration
    - Prepare drawings locating defects
    - Extract 8 concrete core samples

III. COMPILATION OF BRIDGE CONSTRUCTION INFORMATION

Information sources used for this report include bridge drawings, research reports from Georgia Institute of Technology, bridge summary data sheets, and technical information contained in FHWA's "Compilation and Evaluation of Results from High Performance Concrete Bridge Projects" version 3.

Deck Concrete Properties

It is noted that three classes of concrete were used in the Jonesboro Road Bridge. They are class AAA (HPC) for precast bridge members, class AA (HPC) for cast-in-place superstructure, and class AA high early-age strength concrete which may use Type III cement as called for on plans. However, deck concrete (class AA HPC) properties are the focus of our report. The bridge deck had a specified concrete compressive strength of 7200 psi (50 MPa) at 56 days. A rapid chloride permeability of 2000 coulombs or less at 56 days was specified for cast-in-place deck concrete. Table 1 lists the specified concrete properties used in the cast-in-place decks.

TABLE 1: Specified Concrete Properties

Property	Deck Class AA (HPC)
Minimum Cementitious Materials Content:	386 kg/m^3
Max. Water/Cementitious Materials Ratio:	0.35
Min., Max. Percentage of Fly Ash:	0, 15
Min., Max. Percentage of Silica Fume:	5, 10
Min. Percentage of GGBFS:	--
Max. Percentage of GGBFS:	0
Max. Aggregate Size:	19 mm
Slump:	50-125 mm
Air Content:	3.5~6.5%
Compressive Strength - Design:	50 MPa @ 56 days
Chloride Permeability (AASHTO T 277):	$\leq$ 2000 coulombs at 56 days

Specified Deck Concrete Construction Procedures

Prior to the beginning of deck placement operations, the contractor was required to construct a demonstration slab (6 × 5 m) using the proposed HPC mix design for the concrete deck at a location approved by the Engineer. It was also required that the demonstration slab have the same bar reinforcement, same slab thickness, and same forming as the bridge to be constructed, using the same equipment and operations proposed for the bridge deck. Immediately after the disappearance of the water sheen, the surface finish was to be applied, followed by fogging to maintain a film of water on the fresh concrete surface. The surface was required to be kept wet up to the time sheet curing covers were applied. Curing covers were required to be thoroughly soaked and applied as soon as the concrete was set sufficiently to prevent damage. The contractor was required to strictly adhere to the manufacture's written recommendations regarding the use of admixtures, including storage, transportation and method of mixing.

Approved Concrete Mix Proportions

Deck

Class AA HPC was used in the cast-in-place deck of the Jonesboro Road Bridge. The approved proportions for cast-in-place deck are shown in Table 2.

TABLE 2: Approved Mix Proportions for Charenton Canal Bridge

Mix Parameters	Deck (Class AA HPC) Stage I	Deck (Class AA HPC) Stage II
Cement Brand:	Southern Cement	Southern Cement
Cement Type:	I	I
Cement Fineness:	3500 Blaine	3500 Blaine
Cement Quantity:	362 kg/m^3	362 kg/m^3
Fly Ash Brand:	None	Boral
Fly Ash Type:	--	F
Fly Ash Quantity:	--	61 kg/m^3
Silica Fume Brand:	Euclid MSA	Euclid MSA
Silica Fume Quantity:	7 kg/m^3	15 kg/m^3
Fine Aggregate Type:	Natural Sand	Natural Sand
Fine Aggregate FM:	2.35	2.35
Fine Aggregate SG:	2.62	2.62
Fine Aggregate Quantity:	821 kg/m^3	748 kg/m^3
Coarse Aggregate, Max. Size:	3/4-in.	3/4-in.
Coarse Aggregate Type:	Granite Gneiss	Granite Gneiss
Coarse Aggregate Quantity:	1008 kg/m^3	1008 kg/m^3
Coarse Aggregate SG:	2.67	2.67
Water:	134 kg/m^3	144 kg/m^3
Water Reducer Brand:	Eucon WR 91	Eucon WR 91
Water Reducer Type:	A	A
Water Reducer Quantity:	0.75 L/m^3	0.56 L/m^3
High Range Water Reducer Brand:	Eucon 1037	Eucon 1037
High Range Water Reducer Type:	A and F	A and F
High Range Water Reducer Quantity:	5.54 L/m^3	3.06 L/m^3
Air Entrainment Brand:	Euco AEA 92	Euco AEA 92
Air Entrainment Type:	Synthetic Organic Chemical	Synthetic Organic Chemical
Air Entrainment Quantity	0.63 L/m^3	0.46 L/m^3
Water/Cementitious Materials Ratio:	0.34	0.34

Measured Properties from QC Tests of Production Concrete

Cast-in-Place Deck

Measured properties of concrete mix for the cast-in-place deck are summarized in Table 3.

TABLE 3: Measured Properties of Approved Concrete Mixes for Cast-in-Place Deck Panels and Precast Girders

Measured Concrete Properties	Class AA (HPC) Cast-in-place Decks Stage I	Class AA (HPC) Cast-in-place Decks Stage II
Slump:	114 mm	150 mm
Air Content:	5.9%	5.0%
Unit Weight:	2356 kg/m^3	2390 kg/m^3
Compressive Strength:	62 MPa at 28 days	57 MPa at 28 days
Rapid Chloride Permeability (AASHTO T 277):	1650 coulombs at 56 days	1100 coulombs at 56 days

The measured properties from QC tests of Class AA HPC production concrete used in the cast-in-place deck of the Jonesboro Road Bridge are shown in Table 4.

TABLE 4: Measured Properties from QC Tests of Class AA (HPC) Cast-in-Place Deck Concrete

Properties	Production Concrete for Cast-in-Place Deck Class AA (HPC) Stage I
Slump:	100-172 mm
Air Content:	3.2-6.5%
Unit Weight:	2390 kg/m^3
Compressive Strength:	53.4 MPa at 56 days
Chloride Permeability:	3963 coulombs at 56 days

Measured Properties from Research Tests of Production Concrete

Cast-in-Place Deck

The compressive strength, modulus of elasticity, and coefficient of thermal expansion of Class AA HPC concrete used in the Jonesboro Road Bridge deck are shown in Table 5 and Table 6.

TABLE 5: Measured Compressive Strength from Research Tests of Production Concrete Used in the Cast-in-Place Deck

Span	Concrete Age				
	1	3	7	28	56
Span 1	2073	3922	4813	5719	6230
Span 2	2497	4411	5268	6715	6880

NOTES:
(1) All specimens were 4 × 8-in. cylinders.
(2) Unit weight of hardened concrete is 145.1 lb/ft^3 for Span 1 and 144.8 lb/ft^3 for Span 2.

TABLE 6: Measured Modulus of Elasticity from Research Tests of Production Concrete Used in the Cast-in-Place Deck

Span	Compressive Strength, psi	Modulus of Elasticity, ksi
Span 1	6230	3546
Span 2	6880	3673

NOTES: All specimens were 6 × 12-in. cylinders tested at 56 days following ASTM C31 standard curing.

The chloride permeability data for Class AA HPC concrete used in the cast-in-place deck is shown in Table 7.

TABLE 7: Chloride Permeability of Production Concrete Used in the Deck (Stage 1)

Span	Chloride Permeability, coulombs
Span 1	4447
	6162
	4384
	5195
Span 2	4162
	4170
	3790
	5160

NOTES: Specimens were tested at 56 days following ASTM C1202.

Actual Method of Deck Placement

The Jonesboro Road Bridge was constructed in two stages to handle traffic during construction. In the first stage deck placement, the maximum chloride permeability of 2000 coulombs was exceeded. For the second stage deck placement, Class F fly ash and more silica fume were included in the concrete mixture. This helped to reduce the chloride permeability to less than 2000 coulombs.

Construction of the deck occurred in the spring of 2000, with the concrete for the deck delivered by truck and pumped to the deck surface. Concrete was distributed by a mechanical spreader. The concrete was internally vibrated to provide proper consolidation and avoid internal segregation. A final troweled finish was applied followed by tinning for enhanced skid resistance. Surface finishing consisted of vibratory screed followed by a roller screed. Fogging of the concrete deck started when the concrete was in the plastic state. The deck was cured using water soaked burlap covered with white plastic sheet for seven days. The wet burlap was kept moist. This construction practice is particularly important for HPC with low w/cm ratios.

Average Daily Traffic (ADT)

Average daily traffic for both eastbound and westbound lanes was calculated based on a count of all vehicles crossing the bridge during a 5 minutes period beginning at 1250 hrs on March 8, 2004. These vehicle counts gave an ADT of 19,872. The estimate of traffic flow was made while the PSI inspection crew was on-site.

Exposure Condition of the Bridge

The Jonesboro Road Bridge over I-75 is on State Route 920, located in Henry County, south of Atlanta, Georgia. The bridge carries high volume of traffic. The area surrounding the bridge is developed with mixed residential and commercial land use. The National Weather Service reports that the normal maximum temperature varies between 89°F in July and 52°F in January. The normal minimum temperature varies between 70°F in July and 33°F in January. The normal precipitation varies between 5.6 inches per month in March to 3.0 inch per month in October. Very few days per year the temperature drops below 32°F. Based on this information, the bridge has minimal annual exposure to wet/dry and freeze/thaw cycles.

Performed Maintenance

No documents were found which would indicate any maintenance had been performed since the bridge was constructed in 1998.

Inspection Reports

As part of the project, bridge instrumentation and bridge monitoring are being performed by Georgia Institute of Technology. A number of 3-¾-in. cores were drilled during previous bridge inspection. The researchers have developed an instrumentation program to monitor the structural performance of the bridge and its components as described in "Evaluation of Georgia's High Performance Concrete Bridge, Task 6 - Use of High-Strength/High-Performance Concrete for Precast Prestressed Concrete Bridges in Georgia"

General Condition of the Deck Top Surface

Figure 1 shows the general layout of the decks for the Jonesboro Road Bridge. Results of the visual inspection of the decks are shown in Figure 2a and Figure 2b. Surface defects observed and documented during visual inspection primarily included transverse cracks and diagonal cracks (see photos 6 through 8). Other defects observed and documented included small spalls at joints and cracks and broken tinned edges (see photos 10 and 11). However, apparent signs of other serious damages such as freeze-thaw, D-cracking, alkali-silica reaction, and alkali-aggregate reaction were not observed. Both transverse and diagonal cracks were observed on the top surface of cast-in-place deck. Only at one location around a rectangular patch was longitudinal cracking found. Efflorescence was observed on the underside of the bridge. Patches of drilled cores (3-in. diameter) resulting from previous investigations by others, were also observed (see photo 9).

A total of 91 cracks (61 traverse cracks and 30 diagonal cracks) were recorded during the visual survey of the bridge decks (see Figure 2). Of the 91 cracks, 41 cracks were recorded on the eastbound bridge and 50 cracks were recorded on the westbound bridge. The sum of crack lengths was 626.2 ft over a bridge deck area of 31,937.6 ft^2. Crack density (total crack length/deck area) for the eastbound and westbound bridges combined was calculated to be 0.020 ft/ft^2.

Though the number of transverse crack counts for eastbound (33 cracks) and westbound bridge (28 cracks) appears to be similar, the crack density on the eastbound bridge is greater than that in westbound bridge. However, westbound bridge appears to have more diagonal cracks (22 cracks) compared to the eastbound bridge (8 cracks). The majority of the cracks consisted of hairline cracks with a width of less than 0.031 in. A typical transverse crack on the bridge decks is shown in photo 7. The number of cracks that were classified as hairline cracks totaled 81 with a combined total length of 528.5 ft. A relatively small number of cracks were classified as fine cracks, with widths in the range of 0.031 to 0.063 in. The number of these cracks was 10 and their combined total length was 97.7 ft.

It can be noted that cracks were typically limited at span ends along the skew. Small surface spalls, which either occurred due to breaking of tined edges or the crack edges, were observed. Photo 10 illustrates typical spalling due to breaking of crack edges. Figures 2a and Figure 2b also illustrate the locations of drilled cores, which resulted from previous investigations by others.

The number, length and density of cracks for each structure are shown in Tables 8 through 11, and described below according to the crack type.

Transverse Cracks: Figure 2 illustrates the transverse cracks that were identified on the surface of the bridge decks. Tables 8 and 9 provide detailed information regarding transverse cracks identified on the eastbound and the westbound bridge decks. The crack densities (crack length per deck area) range from 0 to 0.043 ft/ft.^{2} for the 8 spans (including both eastbound and westbound lanes) investigated.

The transverse crack counts in the eastbound and westbound bridges were comparable. A total of 33 cracks were observed in four spans of the eastbound bridge with a combined total length of 285.1 ft. A total number of 28 cracks with a combined total length of 127.3 ft were observed in the four spans of the westbound bridge (see Tables 8 and 9). The crack length per deck area for the eastbound and westbound bridges was 0.018 ft/ft.^{2} and 0.008 ft/ft^2 respectively.

TABLE 8: Measured Transverse Cracks on the Surface of Eastbound Bridge Decks

Eastbound Traverse Cracks	Count	Length Range (ft)	Mean Length of Cracks (ft)	Median Length of Cracks (ft)	Total Length of Cracks (ft)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span 1	4	3 to 8.25	4.8	4	19.3	2414	0.008
Span 2	26	4 to 21	9.6	7.3	250.3	5756	0.043
Span 3	0	NA	NA	NA	NA	5756	0
Span 4	3	5 to 5.5	5.2	5	15.5	2043	0.008

NOTES: Transverse cracks include cracks oriented parallel to skewed joints

TABLE 9: Measured Transverse Cracks on the Surface of Westbound Bridge Decks

Westbound Transverse Cracks	Count	Length Range (ft)	Mean Length of Cracks (ft)	Median Length of Cracks (ft)	Total Length of Cracks (ft)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span 1	4	2 to 5	3.2	3.0	16	2414	0.007
Span 2	5	3 to 8	4.1	3.25	20.6	5756	0.004
Span 3	18	1 to 9	5.0	5.3	89.7	5756	0.016
Span 4	1	1 to 1	1	1	1	2043	0.001

NOTES: Transverse cracks include cracks oriented parallel to the skewed joints

Diagonal Cracks: The diagonal crack densities (crack length per deck area) ranged from 0 to 0.029 ft/ft^2 for the 8 spans (including both eastbound and westbound lanes) investigated. Diagonal cracks in the westbound bridge decks were more significant than that in the eastbound bridge. These diagonal cracks were typically present near the joints. The crack length per deck area was 0.004 ft/ft^2 in eastbound bridge and 0.009 ft/ft^2 in westbound bridges (see Tables 10 and 11), respectively.

TABLE 10: Measured Diagonal Cracks on the Surface of Eastbound Bridge Decks

Eastbound Traverse Cracks	Count	Length Range (ft)	Mean Length of Cracks (ft)	Median Length of Cracks (ft)	Total Length of Cracks (ft)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span 1	2	8 to 12	10	10	20	2414	0.008
Span 2	5	5 to 12	7.5	7	37.3	5756	0.006
Span 3	1	5 to 5	5	5	5	5756	0.001
Span 4	0	NA	NA	NA	NA	2043	0

TABLE 11: Measured Diagonal Cracks on the Surface of Westbound Bridge Decks

Westbound Transverse Cracks	Count	Length Range (ft)	Mean Length of Cracks (ft)	Median Length of Cracks (ft)	Total Length of Cracks (ft)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span 1	11	2 to 18	6.7	4.6	67.3	2414	0.029
Span 2	4	1 to 13	5.7	4.5	22.9	5756	0.004
Span 3	2	9 to 16.3	12.7	12.7	25.3	5756	0.004
Span 4	5	3 to 13	7.2	5	36	2043	0.018

Longitudinal Cracks: No longitudinal cracks were observed in the eastbound and westbound decks.

Maximum Crack Width

The maximum width of transverse cracks was measured to be 0.040 in. According to ACI 201, these crack widths are classified as fine cracks.

About 89% of the cracks on the two bridges were hairline cracks with a width of less than 0.031 in. The remaining 11% of the cracks were classified as fine cracks with widths in the range of 0.031 to 0.063 in. The fine cracks were generally located at span ends along the skew and some exhibited spalling due to the breaking of the edges (see photo 10).

General Condition of the Deck Underside

The underside of the deck exhibits no signs of distress. At very limited locations, efflorescence was observed. Photos 4 and 5 show general views of the underside of the deck.

General Condition of the Girders

The girders were inspected without the aide of any access equipment. No signs of distress were noted on any of the girders.

Concrete Core Samples

Eight cores, approximately 3-½-inches long and 3-¾-inches in diameter, were retrieved from the deck. The core sample locations are shown in Figure 2. The locations were evenly distributed along each shoulder of the bridge. The cores were labeled as GA-1 through GA-8 and were transferred to FHWA for further analysis.

TABLE 17: Core Dimensions

Sample	GA-1	GA-2	GA-3	GA-4	GA-5	GA-6	GA-7	GA-8
Diameter (in.)	3¾	3¾	3¾	3¾	3¾	3¾	3¾	3¾
Length (in.)	3¼	3¼	3	3¼	3¼	3	3 ½	3

Petrographic examination was performed on eight concrete cores that were retrieved from the bridge deck. The collected cores represented the cast-in-place concrete in the bridge deck. Cores GA#1, GA#2, GA#3, and GA#4 were from the eastbound of the bridge, while cores GA#5, GA#6, GA#7, and GA#8 were from the westbound of the bridge. Visual inspection revealed that five cores (GA#1, GA#2, GA#3, GA#4 and GA#6) had cracks along the length of the cores. The cause of the cracking was uncertain. It was speculated that the cracks were shrinkage related. There was no evidence of any material related deterioration in the concrete. The rest of the three cores appeared intact, and no defects were visible.

The coarse aggregate in the concrete was crushed granite. Coarse aggregate particles were mostly angular, and the maximum size measured from the examined core samples was about 1 inch. Preferential orientation of coarse aggregate particles was not observed in this concrete, nor was segregation. The fine aggregate fraction was mainly composed of quartz. The source of the fine aggregate was natural sand. The majority of the fine aggregate particles were angular, only a small portion was rounded.

The cement paste was well hydrated considering the age of the concrete. The cement paste contained some unhydrated cement particles. Fly ash particles were present in the cement paste matrix. In general, the cement/aggregate bonding was moderately strong.

The concrete was air entrained. Small, spherical air voids were observed in the concrete, and the air voids were well dispersed in the cement paste. The air content was estimated to be relatively low. A small amount of entrapped air voids were also present in the concrete.

Fine cracks were observed under the microscope. Micro-cracking was present in the cement paste. These cracks were likely due to drying shrinkage

Ettringite was sporadically found in air voids. There was no evidence of deterioration associated with the presence of the ettringite in the concrete. It is common to see ettringite as secondary deposit in concrete.

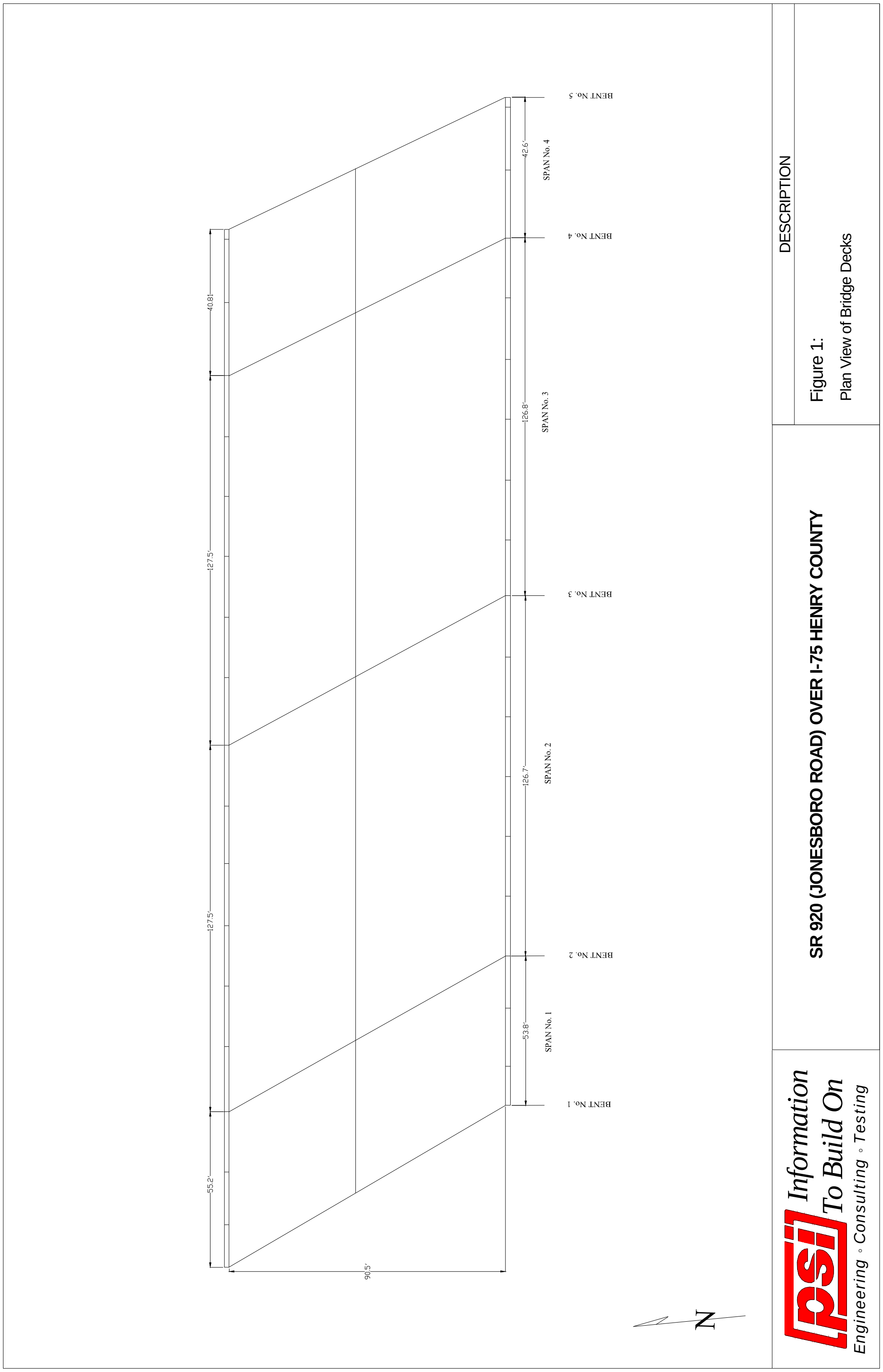
Preliminary Conclusions

The construction of the Jonesboro Road Bridge was part of a demonstration project for the use HPC in bridge structures. It was completed in 2002. Researchers from Georgia Institute of Technology performed material testing, bridge instrumentation, and bridge monitoring throughout this project.

The visual inspection of the bridge decks as part of our study was performed about two years after the bridge opened to traffic. A total of 91 transverse and diagonal cracks were recorded on the bridge with a combined total crack length of 626.2 ft over a bridge deck area of 31,937.6 ft^2. However, About 89% of the cracks on the two bridges were hairline cracks with a width of less than 0.031 in. The remaining 11% of the cracks were classified as fine cracks with widths in the range of 0.031 to 0.063 in. No major distresses were observed in our bridge survey.

In general, the work on the Jonesboro Road Bridge shows that HPC designs provide significantly higher strength that can lead to more efficient designs and improved durability.

Respectfully Submitted,
Professional Service Industries, Inc.
Structural Investigation and Petrography Department


DESCRIPTION
Figure 1:
Plan View of Bridge Decks
SR 920 (JONESBORO ROAD) OVER I-75 HENRY COUNTY
psi
Information
To Build On
Engineering ◦ Consulting ◦ Testing
N
BENT No. 1
BENT No. 2
BENT No. 3
BENT No. 4
BENT No. 5
SPAN No. 1
SPAN No. 2
SPAN No. 3
SPAN No. 4
53.8'
126.7'
126.8'
42.6'
55.2'
127.5'
127.5'
40.8'
90.5'

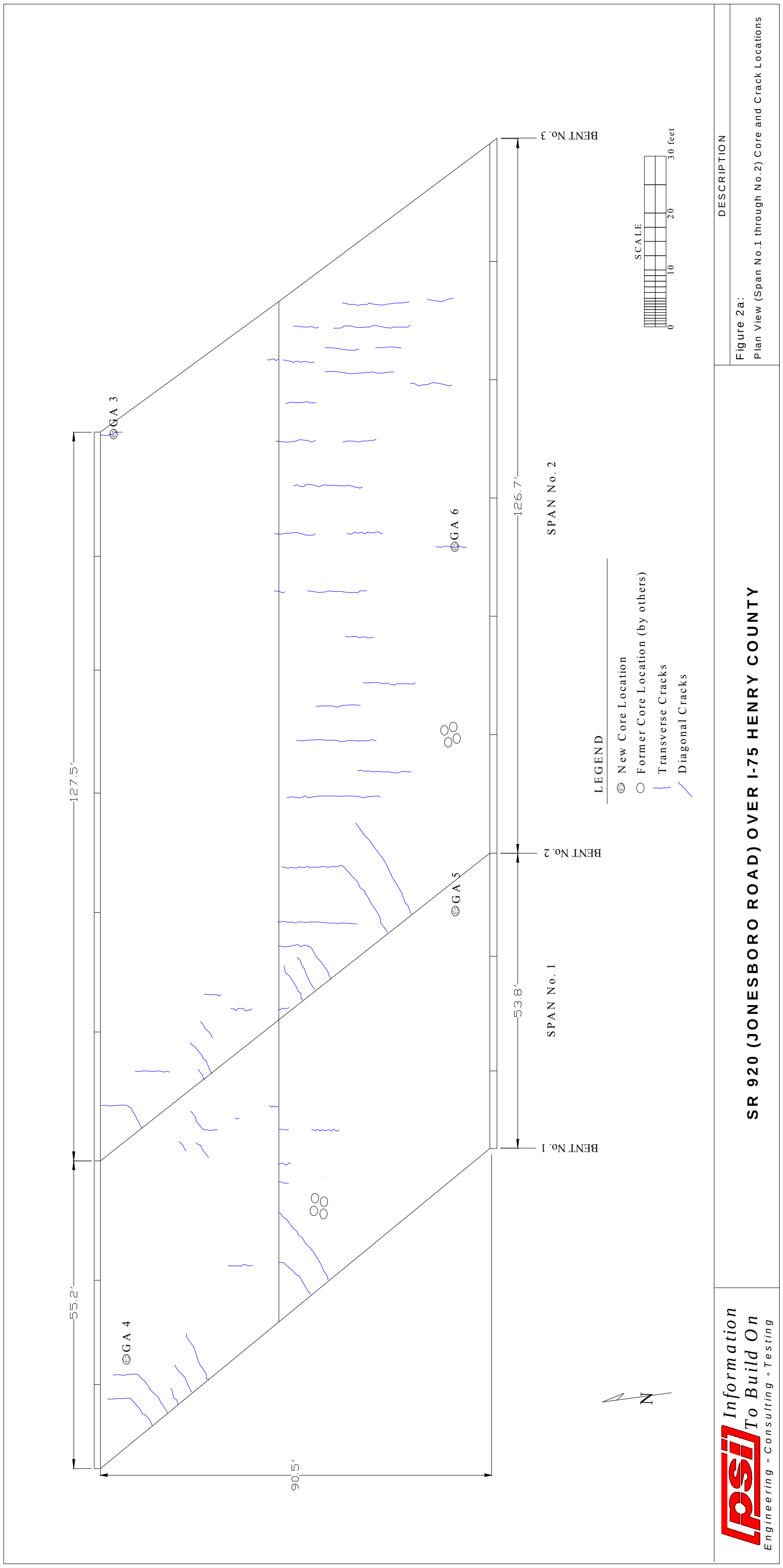


Figure 2a:
Plan View (Span No.1 through No.2) Core and Crack Locations

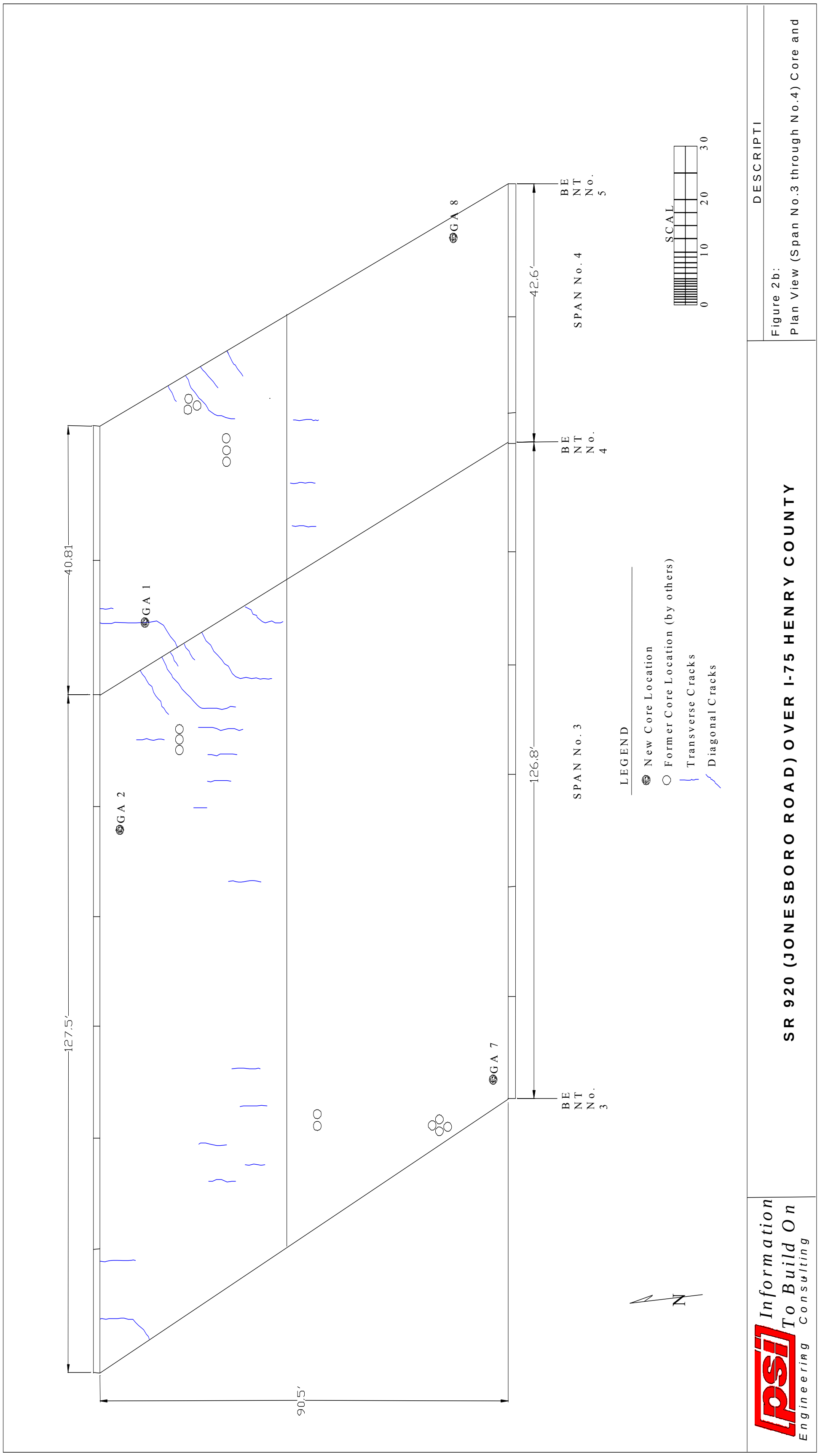


Figure 2b:
Plan View (Span No.3 through No.4) Core and
DESCRIPTI
SR 920 (JONESBORO ROAD) OVER I-75 HENRY COUNTY
SPAN No. 3
SPAN No. 4
BENT No. 3
BENT No. 4
BENT No. 5
LEGEND
New Core Location
Former Core Location (by others)
Transverse Cracks
Diagonal Cracks
GA 1
GA 2
GA 7
GA 8
40.81
127.5'
126.8'
42.6'
90.5'
SCAL
0
10
20
30
N
PSI
Information To Build On
Engineering Consulting

Photographic Documentation

	Photo 1: View looking northeast. The eastbound bridge is in foreground.
	Photo 2: View looking west from the eastern end of the westbound bridge.
	Photo 3: View looking east from the western end of the eastbound bridge.

	Photo 4: View of the underside of bridge, the pier in foreground supports Span 2.
	Photo 5: General condition of the girders at Span 2 facing west (southside). Cracks observed near efflorescence.
	Photo 6: Close up view of diagonal crack at Span 1.

	Photo 7: Close up view of transverse crack at Span 2. Crack width is 0.35 mm.
	Photo 8: View of the diagonal crack extended through the core location.
	Photo 9: Broken tined edges core #6 at the shoulder of eastbound bridge Span 2.

	Photo 10: Close up view of the transverse crack at eastbound bridge Span 1.
	Photo 11: View of the existing drilled cores on Span 3 westbound bridge.

APPENDIX C – Supplement 1

Jonesboro Road Bridge, Atlanta, Georgia
Petrographic Report

PETROGRAPHIC EXAMINATION OF CONCRETE CORES FROM A BRIDGE DECK IN GEORGIA (GA)

A report submitted to the Structures Laboratory at TFHRC

Rongtang Liu
Petrographic Laboratory
FHWA TFHRC
(Reviewed by Richard Meininger, PE; Concrete Laboratory; printed in color 9-12-2006)

July 6, 2006

1. Abstract

Eight concrete cores were received from the Structures Laboratory of the Turner-Fairbank Highway Research Center for petrographic examination. Reportedly, the cores were collected from a concrete bridge deck in Georgia.

Petrographic examination was performed on samples using optical microscopes. Thin sections, as well as polished slabs, were made from the concrete cores for microscopic examination.

Visual inspection of the concrete cores revealed that five of the eight cores (GA#1, GA#2, GA#3, GA#4 and GA#6) had longitudinal cracks. No defects were observed in the other cores. The findings from microscopic examination indicated that the concrete had entrained air voids; the hydration of the cement was reasonable; cracks existed in the paste; and ettringite as secondary deposit was present in some of the air voids.

2. Introduction

The Petrographic Laboratory of the FHWA Turner-Fairbank Highway Research Center was asked by the Structures Laboratory to examine a set of concrete cores retrieved from a bridge deck in Georgia. Six concrete cores, 3-¾-in. in diameter and 3-in. to 3-½-in. in length, were received by the Petrographic Laboratory. The identification on the cores was as follows: GA#1, GA#2, GA#3, GA#4, GA#5, GA#6, GA#7, and GA#8.

3. Laboratory Procedures

Petrographic examination of the concrete samples was performed in accordance with ASTM C 856, "Standard Practice for Petrographic Examination of Hardened Concrete." Sections were polished and examined using a stereomicroscope at magnifications up to 200×. Small rectangular blocks were cut from concrete samples. One surface of each block was polished, dried, placed on petrographic slide with low−viscosity epoxy resin, and reduced to a thickness of approximately 25 micrometers (0.001 in.). These thin sections were examined using a polarized–light microscope at magnification up to 400×, to determine aggregate mineralogy, paste characteristics, and microstructure.

Two ¾-inch thick slabs were sawn from the concrete cores. The slabs were ground and polished to obtain a smooth and plane surface. They were examined using a stereomicroscope at magnifications up to 200×.

4. Findings

Eight (8) thin-section samples taken from the received cores were examined using the polarized light microscope. Two polished sections were examined using the stereomicroscope. The findings are summarized as following:

Aggregates

The coarse aggregate in the concrete is crushed granite. Coarse aggregate particles are mostly angular, and the maximum size is about 1 inch. Preferential orientation of coarse aggregate particles was not observed in this concrete, nor was segregation.

The fine aggregate fraction is mainly composed of quartz. Mica is also present in the concrete as a constituent of the fine aggregate. The source of the fine aggregate is probably natural sand. The majority of the fine aggregate particles are angular, only a small portion is rounded.

Cement Paste

The cement is well hydrated and the paste contains some unhydrated cement particles as seen under the microscope (Figure C1-1). Fly ash particles (as shown in Figure C1-2) are present in the cement matrix.

Air Voids

Small, spherical air voids are observed in the concrete (Figure C1-3), hence the concrete was air entrained. But the air content is relatively low. Entrained air voids are well distributed in the concrete. A small amount of entrapped air voids are also present in the concrete.

Cement-Aggregate Bonding

In general, the cement/aggregate interface is moderately strong. Typical cement/aggregate interface is shown in Figure C1-4.

Secondary Deposit

Occasionally ettringite is observed in some air voids in the concrete, as shown in Figure C1-5.

Cracking

There are visible longitudinal cracks in five of the cores (GA#1, GA#2, GA#3, GA#4 and GA#6). The cause of the cracking is uncertain. It is speculated that shrinkage may be the cause of the cracking. There is no evidence of any material related deterioration in the concrete.

Cracks are also observed under the microscope. Micro-cracking is present in the cement paste, as shown in Figure C1-4 and Figure C1-6. These cracks are probably due to drying shrinkage, although other mechanisms might have also contributed to the distress.

5. Summary

The cement was well hydrated, with some unhydrated cement particles present in the paste. The concrete was air entrained, and the entrained air voids were well distributed in the concrete. In general, the bond between the aggregate and the paste appears moderately strong. Micro-cracks exist in the paste. The cause of the cracking in five of the cores is uncertain. It is speculated that shrinkage, among other mechanisms, may be the major cause of the cracking. There is no evidence of material-related distress in the concrete.

Ettringite was sporadically found in the air voids. There was no evidence of deterioration associated with the existence of the ettringite in the concrete. It is common to see ettringite as secondary deposit in concrete.

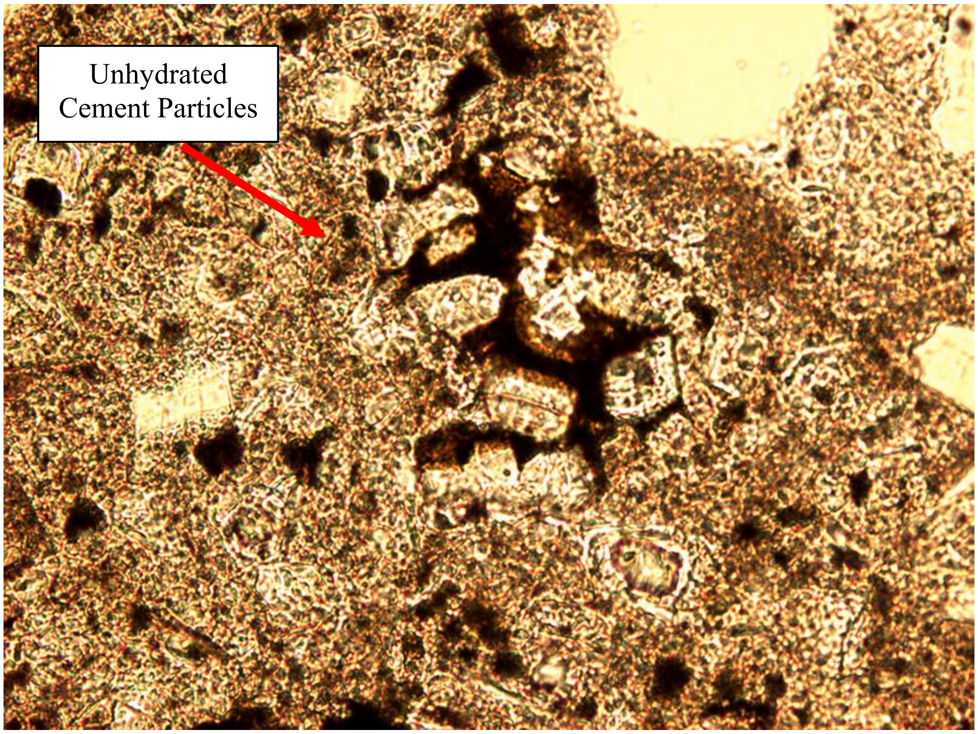


Figure C1-1: Unhydrated cement particles in paste. Width of field is 0.33 mm. Thin-section image.

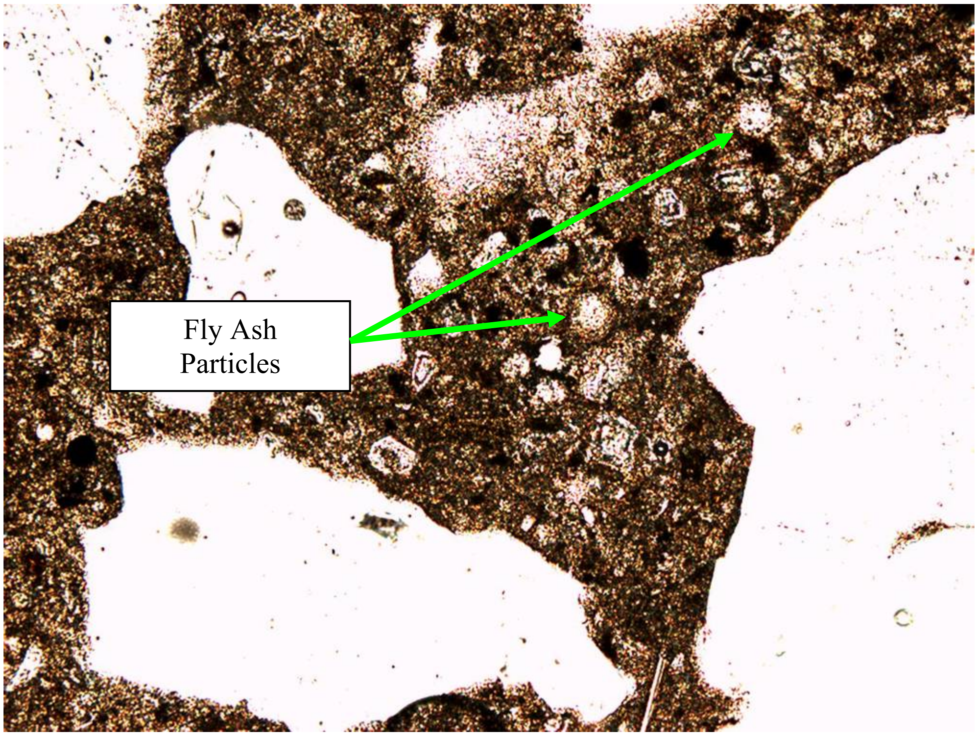


Figure C1-2: Fly ash particles in the paste. Width of field is 0.65 mm. Thin- section image.

Figure C1-3: Entrained air voids in the concrete. Width of field is 6.5 mm. Polished surface image.

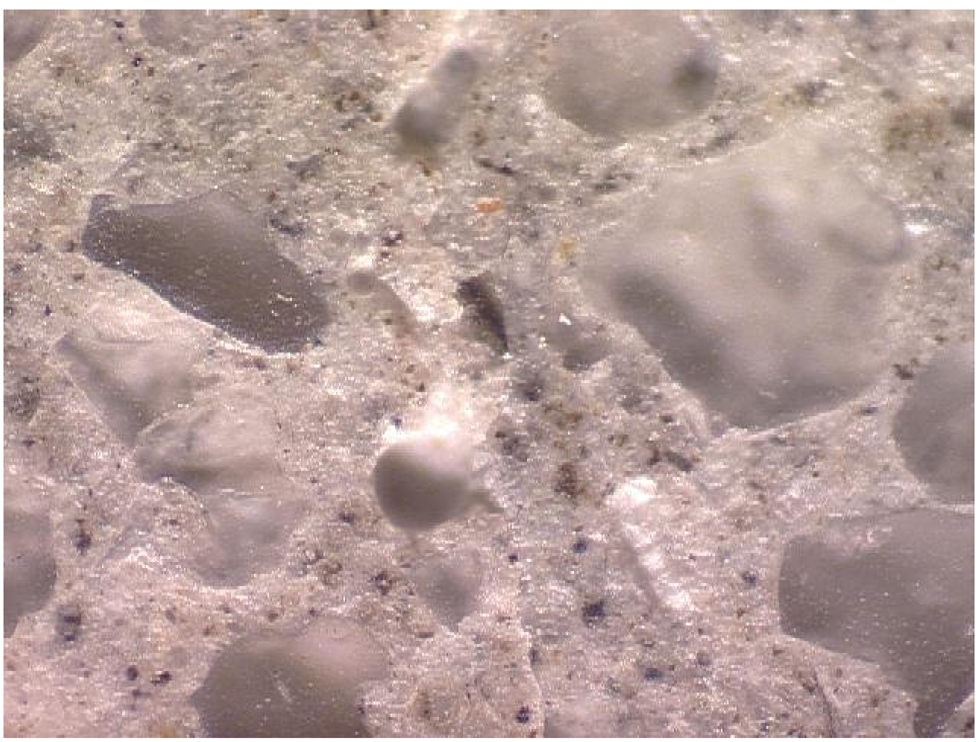


Figure C1-4: The bonding between aggregate and cement paste is moderate. Width of field is 2.0 mm. Polished surface image. Note a crack is at the center of the image.

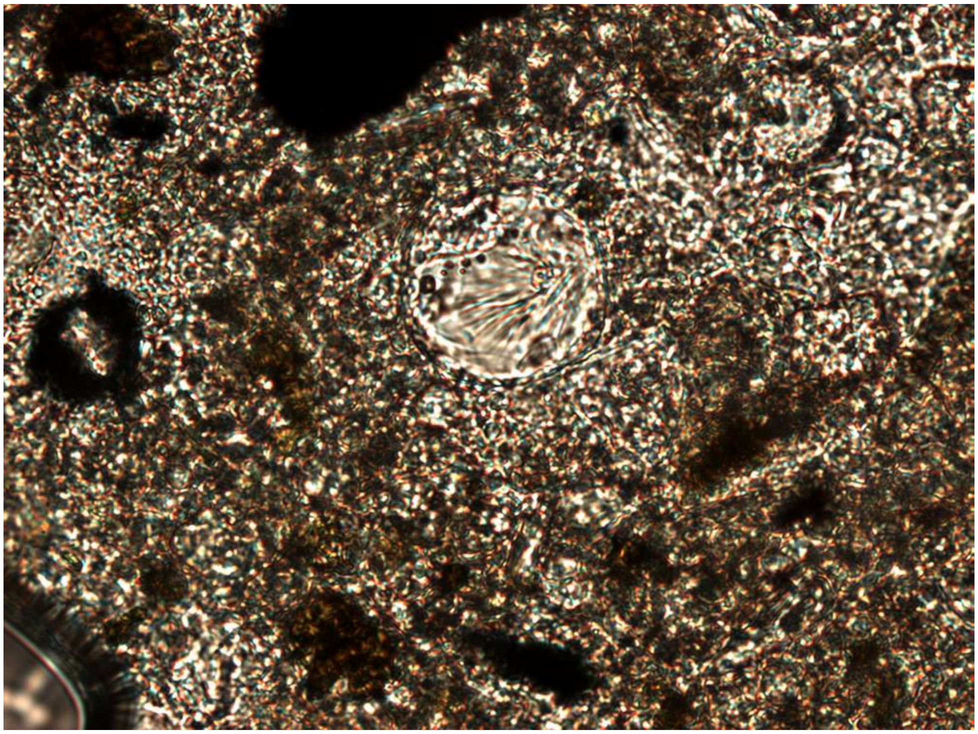


Figure C1-5: Image of ettringite in an air void. Width of field is 0.165 mm. Thin-section image 0.165 mm.

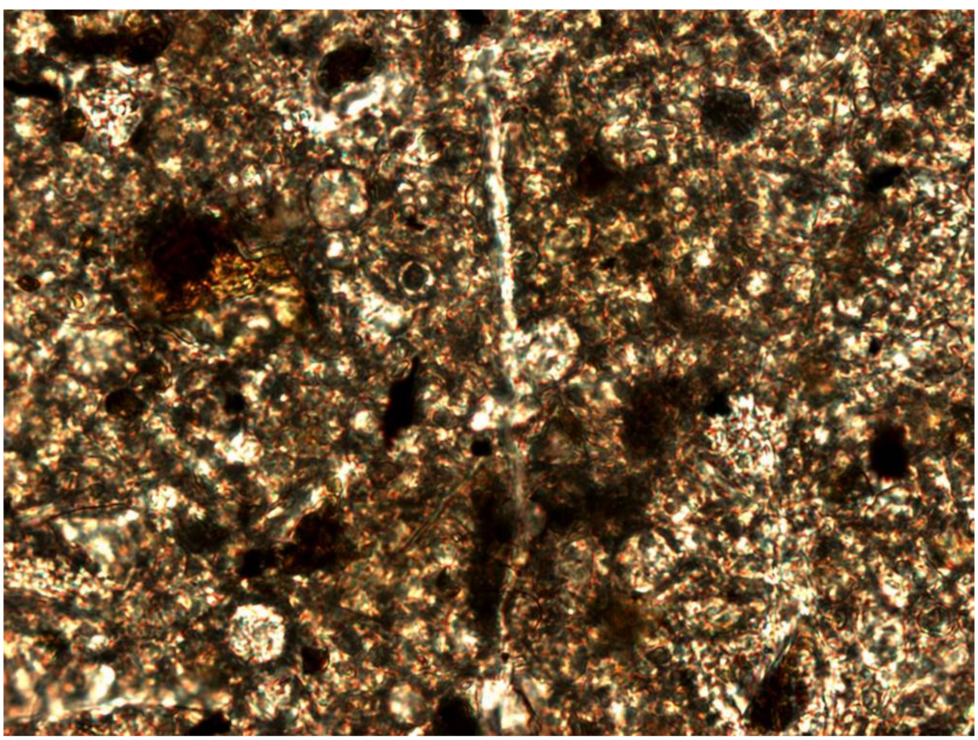


Figure C1-6: Crack in the cement paste. Width of field is 0.165 mm. Thin- section image.

APPENDIX C – Supplement 2

Jonesboro Road Bridge, Atlanta, Georgia
Survey Checklist

Checklist

The following checklist is adapted from 201.1R-92, and provided to facilitate a thorough survey. The definition of terms and associated photographs in 201.1 R-92, are utilized to standardize the reporting of the condition of the concrete in the structures.

1. Description of structure or pavement
   - 1.1 Name, location, type, and size
     The Jonesboro Road Bridge over I-75 on State Route 920, located in Henry County, south of Atlanta is 352-ft (107.5m) long. Clear width of the bridge is 90-ft (27.4-m). It consists of five lanes of traffic with bike lanes and shoulders.
   - 1.2 Owner, project engineer, contractor, when built
     Owner - Georgia Department of Transportation, this bridge is part of a demonstration project for HPC in bridge structures which were co-sponsored by the Federal Highway Administration (FHWA) and the Georgia Department of Transportation (GDOT). The bridge was constructed in 2000 and opened to traffic in February 2002.
   - 1.3 Design
     - 1.3.1 Architect and/or engineer: the Georgia Department of Transportation (GDOT)
     - 1.3.2 Intended use and history of use: To carry high volume of traffic over the State Route 920. Opened to traffic in February 2002.
     - 1.3.4 Special features: Bridge consists of four spans (352-ft in total). AASHTO Type II girders were used for the 53-ft and 45-ft shorter spans in the bridge. AASHTO Type IV girders were used for the 127-ft (38.75-m) long spans. HPC was used in girders and cast-in-place deck panels
   - 1.4 Construction
     - 1.4.1 Contractor-general,
     - 1.4.2 Subcontractors concrete placement: N/A
     - 1.4.3 Concrete supplier:
     - 1.4.4 Agency responsible for testing: N/A
     - 1.4.5 Other subcontractors: N/A
   - 1.5 Photographs
     - 1.5.1 General view Photos 1 through 4
     - 1.5.2 Detailed close up of condition of area Photos 5 through 11
   - 1.7 Sketch map-orientation showing sunny and shady and well and poorly drained regions N/A
   - 1.8
2. Present condition of structure Date of Evaluation The week of March 8, 2004
   - 2.1 Overall alignment of structure No signs of misalignment
     - 2.1.1 Settlement
     - 2.1.2 Deflection
     - 2.1.3 Expansion
     - 2.1.4 Contraction

2.2 Portions showing distress (beams, columns, pavement, walls, etc., subjected to strains and pressures) Efflorescence at the underside of the bridge

2.3 Surface condition of concrete

2.3.1 General (good, satisfactory, poor, dusting, chalking, blisters) Good

2.3.2 Cracks Transverse and longitudinal

2.3.2.1 Location and frequency See Figure 2a and Figure 2b

2.3.2.4 Type and size (see Definitions) See Figure 2a and Figure 2b

Transverse At the beam diaphragm and panel boundaries

Width (from Crack comparator): majority less than 0.03 in.

Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)

Craze N/A

Width (from Crack comparator) ____________

Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)

Map N/A

Width (from Crack comparator) ______________

Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)

D-Cracking N/A

Width (from Crack comparator) ____________

Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)

Diagonal NA

Width (from Crack comparator) NA

Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)

2.3.2.5 Leaching, stalactites N/A

2.3.3 Scaling N/A

2.3.3.1 Area, depth ________________

2.3.3.3 Type (see Definitions) ________________

Light (Less than 1/8 in.)
Medium (1/8 in. – 3/8 in.)
Severe (3/8 in. – 3/4 in.)
Very Severe (Greater than 3/4 in.)

2.3.4 Spalls and popouts Not significant. Cracks may have spalls
2.3.4.1 Number, size, and depth NA
2.3.4.3 Type (see Definitions) NA
Spalls
Small (Less than 3/4 in. depth)
Large (Greater than 3/4 in. depth)
Popouts
Small (Less than 3/8 in. diameter)
Medium (3/8 in. – 2 in. diameter)
Large (Greater than 2 in. diameter)

2.3.5 Extent of corrosion or chemical attack, abrasion, impact, cavitation N/A
2.3.6 Stains, efflorescence Efflorescence at a few locations on the underside of the bridge
2.3.7 Exposed reinforcement none
2.3.8 Curling and warping N/A
2.3.9 Previous patching or other repair N/A
2.3.10 Surface coatings N/A
2.3.10.1 Type and thickness N/A
2.3.10.2 Bond to concrete N/A
2.3.10.3 Condition N/A
2.3.11 Abrasion N/A
2.3.12 Penetrating sealers ___
2.3.12.1 Type N/A
2.3.12.2 Effectiveness N/A
2.3.12.4 Discoloration N/A

2.4 Interior condition of concrete (in situ and samples) N/A
2.4.1 Strength of cores
2.4.2 Density of cores
2.4.3 Moisture content
2.4.4 Evidence of alkali-aggregate or other reactions N/A
2.4.5 Bond to aggregate, reinforcing steel, joints N/A
2.4.6 Pulse velocity
2.4.7 Volume change
2.4.8 Air content and distribution
2.4.9 Chloride-ion content
2.4.10 Cover over reinforcing steel
2.4.11 Half-cell potential to reinforcing steel.
2.4.12 Evidence of reinforcement corrosion
2.4.13 Evidence of corrosion of dissimilar metals
2.4.15 Delaminations N/A
2.4.15.1 Location N/A

2.4.15.2 Number, and size ______ N/A
2.4.15 Depth of carbonation
2.4.16 freezing and thawing distress (frost damage)
2.4.17 Extent of deterioration
2.4.19 Aggregate proportioning, and distribution

3. Nature of loading and detrimental elements
3.1 Exposure
3.1.1 Environment (arid, subtropical, marine, freshwater, industrial, etc.)
N/A
3.1.2 Weather-(July and January mean temperatures, 89ºF and 52ºF
Mean annual rainfall and 4.2
Months in which 60 percent of it occurs) March
3.1.3 Freezing and thawing negligible
3.1.4 Wetting and drying Minimal annual exposure
3.1.6 Drying under dry atmosphere N/A
3.1.6 Chemical attack-sulfates, acids, chloride N/A
3.1.7 Abrasion, erosion, cavitation, impact N/A
3.1.8 Electric currents N/A
3.1.9 Deicing chemicals which contain chloride ions N/A
3.1.10 Heat from adjacent sources N/A
3.2 Drainage N/A
3.2.1 Flashing ______
3.2.2 Weepholes ______
3.2.3 Contour ______
3.2.4 Elevation of drains ______
3.3 Loading Research Test Data Available in Compilation CD Version 3
3.3.1 Dead ______
3.3.2 Live ______
3.3.3 Impact ______
3.3.4 Vibration ______
3.3.5 Traffic index ______
3.3.6 Other ______
3.4 Soils (foundation conditions) ______
3.4.1 Compressibility
3.4.2 Expansive soil
3.4.3 Settlement
3.4.4 Resistivity
3.4.5 Evidence of pumping
3.4.6 Water table (level and fluctuations)

4. Original condition of structure Good
4.1 Condition of formed and finished surfaces Good
4.1.1 Smoothness ______
4.1.2 Air pockets ("bugholes") ______
4.1.3 Sand streaks ______

4.1.4 Honeycomb		
4.1.5 Soft areas (retarded hydration)		
4.1.6 Cold joints		
4.1.9 Staining		
4.1.10 Sand pockets		
4.2 Defects		N/A
4.2.1 Cracking		
4.2.1.1	Plastic shrinkage	
4.2.1.2	Thermal shrinkage	
4.2.1.3	Drying shrinkage	
4.2.3 Curling		

5. Materials of Construction — See Table 2

6. Construction Practices — See Report pg. 3 and 6

APPENDIX D

Charenton Canal Bridge, Charenton, Louisiana

HIGH PERFORMANCE CONCRETE BRIDGE DECK INVESTIGATION

Charenton Canal Bridge
Charenton, Louisiana

I. BACKGROUND

The Charenton Canal Bridge on LA 87 in St. Mary Parish is the first High Performance Concrete (HPC) Bridge built in Louisiana. HPC was used in all structural components. The bridge is 365-ft (111-m) long and it replaces a 55-year-old cast-in-place concrete bridge. Clear width of the bridge is 46.5-ft (14.2-m). It consists of two 12-ft (3.66-m) lanes, one 12-ft (3.66-m) shoulder on the westbound bridge and one 8-ft (2.44-m) shoulder on the eastbound bridge. The Charenton Canal Bridge opened to traffic on November 4, 1999.

The Charenton Canal Bridge has five spans (73-ft (22.3-m) long on average). Each span consists of five Type III AASHTO girders made of precast, prestressed HPC. The girders are evenly spaced at 10-ft (3.1-m) centers and support the cast-in-place concrete deck. The substructure of the bridge consists of cast-in-place concrete bent caps supported on 24 and 30-in. (610 and 762-mm) square precast, prestressed concrete piles. The use of HPC enabled the bridge to be designed with one less line of girders than would be required if regular 6000 psi (41 MPa) concrete was used.

The decks of the Charenton Canal Bridge are 8-in. (203-mm) thick cast-in-place reinforced concrete. The main reinforcement perpendicular to the supporting prestressed concrete girders consists of truss bars measuring ¾-in. (19-mm) and top and bottom straight bars measuring ½-in. (13-mm) in diameter. Longitudinal deck reinforcing steel included ½-in. (13-mm) diameter top and bottom bars. Negative moment continuity for live loads over the piers was provided by the longitudinal reinforcing steel in the deck. No reinforcement was provided to resist a positive moment over the piers. Diaphragms were provided at the end bents, the piers and the mid-spans.

The Charenton Canal Bridge is part of a demonstration project for HPC in bridge structures which was sponsored by the Louisiana Department of Transportation (LDOT). It is anticipated that a 75 to 100-year service life instead of the normal 50-year service life will be achieved. As part of the project, material testing, bridge instrumentation, and bridge monitoring were performed by Tulane University in cooperation with the Louisiana Transportation Research Center. It is evident that the structures are intended to be compared for relative durability and performance based on the extensive use of HPC. Completion of the Charenton Canal Bridge proves that it is feasible to construct an HPC bridge in Louisiana with local materials and local contractors.

II. SCOPE OF SERVICES

Professional Service Industries (PSI) is under contract with the Federal Highway Administration (FHWA) to conduct bridge deck inspections for HPC bridges. Our scope of services on this bridge included a series of tasks and sub-tasks, which are described as follows:

- Collect all available information relevant to the construction of each bridge, including;
    - Deck Concrete Properties
    - Specified Deck Concrete Construction Procedures
    - Approved Concrete Mix Proportions
    - Measured Properties from QC
    - Other Measured Properties
    - Actual Method of Deck Placement
    - Average Daily Traffic (ADT) and Average Daily Truck Traffic (ADTT)
    - Exposure Condition of the Bridge
    - Any Performed Maintenance
    - Any Inspection Reports
- Visually inspect the bridge, and obtain the following:
    - General condition of the deck top surface
    - Determination of the maximum crack width
    - General condition of the deck underside
    - General condition of the girders
    - Photograph areas of significant deterioration
    - Prepare drawings locating defects
    - Extract 6 concrete core samples

III. COMPILATION OF BRIDGE CONSTRUCTION INFORMATION

Information sources used for this report include bridge drawings, inspection reports, bridge summary data sheets, and technical information contained in FHWA's "Compilation and Evaluation of Results from High Performance Concrete Bridge Projects" version 3.

Deck Concrete Properties

It is noted that three classes of HPC were used in the Charenton Canal Bridge. They are class P (HPC) for concrete precast bridge members, class AA (HPC) for cast-in-place superstructure, and class A (HPC) for cast-in-place substructure. However, deck concrete (class AA HPC) properties are the focus of our report. The bridge deck concrete had a specified compressive strength of 4200 psi (29 MPa) at 28 days. A rapid chloride permeability of 2000 coulombs or less at 56 days was specified for concrete used in all members. Table 1 lists the specified properties of class AA HPC used in cast-in-place bridge deck.

TABLE 1: Specified Concrete Properties

Property	Deck Class AA (HPC)
Minimum Cementitious Materials Content:	658 lb/yd^3 *1
Max. Water/Cementitious Materials Ratio:	0.40
Min. Percentage of Fly Ash:	--
Max. Percentage of Fly Ash:	30
Min. Percentage of Silica Fume:	--
Max. Percentage of Silica Fume:	10
Min. Percentage of GGBFS:	--
Max. Percentage of GGBFS:	50
Slump:	2-8 in.
Air Content:	5.5±1.5%
Compressive Strength - Design:	4200 psi @ 28 days *2
Chloride Permeability (AASHTO T 277):	2000 coulombs at 56 days *2

NOTES:
(1) Contractor was later allowed to use 611 lb/yd^3 cementitious materials.
(2) Standard curing until test age.

Specified Deck Concrete Construction Procedures

For Class AA (HPC) concrete used in the cast-in-place bridge deck, the contractor was required to comply with ACI 302—Guide for Concrete Floor and Slab Construction, ACI 308—Standard Practice for Curing Concrete, and ACI 305—Hot Weather Concreting. When silica fume was used, silica fume was required to be added as early as possible in the concrete batching, and continuous fogging above the surface of the concrete during the finishing operation was specified. Fogging continued until the surface would support wet burlap without deformation. Free standing water on the concrete surface prior to concrete final set was not allowed to occur. The concrete was required to be kept wet with a fog nozzle system or soaker hoses for seven curing days and until a concrete compressive strength of 3,200 psi was achieved. The contractor was required to strictly adhere to the manufacture's written recommendations regarding the use of admixtures, including storage, transportation and method of mixing.

To establish adequacy of curing methods and to determine whether concrete had attained the required compressive strength, a minimum of eight test cylinders were fabricated from the last batch of concrete and match cured under the same conditions as the corresponding member. Three cylinders were to be tested no later than 56 calendar days after casting to determine that the required strength had been achieved. The remaining five cylinders were to be tested at a later time as required by the contractor.

Approved Concrete Mix Proportions

Deck

The approved proportions for Class AA HPC used in the cast-in-place deck are shown in Table 2.

TABLE 2: Approved Mix Proportions for Cast-in-Place Deck

Mix Parameters	Deck (Class AA HPC)
Cement Brand:	Lone Star Industries Aucem
Cement Type:	IS
Cement Quantity:	306 lb/yd^3
GGBFS Brand:	Lone Star Industries Aucem
GGBFS Quality:	305 lb/yd^3
Fly Ash Brand:	--
Fly Ash Type:	--
Fly Ash Quality:	--
Fine Aggregate Type:	Nature Sand
Fine Aggregate FM:	2.32
Fine Aggregate SG:	Not available
Fine Aggregate Quantity:	1176 lb/yd^3
Coarse Aggregate, Max. Size:	1-in.
Coarse Aggregate Type:	No. 5 Crushed Limestone
Coarse Aggregate Quantity:	1900 lb/yd^3
Water:	238 lb/yd^3
Water Reducer Brand:	--
Water Reducer Type:	A and D
Water Reducer Quantity:	36.7 fl oz/yd^3
High Range Water Reducer Brand:	--
High Range Water Reducer Type:	--
High Range Water Reducer Quantity:	--
Retarder Brand:	Monex LR
Retarder Type:	A and D
Retarder Quantity:	36.7 fl oz/yd^3
Air Entrainment Brand:	Monex Air 31
Air Entrainment Type:	Salt of Benzyl Sulfonate
Air Entrainment Quantity	4.0 fl oz/yd^3
Water/Cementitious Materials Ratio:	0.39

NOTES: GGBFS is pre-blended by cement supplier.

Measured properties of approved concrete mixture for the cast-in-place deck are summarized in Table 3.

TABLE 3: Measured Properties of Approved Concrete Mixtures for Cast-in-Place Deck

Measured Concrete Properties	Class AA (HPC) Cast-in-Place Deck
Slump:	4- in.
Air Content:	5%
Compressive strength:	5680 psi at 28 days
Rapid Chloride Permeability (AASHTO T 277):	1019 coulombs at 56 days

The properties of the cement used in the cast-in-place deck are shown in Table 4.

TABLE 4: Properties of the Cement used in the Cast-in-Place Deck

Component	Weight %
SiO_2	27.98
Al_2O_3	7.46
Fe_2O_3	2.08
CaO	52.03
MgO	5.24
SO_3	2.70
Na_2O	0.14
K_2O	0.42
TiO_2	0.38
P_2O_5	0.04
Mn_2O_3	0.15
SrO	0.07
L.O.I. (950 ^{o}C)	0.46
Total	99.13
Alkalies as Na_2O Insoluble Residue	0.42
Blaine Fineness (m^2/kg)	---

Measured Properties from QC Tests of Production Concrete

Cast-in-Place Deck

The measured properties from QC tests of Class AA HPC production concrete used in the cast-in-place deck are shown in Table 5.

TABLE 5: Measured Properties from QC Tests of Class AA (HPC) Deck Concrete

Span	Compressive Strength (psi)	
	28 days	58 days
1	5349	5406
2 and 3	5537	5765
4 and 5	5592	6185
Average	5493	5785

NOTES:

(1) Concrete received a 4-6 hours fog spraying followed by wet burlap curing for 7 days.

(2) Test cylinders are 6 ×12-in., received ASTM C31 standard curing.

Measured Properties from Research Tests of Production Concrete

Cast-in-Place Deck

The compressive strength, modulus of elasticity, and coefficient of thermal expansion of Class AA HPC production concrete used in the bridge deck are shown in Table 6.

TABLE 6: Measured Properties from Research Tests of Production Concrete Used in the Cast-in-Place Deck

Properties	Concrete Age		
	7 days	28 days	90 days
Compressive Strength, psi			
Field	3086	4455	4861
	3275	4462	4896
	3086	4258	5142
Average	3149	4392	4966
Modulus of Elasticity (ASTM C 469), ksi			
Field	3514	4042	4424
	3450	4278	4342
	3579	4162	4526
Average	3514	4161	4431
Coefficient of Thermal Expansion (CRD C-39), millionths/^{o}F			
Field	3.6	6.5	5.2
	2.4	5.9	4.4
	3.4	4.5	5.3
Average	3.1	5.6	5.0
Grand Average			4.6

NOTES:
(1) All specimens were cured on site in molds for seven days and in air on site thereafter.
(2) All specimens were 6 × 12-in. cylinders.
(3) All specimens were taken from production concrete used in Span 3.

The chloride permeability data for Class AA HPC production concrete used in the cast-in-place deck is shown in Table 7.

TABLE 7: Chloride Permeability of Production Concrete Used in the Deck

	Concrete Age		
	28 days*1	56 days*1	56 days*2
Span No. 1	1348	1075	824
	1361	1469	---
	1328	---	---
	1269	---	---
Average	1327	1272	824
Span No. 2	1917	1394	1037
	2428	1269	1123
	2118	---	---
	2297	---	---
Average	2190	1332	1080
Span No. 3	1347	1867	1061
	1467	1754	1159
	1766	---	---
	1539	---	---
Average	1530	1811	1110
Span No. 4	1548	1155	876
	1653	1479	---
	1420	---	---
	1474	---	---
Average	1524	1317	876
Span No. 5	1705	1568	795
	2284	884	851
	2140	---	---
	1598	---	---
Average	1932	1226	823
Overall Average	1700	1390	965
Northbound Slab	---	---	939
Approach Slab	---	---	677
Average	---	---	808
Southbound Slab	---	---	909
Approach Slab	---	---	793
Average	---	---	851

NOTES:

(1) All chloride permeability test specimens were cut from 4 × 8-in. cylinders cured on site in the molds for seven days and in air thereafter.

(2) All specimens were 4 × 8-in. cores.

Actual Method of Deck Placement

Construction of the deck occurred in the September of 1999. For the production of HPC in ready-mix plant, all other batching operations were suspended during the HPC batching process to eliminate the possibility of any additional moisture. Those procedures became the standard approach for producing HPC. It is believed that the HPC mix proportion having a low water-cementitious material ratio would be greatly affected by the slightest addition of water. Providing additional moisture control devices at the batch plant proved extremely valuable. The concrete deck was cast in two placements. In the first placement, the decks of Spans 4 and 5 were cast with a transverse construction joint placed in Span 4 at 6 ft from the Span 3 end. In the second placement, the remaining 6 ft of deck in Span 4 was cast together with the decks of Spans 1 through 3.

The concrete was internally vibrated to provide proper consolidation and avoid internal segregation. Fogging of the concrete deck started when the concrete was in the plastic state. This procedure avoided the surface moisture evaporation and plastic shrinkage cracks. This construction practice is particularly important for HPC.

Concrete was distributed by a mechanical spreader. A final troweled finish was applied followed by the tining for enhanced skidding resistance. The deck was cured using wet burlap for 7 days. Wet burlaps were kept moist.

Average Daily Traffic (ADT) and Average Daily Truck Traffic (ADTT)

Average daily traffic for both eastbound and westbound lanes was calculated based on a count of all vehicles crossing the bridge during a 15 minute period beginning at 0830 hrs on February 19, 2004. These vehicle counts gave at an ADT of 1920 and an ADTT of 96. The estimation of traffic flow was made while the PSI inspection crew was on-site.

Exposure Condition of the Bridge

The area surrounding the bridge is developed with mixed residential and commercial land use. The bridge crosses a river that supports navigational traffic and crosses a flood-control channel. The bridge is located in a coastal region that experiences appreciable amounts of hurricane activity.

The National Weather Service reports that the normal maximum temperature varies between 91°F in July and 60°F in January. The normal minimum temperature varies between 74°F in July and 42°F in January. The normal precipitation varies between 6.6-in. per month in August to 2.8-in. per month in October. Very few days per year does the temperature drop below 32°F. Based on this information, the bridge has minimal annual exposure to freeze/thaw cycles.

Performed Maintenance

No documents were found which would indicate any maintenance had been performed since the bridge was constructed in 1997.

Inspection Reports

As part of the project, bridge instrumentation and bridge monitoring are being performed by Tulane University in cooperation with the Louisiana Transportation Research Center. A number of 4 in. cores were drilled from previous bridge inspection, as is shown in

Figure 2a and Figure 2b. The researchers have developed an instrumentation program to monitor the structural performance of the bridge and its components as described in "Implementation of High Performance Concrete in Louisiana Bridges - Interim Report."

IV. BRIDGE DECK INSPECTION

PSI personnel performed a visual inspection of the bridge decks during the week of February 16, 2004. The results of the inspection are summarized as follows.

General Condition of the Deck Top Surface

Figure 1 shows the general layout of the decks for the Charenton Canal Bridge. Results of visual inspection of the decks are shown in Figure 2a and Figure 2b. Surface defects observed and documented during visual inspection primarily included transverse cracks (see photos 9 through 11). Other defects observed and documented included small spalls at joints and cracks, abrasion, and broken tinned edges (see photo 10). However, apparent signs of other serious damages such as freeze-thaw, D-cracking, alkali-silica reaction, and alkali-aggregate reaction were not observed. Efflorescence can be seen on the concrete barrier wall along the bridge. In addition, drilled cores (3-in. diameter) resulted from previous investigation by others, were also observed (see Figure 2a and Figure 2b).

A total of 46 cracks were recorded during visual survey of the bridge decks (see Figure 2). Of the 46 cracks, 33 cracks were on the eastbound bridge and 13 cracks were on the westbound bridge. The sum of crack lengths was 187.4 ft over a bridge deck area of 16,060 ft^2. The crack densities (crack length per deck area) range from 0 to 0.030 ft/ft^2 for the 10 spans (including both eastbound and westbound lanes) investigated. Crack density (total crack length / deck area) for the eastbound and westbound bridges combined was calculated to be 0.012 ft/ft^2.

All cracks recorded were classified as hairline cracks, with widths less than 0.031 in. It can be noted that cracks were typically limited at span ends. Small surface spalls, which either occurred due to breaking of tined edges or the crack edges, were observed. Figure 2a and Figure 2b also illustrates the locations of drilled cores, which resulted from previous investigation by others. The number, length and density of cracks for each structure are shown in Table 8 and Table 9.

TABLE 8: Measured Transverse Cracks on the Surface of Eastbound Bridge Decks

Eastbound Traverse Cracks	Count	Length Range (feet)	Mean Length of Cracks (feet)	Median Length of Cracks (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span 1	12	0.5 to 4.0	2.8	3.2	33.1	1606	0.021
Span 2	6	0.6 to 2.3	1.3	1.1	7.8	1606	0.005
Span 3	5	0.9 to 5.7	2.9	3	14.6	1606	0.009
Span 4	10	0.5 to 17.3	4.8	3.3	48.2	1606	0.030
Span 5	0	0	0	0	0	1606	0

NOTES: Transverse cracks include cracks oriented parallel to skewed joints

TABLE 9: Measured Transverse Cracks on the Surface of Westbound Bridge Decks

Westbound Transverse Cracks	Count	Length Range (feet)	Mean Length of Cracks (feet)	Median Length of Cracks (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span 1	7	0.7 to 9.0	2.6	3.3	31.4	1606	0.020
Span 2	1	2.3 to 2.3	2.3	2.3	2.3	1606	0.001
Span 3	1	5.5 to 5.5	5.5	5.5	5.5	1606	0.003
Span 4	1	16.4 to 16.4	16.4	16.4	16.4	1606	0.010
Span 5	3	4.2 to 15.6	9.4	9.4	28.1	1606	0.017

NOTES: Transverse cracks include cracks oriented parallel to the skewed joints

Maximum Crack Width

The maximum width of transverse cracks was measured to be 0.016 in. According to ACI 201, these crack widths are classified as hairline cracks.

General Condition of the Deck Underside

The underside of the deck exhibits no signs of distress. At very limited locations, efflorescence was observed. Photos 5 through 7 show a general view of the underside of the deck.

General Condition of the Girders

The girders were inspected from a motor boat, without the aide of any access equipment. No signs of distress were noted on any of the girders (see Photo 5).

Concrete Core Samples

Six cores, approximately 3.5-inches long and 4 inches in diameter, were retrieved from the decks. The core sample locations are shown on Figure 2a and Figure 2b. The locations were evenly distributed along each shoulder of the bridge. The cores were labeled LA-1 through LA-6 and were transferred to FHWA for further analysis.

TABLE 10: Core Dimensions

Sample	LAS-1	LAS-2	LAS-3	LAS-4	LAS-5	LAS-6
Diameter (in.)	3¾	3¾	3¾	3¾	3¾	3¾
Length (in.)	3¾	3¾	4	2½	3½	3¾

Preliminary Conclusions

The construction of the Charenton Canal Bridge is part of a demonstration project for HPC in bridge structures. It was completed in 1999. Researchers from Tulane University in cooperation with the Louisiana Transportation Research Center performed material testing, bridge instrumentation, and bridge monitoring throughout this project.

The visual inspection of the bridge decks as part of our study was performed about four years after the bridge opened to traffic. The eastbound and westbound bridges are exhibiting comparable magnitude and pattern of cracking. A total of 46 transverse cracks were recorded on the bridge with a combined total crack length of 187.4 ft over a bridge deck area of 16,060 ft^2. However, all these cracks were hairline cracks with width less than 0.016 in. No major distresses were observed in our bridge survey.

Though the number of transverse crack counts for eastbound (33 cracks) is more than that for the westbound bridge (13 cracks), the crack densities on eastbound and westbound bridges appear to be similar (i.e., 0.006 ft/ft^2 for the eastbound bridge and 0.005 ft/ft^2 for the westbound bridge).

Compared to other spans in the bridge, crack count in Span 1 is greater on both eastbound and westbound bridges. A higher crack density is calculated. Span 1 ends along the skew. Some of these cracks were exhibiting spalling due to breaking of the edges. The development and widening of these cracks may be attributed to the layout of the cast-in-place deck at the span ends. In addition, the structural system of the Charenton Canal Bridge is flexible compared to conventional bridges considering the wide beam spacing and large span. This relatively flexible structural system might have contributed to the development and widening of some cracks.

It is noted that relatively large numbers of short-length transverse cracks were observed in Span 4 eastbound lanes and Span 5 westbound lanes. Settlement of foundation supporting the eastern end of the westbound bridge piers, as evidenced by pictures 12 and 13, may contribute to the observed transverse cracks. A homeowner nearby actually approached our bridge inspection personals and informed us his observation regarding the settlement and cracking at the western end of the bridge.

In general, the work on the Charenton Canal Bridge shows that HPC designs provide significantly higher strength that can lead to more efficient designs requiring fewer piers and, more important, an improved durability. The HPC bridge components have a 56-day permeability of 1079 coulombs in accordance with the mix design. Its ability to resist chlorides and protect steel reinforcement from corrosion will reduce maintenance costs during the life span. A 75- to 100-year service life instead of the normal 50-year service life is anticipated.

Petrographic examination was performed on six cores that were retrieved from the bridge deck. All of the cores showed evidence of being broken off at about three inches in length. Four of the cores exhibited longitudinal cracks, while the remaining two cores showed no visible defects or signs of deterioration.

The coarse aggregate was a mostly angular crushed limestone with a maximum size of about one inch. Preferential orientation of aggregate particles was not observed. The fine aggregate was natural sand with round to angular particles. The fine aggregate was predominately composed of quartz, with a very small amount of feldspar.

The cement paste was reasonably hydrated with some unhydrated particles. Ground granulated blast furnace slag (GGBFS) particles were present in the concrete. Hence, the concrete mixture contained GGBFS as a supplementary cementitious material.

The concrete was air entrained. Small, spherical air voids were observed in the concrete. The entrained air voids were well distributed and the air content was at a normal level. There was a small amount of entrapped air observed in the concrete.

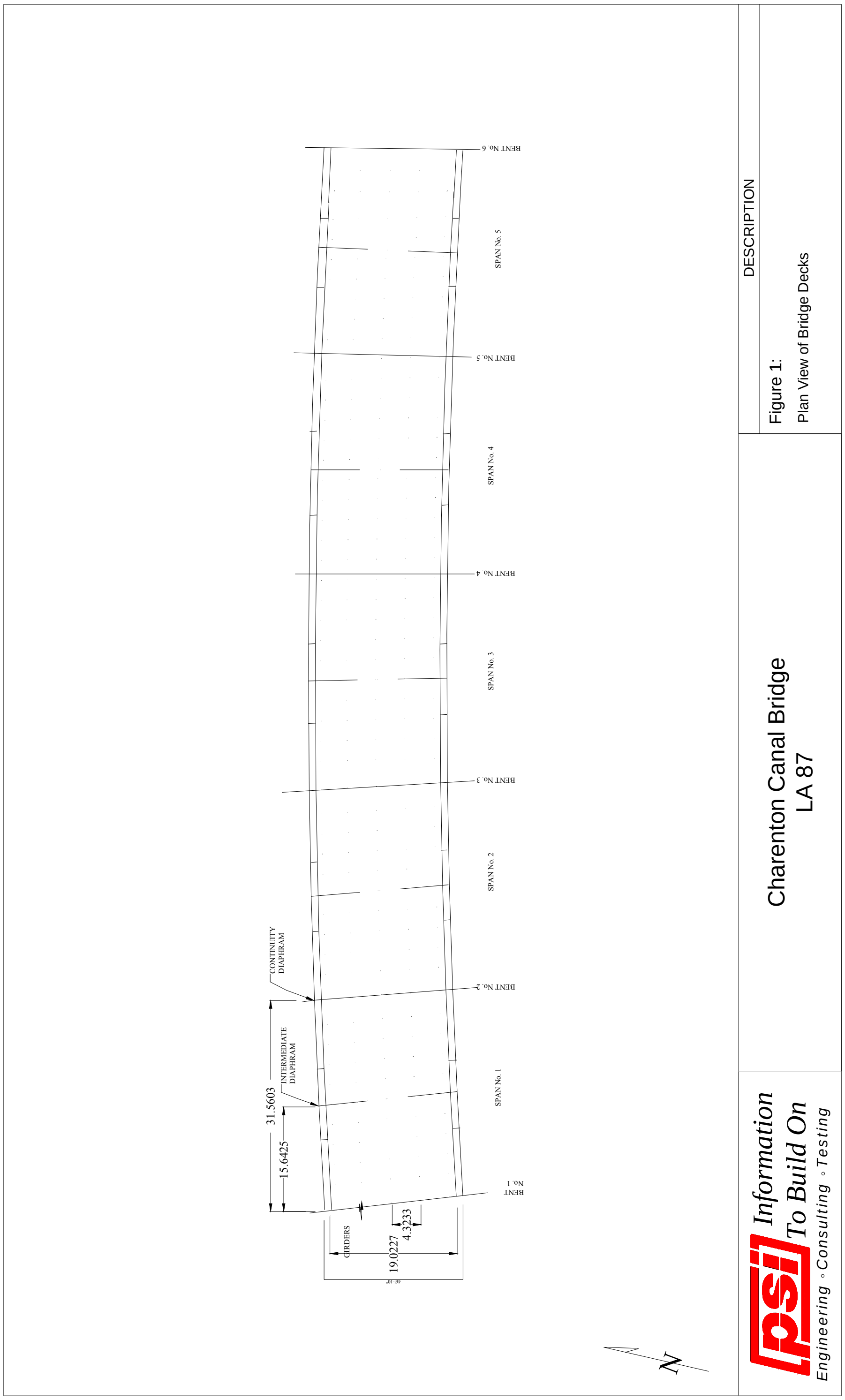
Isolated micro-cracks were sporadically observed in the cement paste. Cracking at the aggregate paste interface was observed. Cracks partially surrounding the fine aggregate particles were also observed in the concrete.

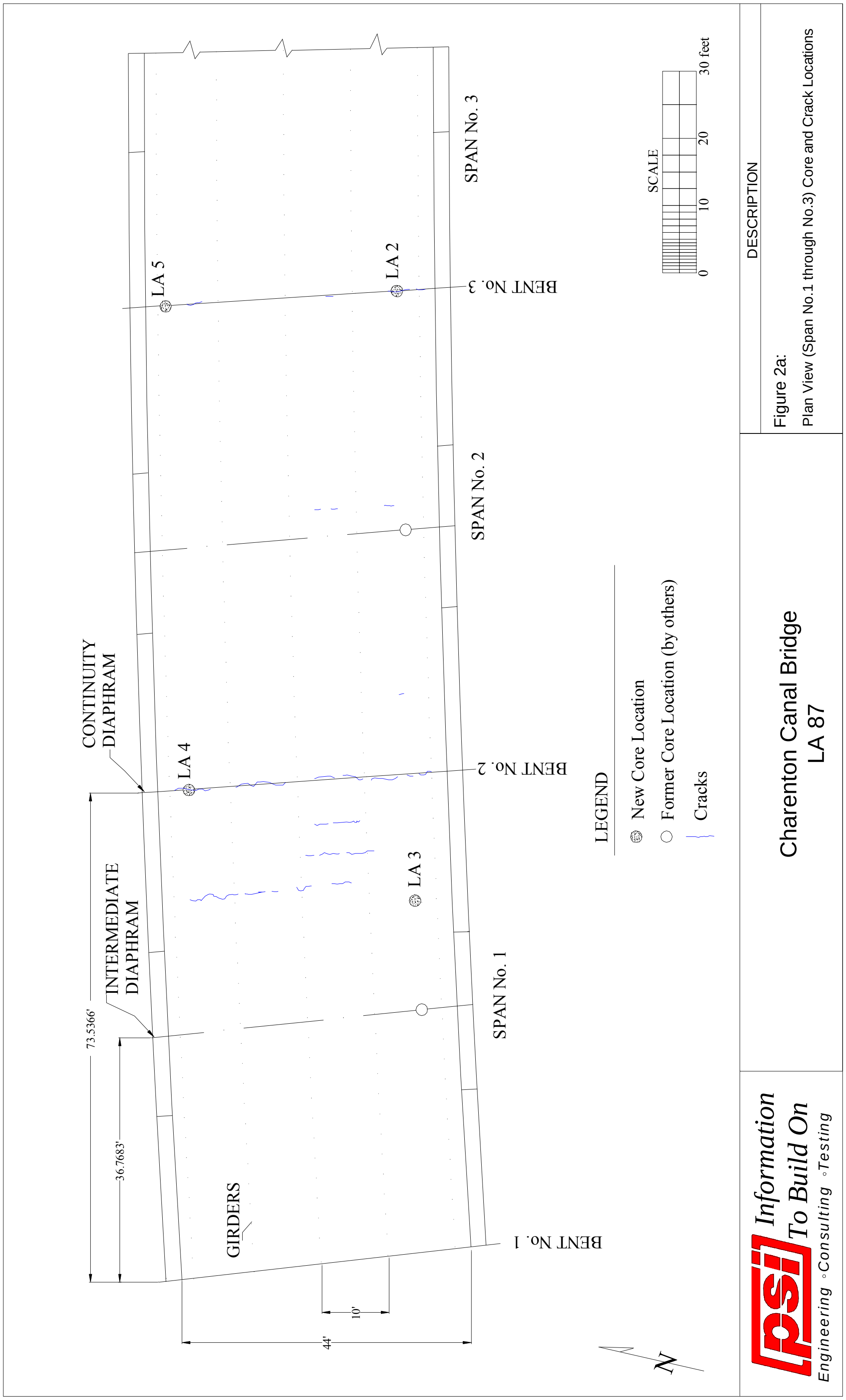
There were no secondary deposits observed in the concrete.

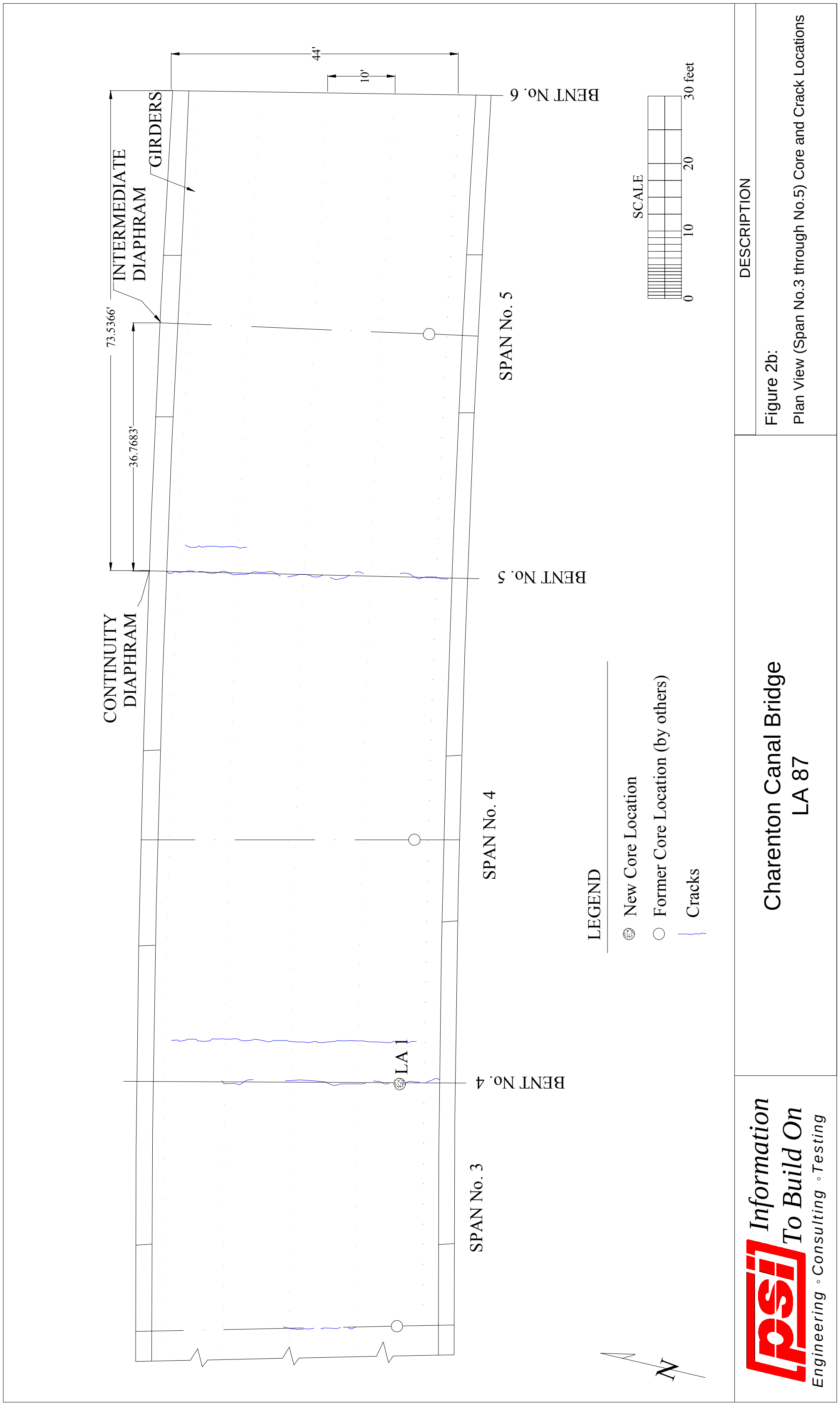
When observed under the microscope, the interface between the aggregate and the cement paste was very porous. This suggested a rather weak bond between the aggregate and the paste. Cracking was also observed at the interface between the fine aggregate particles and the cement paste.

It is speculated that shrinkage may have caused the cracking. A weak bond between the aggregate and cement paste is prone to the formation of interfacial cracking when the concrete experiences shrinkage.

Respectfully Submitted,
Professional Service Industries, Inc.
Structural Investigation & Petrography Department


BENT No. 6
SPAN No. 5
BENT No. 5
SPAN No. 4
BENT No. 4
SPAN No. 3
BENT No. 3
SPAN No. 2
BENT No. 2
SPAN No. 1
BENT No. 1
CONTINUITY DIAPHRAM
INTERMEDIATE DIAPHRAM
31.5603
15.6425
GIRDERS
19.0227
4.3233
N
DESCRIPTION
Figure 1:
Plan View of Bridge Decks
Charenton Canal Bridge
LA 87
psi
Information To Build On
Engineering • Consulting • Testing


SPAN No. 3
SPAN No. 2
SPAN No. 1
BENT No. 3
BENT No. 2
BENT No. 1
LA 5
LA 2
LA 4
LA 3
CONTINUITY DIAPHRAM
INTERMEDIATE DIAPHRAM
GIRDERS
73.5366'
36.7683'
10'
44'
N
SCALE
0
10
20
30 feet
LEGEND
New Core Location
Former Core Location (by others)
Cracks
DESCRIPTION
Figure 2a:
Plan View (Span No.1 through No.3) Core and Crack Locations
Charenton Canal Bridge
LA 87
psi
Information To Build On
Engineering • Consulting • Testing


44'
10'
BENT No. 6
INTERMEDIATE DIAPHRAM
GIRDERS
73.5366'
36.7683'
SPAN No. 5
SCALE
0
10
20
30 feet
DESCRIPTION
Figure 2b:
Plan View (Span No.3 through No.5) Core and Crack Locations
BENT No. 5
CONTINUITY DIAPHRAM
LEGEND
New Core Location
Former Core Location (by others)
Cracks
Charenton Canal Bridge
LA 87
SPAN No. 4
LA 1
BENT No. 4
SPAN No. 3
N
psi
Information To Build On
Engineering • Consulting • Testing

Photographic Documentation

	Photo 1: View looking northwest. The westbound bridge is in foreground.
	Photo 2: View looking east from the western end of the westbound bridge.
	Photo 3: View looking west from the eastern end of the westbound bridge.

	Photo 4: View looking northeast at the western end of the eastbound bridge.
	Photo 5: View of the underside of bridge, the pier in foreground supports Span 4.
	Photo 6: General condition of the girders at Span 2 facing west (southside).

	Photo 7: White precipitants along the overhang on the eastbound bridge.
	Photo 8: View of the concrete barrier at westbound bridge showing cracking and efflorescence.
	Photo 9: View of the transverse cracks at eastbound bridge Span 1 showing the transverse cracking pattern.

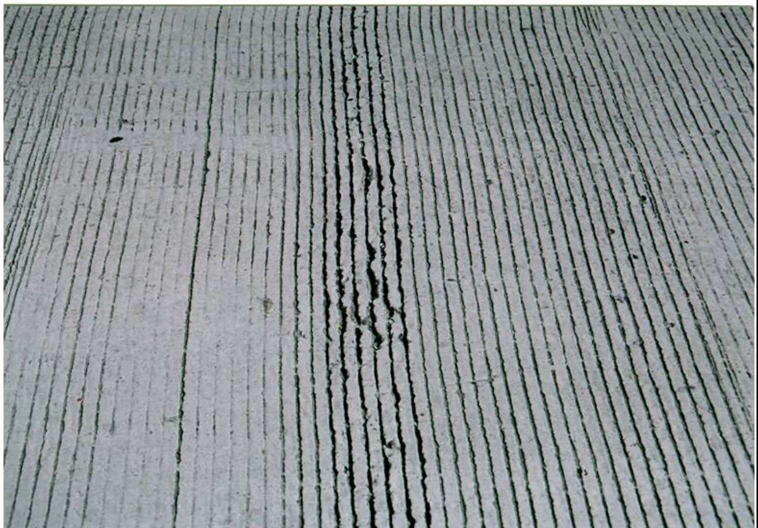
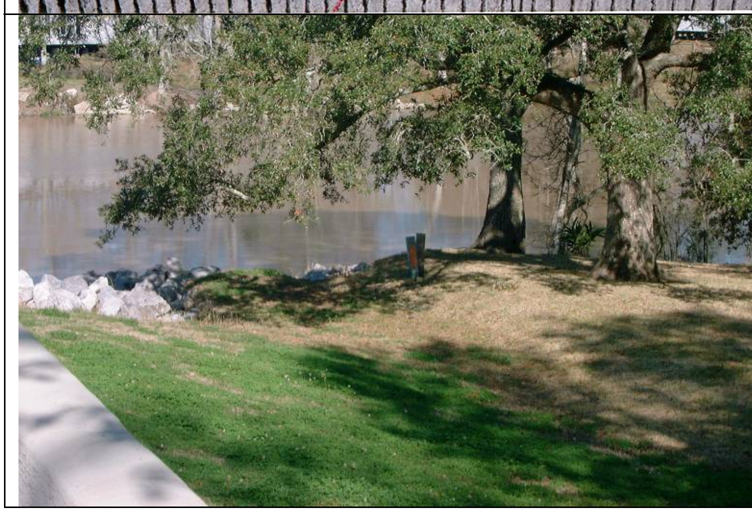
	Photo 10: Broken tined edges at eastbound bridge Span 3.
	Photo 11: Close up view of the transverse crack at eastbound bridge Span 1.
	Photo 12: Close up view of the settlement at the eastern end of westbound bridge Span 5.

	Photo 13: Close up view of the settlement related cracking at the eastern end of bridge Span 5.

APPENDIX D – Supplement 1

Charenton Canal Bridge, Charenton, Louisiana
Petrographic Examination

PETROGRAPHIC EXAMINATION OF CONCRETE CORES FROM A BRIDGE IN LOUISIANA (LA)

A report submitted to the Structures Laboratory at TFHRC

Rongtang Liu
Petrographic Laboratory
FHWA TFHRC
(Reviewed by Richard Meininger, PE; Concrete Laboratory; printed in color 9-12-2006)

August 9, 2006

Introduction

Six concrete cores were received from the Structures Laboratory of the Turner-Fairbank Highway Research Center for petrographic examination. The cores were collected from a concrete bridge in Louisiana. The identification on the cores was as following: LA-1, LA-2, LA-3, LA-4, LA-5, and LA-6 (Figure D1-1).

All of the cores showed evidence of being broken off, and not being drilled all the way through. The dimensions of the cores are as follows:

Core ID	Diameter (in.)	Length (in.)
LA-1	3.75	3
LA-2	3.75	3
LA-3	3.75	3 1/2
LA-4	3.75	2 1/2
LA-5	3.75	3
LA-6	3.75	3

Thin sections, as well as polished slabs, were made from the concrete cores for microscopic examination. Petrographic examination was performed on these samples using optical microscopes.

Visual inspection of the concrete cores revealed that cores LA-1, LA-2, LA-4 and LA-5 have longitudinal cracks. No gross visual defects were observed in the other cores. The findings from microscopic examination indicate that the concrete has normal level of entrained air voids; the hydration of the cement is reasonable; the presence of some unhydrated cement particles is also observed in the paste; ground granulated blast-furnace slag is present in the concrete as supplementary cementitious material; occurrences of porous interface zones between coarse aggregate and paste were found; cracking is present in both the paste and at the aggregate-cement interface.

Laboratory Procedures

Petrographic examination of the concrete samples was performed in accordance with ASTM C 856, "Standard Practice for Petrographic Examination of Hardened Concrete." Sections were polished and examined using a stereomicroscope at magnifications up to 200×. Small rectangular blocks were cut from concrete samples. One surface of each block was polished, dried, placed on petrographic slide with low–viscosity epoxy resin, and reduced to a thickness of approximately 25 micrometers (0.001 in.). These thin sections were examined using a polarized–light microscope at magnification up to 400×, to determine aggregate mineralogy, paste characteristics, and microstructure.

Two ½-inch thick slabs were sawn from the concrete cores. The slabs were ground and polished to obtain a smooth and plane surface. They were examined using a stereomicroscope at magnifications up to 200×.

Findings

Six thin section samples taken from the received cores were examined using the polarized light microscope. Two polished sections were examined using the stereomicroscope. The findings are summarized as follows:

Aggregates
The coarse aggregate in the concrete is crushed limestone. Coarse aggregate particles are mostly angular, and the maximum size is about 1 inch. Preferential orientation of coarse aggregate particles is not observed in this concrete.

The fine aggregate fraction is predominantly composed of quartz, with a very small amount of feldspar. The fine aggregate is from natural sand and the particles appear rounded to angular.

Cement Paste
The cement is reasonably hydrated and the paste contains some unhydrated cement particles (Figure D1-2). Ground granulated blast-furnace slag particles are also found in the concrete (Figure D1-3).

Air Voids
Small, spherical air voids are observed in the concrete (Figure D1-4), hence the concrete was air entrained. Entrained air voids are well distributed in the concrete, and the air content is at normal level. Small amount of entrapped air voids are also present in the concrete.

Cement-Aggregate Bonding
In general, the cement/aggregate interface is poor to moderate, as shown in Figure D1-5. Very porous interface is not uncommon in this concrete (Figure D1-6), suggesting a rather weak bonding between the aggregate and paste. Cracking at the fine aggregate-

paste interface is also found in the concrete samples, as shown in Figures D1-9 and D1-10.

Secondary Deposit
No secondary deposits (such as ettringite) are found in the concrete samples.

Cracking
Examination of thin sections revealed cracking in the cement paste as well as at the paste-aggregate interface (Figures D1-7 through D1-10). Some random cracks are sporadically seen in the cement paste. Cracks partially surrounded fine aggregate particles are also present in the concrete.

Summary

The concrete is air entrained and the air content is estimated to be at a normal level. The entrained air voids are well distributed in the concrete. Cement was reasonably hydrated and unhydrated cement particles are present in the concrete. Ground granulated blast-furnace slag was added in the concrete as supplementary cementitious material.

Very porous interface is present in the concrete, and it is mainly associated with the coarse aggregate-paste interfaces. The bond between the aggregate and the paste appears poor to moderate. At some fine aggregate-paste interface cracking is also found.

In addition to the major cracks visible in four of the six cores, much smaller size cracks are found in the concrete as observed under the microscope. These small cracks exist mainly in the cement paste and the fine aggregate-paste interface. It is speculated that shrinkage may be the cause of the cracking. Weak bonding between the aggregate and paste is prone to the formation of interface cracking when concrete experiences shrinkage.

Figure D1-1: Six concrete cores as received.

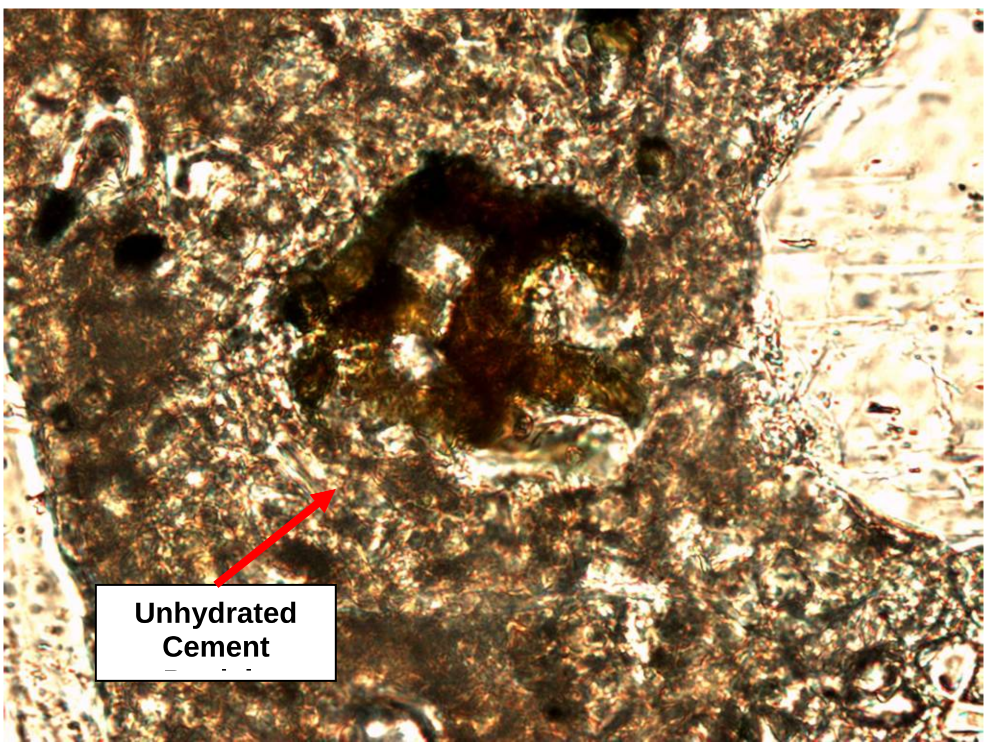


Figure D1-2: Unhydrated cement particle is observed in the concrete. Thin section photomicrograph. Width of field is 0.165 mm.

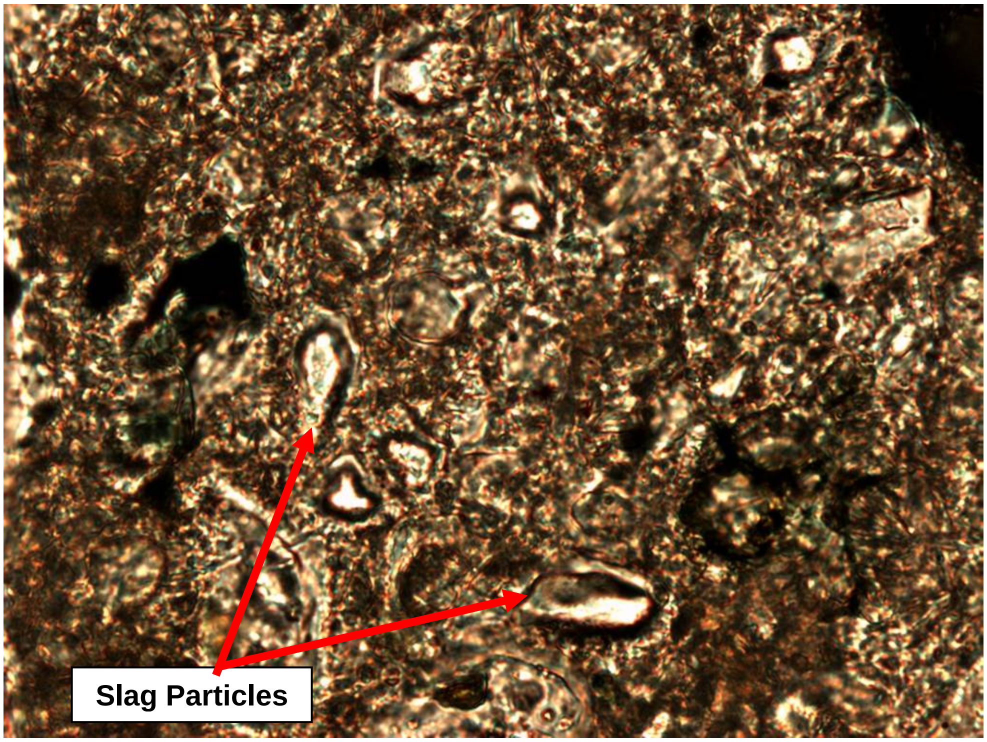


Figure D1-3: Ground granulated blast-furnace slag particles are present in the concrete, as shown in this thin section photomicrograph. Width of field is 0.165 mm.

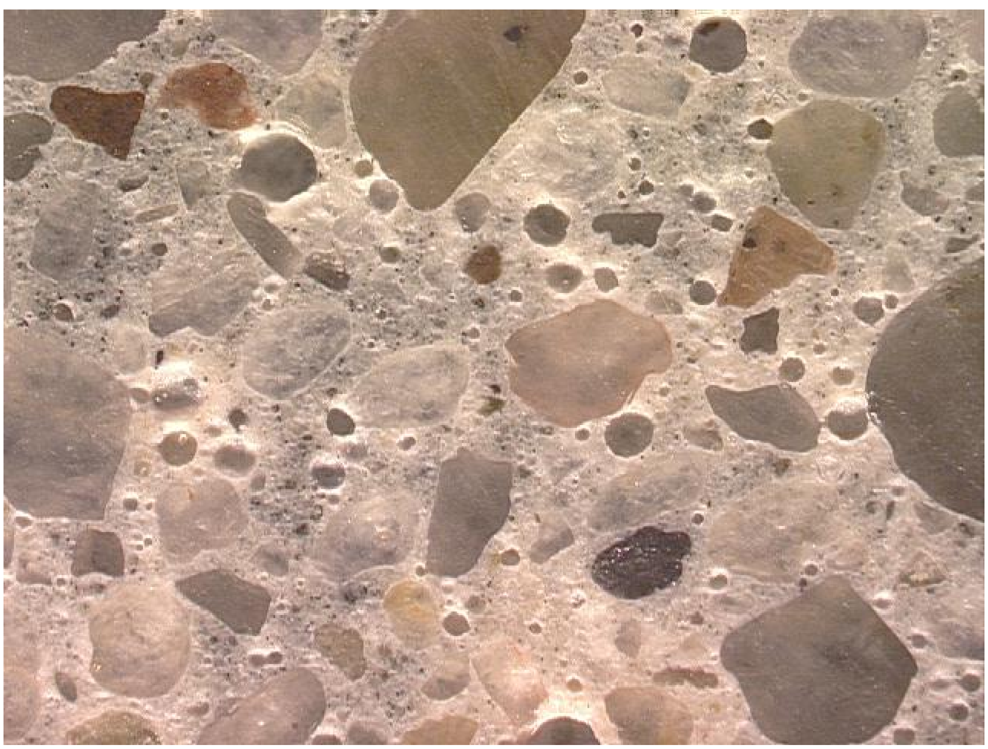


Figure D1-4: Entrained air voids in the concrete. Width of field is 4.0 mm. Polished surface image.

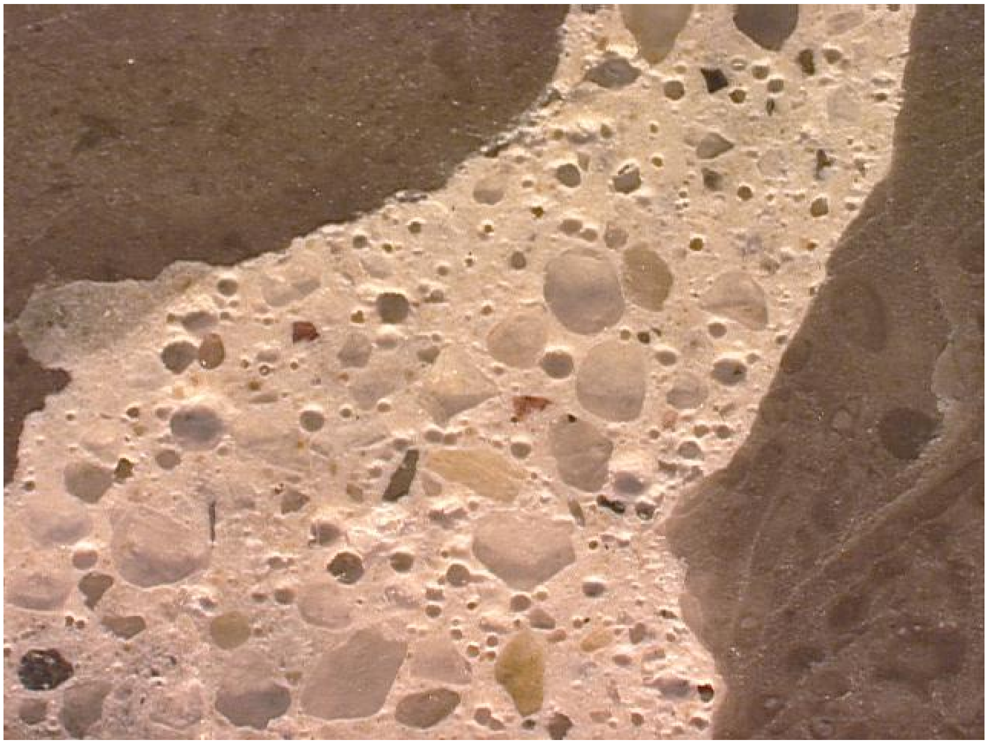


Figure D1-5: The bonding between aggregate and cement paste is poor to moderate. Width of field is 4.0 mm. Polished surface image.

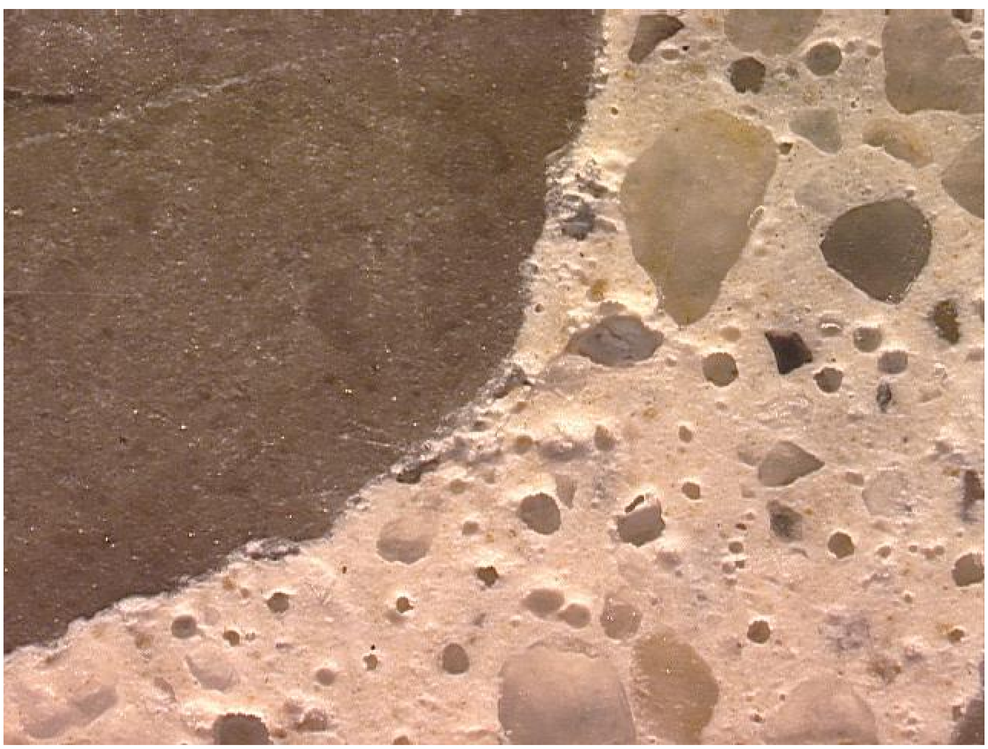


Figure D1-6: Very porous interface between coarse aggregate and cement paste. Width of field is 4.0 mm. Polished surface image.

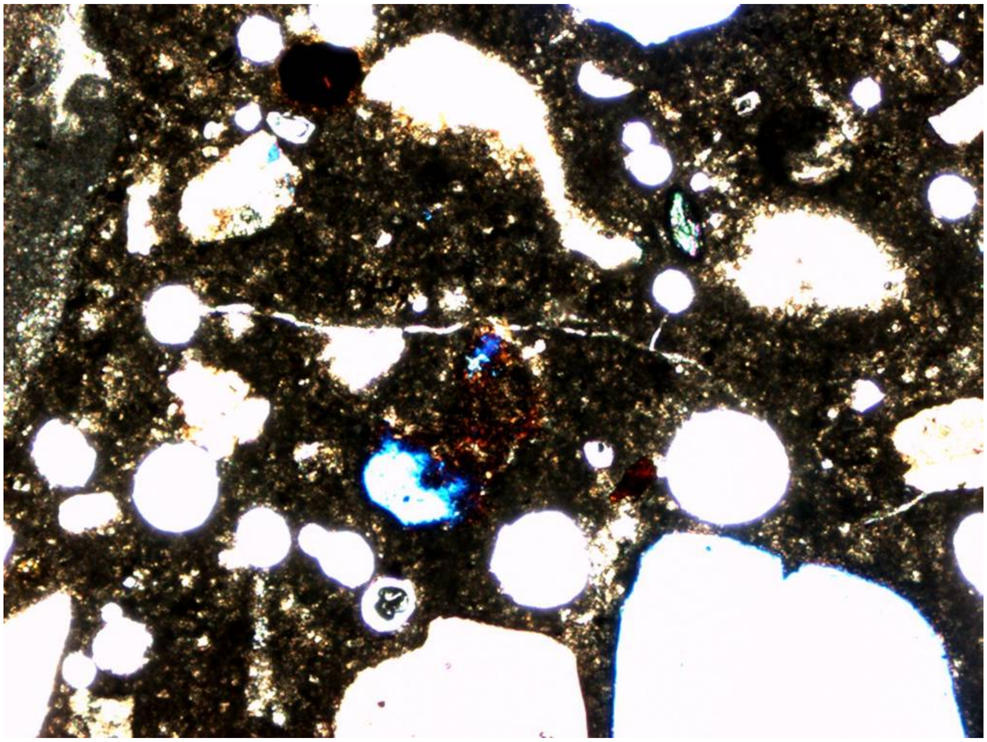


Figure D1-7: Cracks as seen in a thin section. Width of field is 1.6 mm.

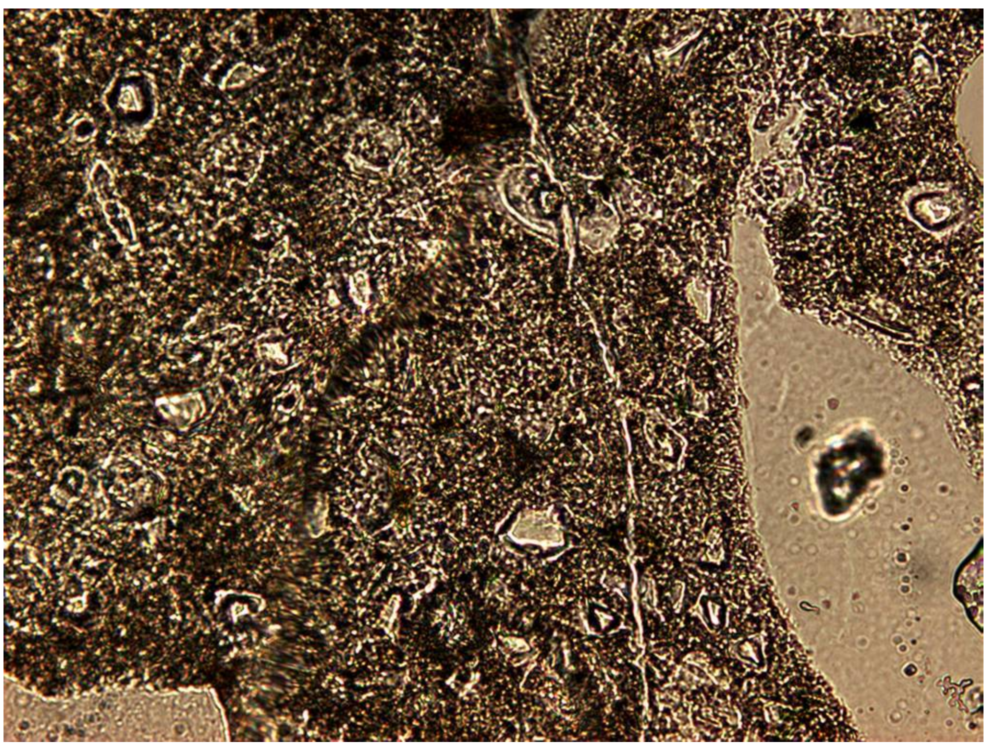


Figure D1-8: Cracks in paste. Width of field is 0.33 mm. Thin section photomicrograph.

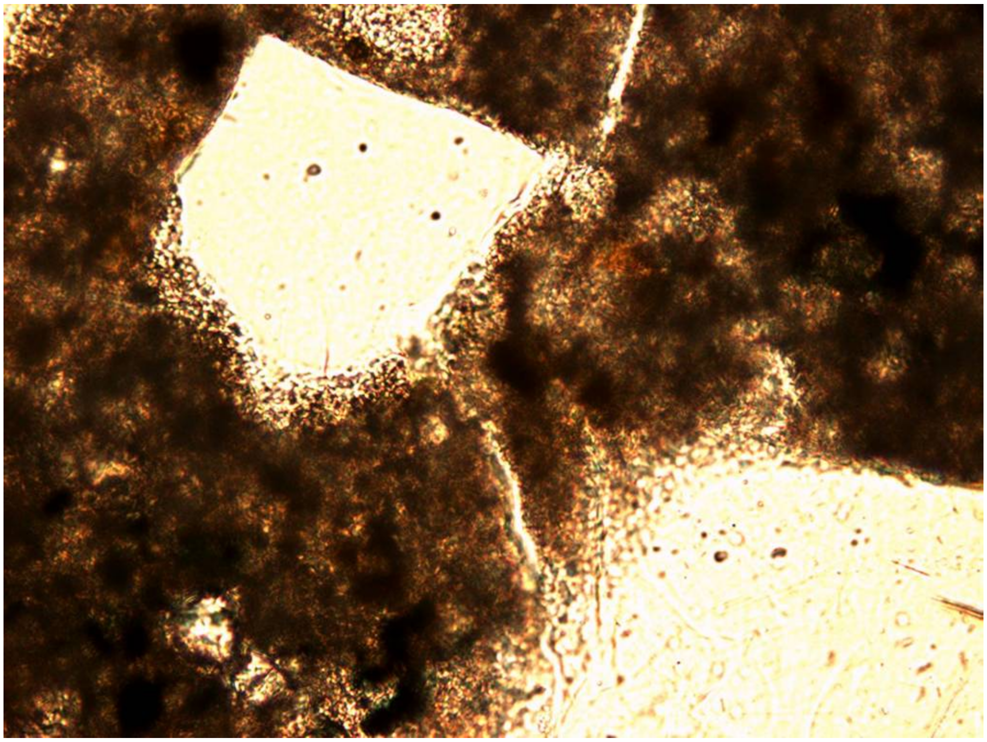


Figure D1-9: Crack along the fine aggregate/paste interface. Width of field is 0.33 mm. Thin section photomicrograph.

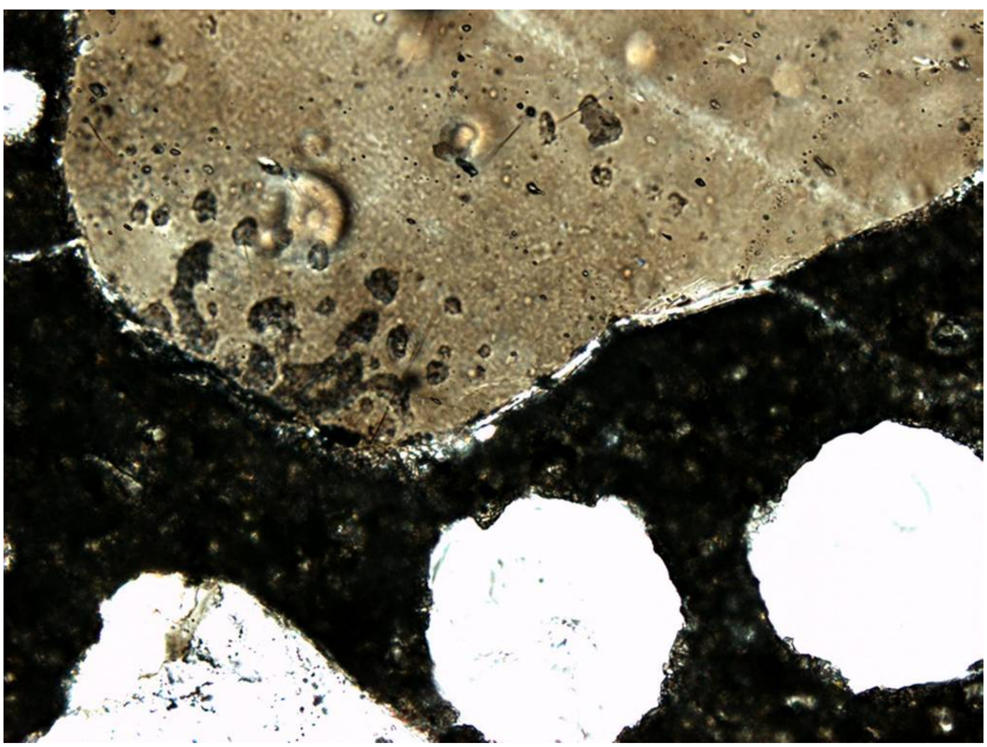


Figure D1-10: Another thin section photomicrograph showing crack along the fine aggregate/paste interface. Width of field is 0.65 mm.

APPENDIX D – Supplement 2

Charenton Canal Bridge, Charenton, Louisiana
Survey Checklist

Checklist

The following checklist is adapted from 201.1R-92, and provided to facilitate a thorough survey. The definition of terms and associated photographs in 201.1 R-92, are utilized to standardize the reporting of the condition of the concrete in the structures.

1. Description of structure or pavement
    - 1.1 Name, location, type, and size: The Charenton Canal Bridge on LA 87 in St. Mary Parish, Louisiana is 365-ft (111-m) long. Clear width of the bridge is 46.5-ft (14.2-m). It consists of two 12-ft (3.66-m) lanes, one 12-ft (3.66-m) shoulder on the westbound bridge and one 8-ft (2.44-m) shoulder on the eastbound bridge.
    - 1.2 Owner, project engineer, contractor, when built: Owner-Louisiana Department of Transportation. This bridge is part of a demonstration project for HPC in bridge structures which were co-sponsored by the Federal Highway Administration (FHWA) and the Louisiana Department of Transportation (LDOTD). The bridge was constructed in 1997 and opened to traffic in November 1999. The contractor was Gulf Coast Prestress Inc. at Pass Christian, Mississippi and Coastal Bridge company at Baton Rouge, Louisiana.
    - 1.3 Design
        - 1.3.1 Architect and/or engineer: The Louisiana Department of Transportation (LDOTD)
        - 1.3.2 Intended use and history of use: To replaces a 55-year-old cast-in-place concrete bridge and carry traffic over the LA 87. Opened to traffic in November 1999.
        - 1.3.3 Special features: Bridge consists of five spans (375-ft in total). Each span consists of five Type III AASHTO girders made of precast, prestressed HPC. The girders are evenly spaced at 10-ft (3.1-m) centers and support the cast-in-place concrete deck. HPC was used in all structural components
    - 1.4 Construction
        - 1.4.1 Contractor-general Gulf Coast Prestress Inc. at Pass Christian, Mississippi and Coastal Bridge company at Baton Rouge, Louisiana
        - 1.4.2 Subcontractors concrete placement: N/A
        - 1.4.3 Concrete supplier: Gulf Coast Prestress Inc.
        - 1.4.4 Agency responsible for testing: N/A
        - 1.4.5 Other subcontractors: N/A
    - 1.5 Photographs
        - 1.5.1 General view Photos 1 through 4
        - 1.5.2 Detailed close up of condition of area Photos 5 through 11
    - 1.9 Sketch map-orientation showing sunny and shady and well and poorly drained regions N/A

2. Present condition of structure Date of Evaluation: The week of February 16, 2004

2.1 Overall alignment of structure No signs of misalignment
- 2.1.I Settlement
- 2.1.2 Deflection
- 2.1.3 Expansion
- 2.1.4 Contraction

2.2 Portions showing distress (beams, columns, pavement, walls, etc., subjected to strains and pressures) Cracks and Efflorescence along the concrete barrier wall

2.3 Surface condition of concrete
- 2.3.1 General (good, satisfactory, poor, dusting, chalking, blisters) Good
- 2.3.2 Cracks Transverse
- 2.3.2.1 Location and frequency See Figure 2a and Figure 2b
  - 2.3.2.6 Type and size (see Definitions) See Figure 2a and Figure 2b

Transverse At the beam diaphragm and panel boundaries
Width (from Crack comparator) Less than 0.03 in..
- Hairline (Less than 1/32 in.)
- Fine (1/32 in. - 1/16 in.)
- Medium (1/16 – 1/8 in.)
- Wide (Greater than 1/8 in.)

Craze N/A
Width (from Crack comparator) ____________
- Hairline (Less than 1/32 in.)
- Fine (1/32 in. - 1/16 in.)
- Medium (1/16 – 1/8 in.)
- Wide (Greater than 1/8 in.)

Map N/A
Width (from Crack comparator) ____________
- Hairline (Less than 1/32 in.)
- Fine (1/32 in. - 1/16 in.)
- Medium (1/16 – 1/8 in.)
- Wide (Greater than 1/8 in.)

D-Cracking N/A
Width (from Crack comparator) ____________
- Hairline (Less than 1/32 in.)
- Fine (1/32 in. - 1/16 in.)
- Medium (1/16 – 1/8 in.)
- Wide (Greater than 1/8 in.)

Diagonal NA
Width (from Crack comparator) NA
- Hairline (Less than 1/32 in.)
- Fine (1/32 in. - 1/16 in.)
- Medium (1/16 – 1/8 in.)
- Wide (Greater than 1/8 in.)

2.3.2.7 Leaching, stalactites N/A

2.3.3 Scaling N/A

2.3.3.1 Area, depth ______

2.3.3.4 Type (see Definitions) ______

Light (Less than 1/8 in.)

Medium (1/8 in. – 3/8 in.)

Severe (3/8 in. – 3/4 in.)

Very Severe (Greater than 3/4 in.)

2.3.4 Spalls and popouts None Observed

2.3.4.1 Number, size, and depth NA

2.3.4.4 Type (see Definitions) NA

Spalls

Small (Less than 3/4 in. depth)

Large (Greater than 3/4 in. depth)

Popouts

Small (Less than 3/8 in. diameter)

Medium (3/8 in. – 2 in. diameter)

Large (Greater than 2 in. diameter)

2.3.5 Extent of corrosion or chemical attack, abrasion, impact, cavitation N/A

2.3.6 Stains, efflorescence Efflorescence at a few locations on the underside of the bridge and along the concrete barrier wall

2.3.7 Exposed reinforcement None

2.3.8 Curling and warping N/A

2.3.9 Previous patching or other repair N/A

2.3.10 Surface coatings N/A

2.3.10.1 Type and thickness N/A

2.3.10.2 Bond to concrete N/A

2.3.10.3 Condition N/A

2.3.11 Abrasion N/A

2.3.12 Penetrating sealers __

2.3.12.1 Type N/A

2.3.12.2 Effectiveness N/A

2.3.12.5 Discoloration N/A

2.4 Interior condition of concrete (in situ and samples) N/A

2.4.1 Strength of cores

2.4.2 Density of cores

2.4.3 Moisture content

2.4.4 Evidence of alkali-aggregate or other reactions N/A

2.4.5 Bond to aggregate, reinforcing steel, joints N/A

2.4.6 Pulse velocity

2.4.7 Volume change

2.4.8 Air content and distribution

2.4.9 Chloride-ion content

2.4.10 Cover over reinforcing steel

2.4.11 Half-cell potential to reinforcing steel.

2.4.12 Evidence of reinforcement corrosion
2.4.13 Evidence of corrosion of dissimilar metals
2.4.16 Delaminations ______ N/A
2.4.16.1 Location ______ N/A
2.4.16.2 Number, and size ______ N/A
2.4.15 Depth of carbonation
2.4.16 Freezing and thawing distress (frost damage)
2.4.17 Extent of deterioration
2.4.20 Aggregate proportioning, and distribution

3. Nature of loading and detrimental elements
3.1 Exposure
3.1.1 Environment (arid, subtropical, marine, freshwater, industrial, etc.) marine
3.1.2 Weather-(July and January mean temperatures, 91ºF and 60ºF
mean annual rainfall and 5.1-in
months in which 60 percent of it occurs) July
3.1.3 Freezing and thawing negligible
3.1.4 Wetting and drying Minimal annual exposure
3.1.7 Drying under dry atmosphere ______ N/A
3.1.6 Chemical attack-sulfates, acids, chloride ______ N/A
3.1.7 Abrasion, erosion, cavitation, impact ______ N/A
3.1.8 Electric currents ______ N/A
3.1.9 Deicing chemicals which contain chloride ions ______ N/A
3.1.10 Heat from adjacent sources ______ N/A
3.2 Drainage ______ N/A
3.2.1 Flashing ______
3.2.2 Weepholes ______
3.2.3 Contour ______
3.2.4 Elevation of drains ______
3.3 Loading Research Test Data Available in Compilation CD Version 3
3.3.1 Dead ______
3.3.2 Live ______
3.3.3 Impact ______
3.3.4 Vibration ______
3.3.5 Traffic index ______
3.3.6 Other ______
3.4 Soils (foundation conditions) ______
3.4.1 Compressibility
3.4.2 Expansive soil
3.4.3 Settlement
3.4.4 Resistivity
3.4.5 Evidence of pumping
3.4.6 Water table (level and fluctuations)

4.	Original condition of structure			Good
	4.1	Condition of formed and finished surfaces		Good
		4.1.1 Smoothness		
		4.1.2 Air pockets ("bugholes")		
		4.1.3 Sand streaks		
		4.1.4 Honeycomb		
		4.1.5 Soft areas (retarded hydration)		
		4.1.6 Cold joints		
		4.1.11 Staining		
		4.1.12 Sand pockets		
	4.2	Defects		N/A
		4.2.1 Cracking		
		4.2.1.1	Plastic shrinkage	
		4.2.1.2	Thermal shrinkage	
		4.2.1.3	Drying shrinkage	
		4.2.4 Curling		
5.	Materials of Construction			See Table 2 and Table 3
6.	Construction Practices			See Report pg. 8

APPENDIX E

120th Street and Giles Road Bridge near Omaha, Nebraska

HIGH PERFORMANCE CONCRETE BRIDGE DECK INVESTIGATION

120th Street and Giles Road Bridge near Omaha, Nebraska

I. BACKGROUND

The 120th Street and Giles Road Bridge in Sarpy County, near Omaha, Nebraska is the first High Performance Concrete (HPC) bridge built by the Nebraska Department of Roads. HPC was used in girders and bridge deck. The bridge was built in the summer of 1995, and opened to traffic in July 1996.

The 120th Street and Giles Road Bridge consists of three equal 75-ft (22.9-m) spans. Total length of bridge is 225-ft (68.6-m). It utilizes seven lines of NU1100 (1100-mm deep) pre-tensioned concrete girders. Clear width of the bridge is 82 ft (25.8 m). The girders were pre-tensioned with thirty or thirty-four (depending on the span) 0.5-in. (12.7-mm) diameter strands at 2-in. (50-mm) center-to-center spacing. The cast-in-place deck has a thickness of 7-½ in. (190.5-mm). The HPC bridge used simple-span girders with negative-moment reinforcement in the deck.

It is noted that a conventional concrete bridge with identical geometry was constructed less than a half mile (0.8 km) from the HPC bridge. The conventional bridge is used as a control structure to help evaluate the service life of the HPC bridge. In addition, the HPC bridge was already designed using conventional concrete. This allowed the Nebraska Department of Roads to establish incremental costs for design and construction.

Nebraska uses deicing salts and is in a region of high freeze/thaw cycles; therefore, the focus was to specify a durable deck concrete. The compressive strength specified for the concrete girders is 12,000 psi (83 MPa) at 56 days. Compressive strength of 8,000 psi (55 MPa) at 56-days and a chloride penetration of less than 1800 coulombs at 56 days were specified for bridge deck concrete. Fly ash was used in the deck concrete to meet the chloride permeability requirement. The specified strengths for the girders and deck were intentionally higher than required by design as part of the implementation strategy. The water-to-cementitious material ratio for the girders was specified as less than 0.28.

The 120th Street and Giles Road Bridge is part of a demonstration project for HPC in bridge structures, which are co-sponsored by the Federal Highway Administration (FHWA) and the Nebraska Department of Roads. The goal of this showcase HPC bridge project was to optimize the design and implement a strategy that eliminates or reduces the fear of producing, placing, and curing HPC. The University of Nebraska at Omaha Center for Infrastructure Research did trial batches and testing to optimize mix designs for the girders and the deck. The researchers also instrumented, monitored, and tested the girder and deck concrete during and after construction. Instrumentation included embedded thermocouples, electrical resistance strain gauges, and vibrating wire gauges.

Following the success of the 120th Street and Giles Road Bridge, the Nebraska Department of Roads decided to initiate a strategic plan for the implementation of HPC on a statewide basis.

II. SCOPE OF SERVICES

Professional Service Industries (PSI) is under contract with the Federal Highway Administration (FHWA) to conduct bridge deck inspections for HPC bridges. Our scope of services on this bridge included a series of tasks and sub-tasks, which are described as follows:

- Collect all available information relevant to the construction of each bridge, including;
    - Deck Concrete Properties
    - Specified Deck Concrete Construction Procedures
    - Approved Concrete Mix Proportions
    - Measured Properties from QC
    - Other Measured Properties
    - Actual Method of Deck Placement
    - Average Daily Traffic (ADT)
    - Exposure Condition of the Bridge
    - Any Performed Maintenance
    - Any Inspection Reports
- Visually inspect the bridge, and obtain the following:
    - General condition of the deck top surface
    - Determination of the maximum crack width
    - General condition of the deck underside
    - General condition of the girders
    - Photograph areas of significant deterioration
    - Prepare drawings locating defects
    - Extract 6 concrete core samples

III. COMPILATION OF BRIDGE CONSTRUCTION INFORMATION

Information sources used for this report include bridge drawings, inspection reports, bridge summary data sheets, and technical information contained in FHWA's "Compilation and Evaluation of Results from High Performance Concrete Bridge Projects" version 3.

Deck Concrete Properties

The concrete mixture design for the cast-in-place bridge deck of the 120th Street and Giles Road Bridge is based on strength of 8,000 psi (55 MPa) at 56-days and a chloride penetration of less than 1800 coulombs at 56 days. Contractor was allowed to use 28-day strength of field-cured cylinders at 95% of the 56-day strength as an acceptance criterion. Chloride permeability less than 1900 coulombs at 28 days was also acceptable. Fly ash and silica fume were used. For adequate protection against the likelihood of freeze-thaw cycles, the air content was specified to be between 5 and 7.5 percent. Slump for cast-in-place deck concrete should be less than 8 in.

Specified Deck Concrete Construction Procedures

In the construction of the 120th Street and Giles Road Bridge, the contractor was required to demonstrate proper batching, placement, finishing, and curing by placing a 4 yd^3 (3 m^3) trial pour. The objective of the trial pour was to simulate the actual job conditions including plant conditions, transit equipment, travel time, admixtures, forming, placement equipment and personnel. The concrete could not be placed when the rate of evaporation exceeded 0.15 lb water/ft^2 / hour (0.73 kg water/m^2 / hour) as determined by the monograph provided in the contract. Wind speed, air temperature, and humidity were measured by the contractor and verified by the owner. At the beginning of the deck pour, if the air temperature in the shade was above 80ºF (27ºC) the contractor was not allowed to place any concrete.

The contractor was required to submit the mix design 30 days prior to placing the concrete with the following test results: 56-day compressive strength based on a minimum of sixty 4 x 8 in. (100 x 200 mm) cylinders; chloride permeability; flexural strength; alkali reactivity test of aggregates for 16 and 30 days; modulus of elasticity; splitting tensile strength; shrinkage and abrasion resistance. These tests primarily provided information on the mix characteristics; however, the chloride permeability results were used as a basis of acceptance or rejection.

Approved Concrete Mix Proportions

Deck

Table 1 lists the approved concrete properties for the cast-in-place deck. Note that the selected mix design was chosen based on performance during trial batching.

TABLE 1: Approved Concrete Properties

Property	Value
Cement type:	Type IP
Minimum Cementitious Materials Content:	750 lb/yd^3
Max. Water/Cementitious Materials Ratio:	0.39
Fly Ash Type:	C
Fly Ash Quantity:	75 lb/yd^3
Fine Aggregate Quality:	1400 lb/yd^3
Coarse Aggregate Maximum Size:	½-in
Coarse Aggregate Type:	Limestone
Coarse Aggregate Quantity:	1400 lb/yd^3
Water:	255 lb/yd^3
Water Reducer Brand:	MasterBuilders Pozzolith 322N
Water Reducer Type:	A
Water Reducer Quantity:	4 fl oz/yd^3
High Range Water Reducer Brand:	MasterBuilders Rheobuild 1000
High Range Water Reducer Type:	F
High Range Water Reducer Quantity:	18 fl oz/yd^3
Air Entrainment Quantity:	5 fl oz $/yd^3$
Water/Cementitious Materials Ratio:	0.31

Air content, slump, and compressive strengths, modulus of elasticity, splitting tensile strength, and shrinkage properties are summarized in Table 2. Table 3 provides the measured modulus of rupture of the deck concrete mix from QC tests.

TABLE 2: Approved Properties of the Production Concrete Mixture for the Cast-in-Place Deck

Property	Value
Slump:	3 ¾-8 ½ in.
Air Content:	2.5 to 8.8%
Unit Weight:	142.7 to 149.6 lb/ft^3
Compressive Strength:	6919 psi at 7 days 8628 psi at 28 days 9133 psi at 56 days
Modulus of Elasticity: (4 ×8-in cylinders, ASTM C469)	5440 ksi (age not stated)
Splitting Tensile Strength: (6 ×12-in cylinders, water cured, ASTM C496)	598, 617, and 617 psi at 28 days
Shrinkage: (2 × 2 × 10-in prism, per ASTM C157)	0.039% at 64 weeks
Chloride Permeability: (AASHTO T277)	507 and 671 coulombs (Age not stated)

NOTE: Compressive strengths were measured on 4x8-in cylinders. The reported compressive strengths are the average from 29 batches for the decks.

TABLE 3: Measured Modulus of Rupture of the Production Concrete Mixture for the Cast-in-Place Deck (ASTM C 78)

Days Cured		Test Age	Modulus of Rupture, psi
Field	Laboratory		
1	27	28	855
1	48	49	850
1	27	28	1030
1	48	49	890
1	27	28	925
1	47	49	915
1	27	28	840
1	47	48	865

NOTE: The reported modulus of rupture is the average of two specimens.

Measured Properties from QC Tests of Production Concrete

Cast-in-Place Deck

The measured compressive strength of the production concrete for the cast-in-place deck is present in Tale 4.

TABLE 4: Measured Compressive Strength of the Production Concrete Mixture for the Cast-in-Place Deck

Concrete Age, days	7	28	56
Compressive Strength, psi	7252	9606	10,433

Measured Properties from Research Tests of Production Concrete

Research tests of three different concrete mixtures were performed. The mixture proportion, compressive strength, and modulus of elasticity were summarized in Table 5, 6, and 7, respectively. The compressive strength of the 120th Street and Giles Road Bridge is well above the specified values.

TABLE 5: Mixture Proportions of 3 Types of Concrete Used in the Research Test of Production Concrete

Mix No.:	1 (12 SF)	2 (12 FA)	3 (8 FA)
Cement Type:	I	III	IP
Cement Quantity:	750 lb/yd^3	680 lb/yd^3	750 lb/yd^3
Fly Ash Type:	C	C	C
Fly Ash Quantity:	200 lb/yd^3	320 lb/yd^3	75 lb/yd^3
Silica Fume Quantity:	50 lb/yd^3	—	—
Fine Aggregate:	990 lb/yd^3	933 lb/yd^3	1400 lb/yd^3
Coarse Aggregate, Max. Size:	1/2 in	3/8 in	1/2 in
Coarse Aggregate, Type:	Limestone	Limestone	Limestone
Coarse Aggregate, Quantity:	1860 lb/yd^3	1913 lb/yd^3	1400 lb/yd^3
Water:	240 lb/yd^3	254 lb/yd^3	255 lb/yd^3
High-Range Water-Reducer:	4 oz/100 lb	4 oz/100 lb	4 oz/100 lb
Retarder:	30 oz/100 lb	34.2 oz/100 lb	18 oz/100 lb
Air Entrainment:	—	—	—
Water/Cementitious Materials Ratio:	0.24	0.25	0.31

TABLE 6: Measured Compressive Strengths of 3 Types of Concrete Used in the Research Test of Production Concrete

Concrete Age, days	Mix No.		
	1 (12 SF)	2 (12 FA)	3 (8 FA)
7	10,200	8730	7360
14	14,350	11,450	8090
28	11,750	12,360	9030
42	13,150	13,660	9670
56	14,350	13,720	9940

NOTE: Compressive strengths were measured on 4x8-in cylinders. Specimens were cured in water until test.

Actual Method of Deck Placement

Construction of the 120th Street and Giles Road Bridge began in summer of 1995. Placing High Performance Concrete in the bridge deck was not allowed until successful completion of a test pour. Successful completion was defined as achieving concrete compressive strengths in excess of the minimum specified strength and demonstrating a successful fogging and curing operation with minimal cracking of the test pour.

TABLE 7: Measured Modulus of Elasticity of 3 Types of Concrete Used in the Research Test of Production Concrete

Mix No.	Date Cast	Concrete Age, days	Compressive Strength, psi	Modulus of Elasticity, ksi
1 (12 SF)	5/30/96	7	8530	5353
		14	9700	6472
		28	11,420	7295
		56	13,290	7453
	6/20/96	7	9890	6445
		14	11,910	7404
		28	12,910	7528
		56	14,240	8026
	7/16/96	1	5520	4491
		7	9540	6704
		14	10,334	6563
		28	11,383	7780
		60	14,980	7482
	7/18/96	4	7820	5587
		7	8940	6890
		14	10,500	7073
		28	11,710	7338
		140	14,980	7401
2 (12 FA)	9/20/96	80	16,400	8188
		86	17,060	8004
	10/4/96	56	13,410	7259
		72	13,150	7335
	11/7/96	7	8730	5564
		14	11,450	6427
		28	12,830	6511
		43	13,659	7519
		56	13,721	7593
	11/8/96	28	13,840	7299
		42	14,776	7304
3 (8 FA)	1/31/97	10	7615	5677
		14	8090	5808
		28	9030	6059
		35	9455	6162
		42	9671	6064
		56	9942	6281
	2/21/97	14	8381	5613
		21	9337	6088
		28	9069	6025
		35	9640	6517
		50	10,296	6231
	9/21/96	82	12,828	7072
		86	17,062	7642

Air entraining admixture was allowed at the project site if the supplier had approval. Air content was between 5 percent and 7.5 percent. The actual slump was required to be consistent within ±1 in. (25 mm). Water was not allowed to be added to the concrete after it was batched and placed on the truck for delivery to the project site. The special provisions required the deck to be water-cured for 8 curing days. E-CON vapor retardant was applied to reduce evaporation, and continuous fogging was followed by wet curing under burlap. During the initial set the air above the surface was to be kept in a state of high humidity. This was to be accomplished by spray nozzles, which atomized water into a mist, and the water did not flow or accumulate on the surface for at least 5 hours. Afterwards, wet mat curing was used for 8 days.

Average Daily Traffic (ADT) and Average Daily Truck Traffic (ADTT)

Average daily traffic for both northbound and southbound lanes was calculated based on a count of all vehicles crossing the bridge during a 5 minutes period beginning at 0910 hrs on June 28, 2004. The northbound ADT was 19,872, including 18,144 cars and 1,728 trucks. The southbound ADT was 14,112, including 12,960 cars and 1,152 trucks. The estimation of traffic flow was made while the PSI inspection crew was on-site.

Exposure Condition of the Bridge

The 120th Street and Giles Road Bridge in Sarpy County, near Omaha, Nebraska is normal over water. It experiences high volume of traffic and a wide range of climate conditions throughout the year. The mean daily maximum temperatures for Omaha range from 31°F in January to a high of 88°F in July. Mean daily minimum temperatures in Omaha vary between 11°F in January and 66°F in July. The Omaha area experiences about 29.3 in. precipitation per year, implying that the bridge experiences many wet/dry cycles. The temperature history throughout the year indicates a considerable number of freeze-thaw cycles.

Performed Maintenance

No documents were found which would indicate any maintenance had been performed since the bridge was constructed.

Inspection Reports

As part of the project, University of Nebraska at Omaha Center for Infrastructure Research instrumented, monitored, and tested the girder and deck concrete during and after construction.

It is reported that a survey/inspection of the bridge was conducted on July 17, 1997, by Mr. Milo Cress of FHWA. A number of well-distributed hairline cracks were observed on the top surface of the sidewalks and the median. The narrow width of the cracks and the wide distribution with no concentration at certain locations indicate that these are shrinkage cracks. The concrete used at these locations was different from the mix used for the concrete deck. It appears that the lower strength and less stringent curing process used for the sidewalks and median concrete resulted in these shrinkage cracks.

Several cracks were also visible on the bottom surface of the deck. The orientation of all of these cracks is perpendicular to the axis of the girders. In addition, these cracks are well distributed. The orientation and narrow width of these cracks indicate that the cracks are not related to any structural overstress, rather than they are shrinkage cracks. The cracks can be related to several reasons such as: 1) strict curing requirements were applied to the top surface of the deck, but the bottom surface received no special attention and therefore, the bottom surface of the deck was deprived of moisture needed for curing, and 2) the high temperature variation between the top surface and the bottom surface of the deck.

IV. BRIDGE DECK INSPECTION

PSI personnel performed a visual inspection of the bridge decks during the week of June 28, 2004. The results of the inspection are summarized as follows.

General Condition of the Deck Top Surface

Figure 1 shows the general layout of the decks for the 120th Street and Giles Road Bridge. Results of visual inspection of the decks are shown in Figure 2. Figure 2 also illustrates the locations of drilled cores. Surface defects observed and documented during visual inspection primarily included diagonal cracks, transverse cracks, and longitudinal cracks (see photos 4 through 6). There are broken and polished tined surfaces on the deck (photos 7 and 8). Apparent signs of other serious damages such as freeze-thaw, D-cracking, alkali-silica reaction, and alkali-aggregate reaction were not observed.

A total of 259 cracks (170 traverse cracks, 25 longitudinal cracks, and 64 diagonal cracks) were recorded during visual survey of the bridge decks (see Figure 2). The sum of crack lengths was 1,664.5 ft over a bridge deck area of 18,450 ft^2. Crack density (total crack length / deck area) for the bridge was calculated to be 0.0902 ft/ft^2.

It is noted that the number of transverse cracks account for the majority of cracks recorded (65.6%), and the total length is 1,082.5 ft. The 64 diagonal cracks have a total length of 349 ft. The total length for longitudinal cracks is 233 ft. Span 2 and Span 3 have similar crack counts (i.e., 99 cracks measured on Span 2, and 102 cracks measured on Span 3). Span 1 has 58 cracks combined.

All cracks measured are hairline cracks with a width of less than 0.031 in. Typical crack patterns on the bridge decks are shown in photos 4 through 6.

Diagonal cracks were typically limited at span ends. Transverse cracks were typically found in the traffic lanes and shoulders. Small surface spalls, either due to breaking of tined edges or the cracked edges, and polished surfaces were observed (see photos 7 and 8).

The number, length and density of cracks for entire bridge decks in both directions are shown in Tables 8 through 10, and described below according to the crack type.

Transverse Cracks: Figure 2 illustrates the transverse cracks that were identified on the surface of the bridge decks. Table 8 provides the detailed information regarding transverse cracks identified on the bridge decks. The crack densities (crack length per deck area) range from 0.0390 to 0.0756 ft/ft^2 for the 3 spans investigated.

TABLE 8: Measured Transverse Cracks on the Bridge Decks

Traverse Cracks	Count	Length Range (feet)	Mean Length of Cracks (feet)	Median Length of Cracks (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span 1	38	2 to 12	6.3	6	240	6150	0.0390
Span 2	77	2 to 13	6.1	6	465.5	6150	0.0756
Span 3	55	1 to 14	6.9	6	377	6150	0.0613

Diagonal Cracks: The diagonal crack densities (crack length per deck area) range from 0.0129 to 0.0295 ft/ft^2 for the 3 spans investigated. Diagonal cracks in the bridge decks typically present near the joints.

TABLE 9: Measured Diagonal Cracks on the Bridge Decks

Traverse Cracks	Count	Length Range (feet)	Mean Length of Cracks (feet)	Median Length of Cracks (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span 1	15	2 to 8	5.2	6	78.5	6150	0.0128
Span 2	16	1 to 12	5.6	5	89	6150	0.0145
Span 3	33	1 to 13	5.5	4	181.5	6150	0.0295

Longitudinal Cracks: The length of longitudinal cracks is insignificant. Several of the longitudinal cracks were along the beams and at the boundaries of the precast deck panels. The longitudinal crack densities (crack length per deck area) range from 0.0021 to 0.0204 ft/ft^2 for the 3 spans investigated.

TABLE 10: Measured Longitudinal Cracks on the Bridge Decks

Traverse Cracks	Count	Length Range (feet)	Mean Length of Cracks (feet)	Median Length of Cracks (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span 1	5	2 to 38	15.4	10	77	6150	0.0125
Span 2	6	1 to 7	3	4	18	6150	0.0029
Span 3	14	2 to 20	9.9	9.5	138	6150	0.0224

Maximum Crack Width

The maximum width of transverse cracks was measured to be 0.018 in. According to ACI 201, these crack widths are classified as hairline cracks. The fine width cracks were generally located at span ends and some exhibited spalling due to the breaking of the edges.

General Condition of the Deck Underside

The underside of the deck is in good condition in general. Photo 9 shows the general view of the underside of the deck.

General Condition of the Girders

The girders were inspected without the aide of any access equipment. Spalling of joints at the second girder on north side was observed (see photo 10).

Concrete Core Samples

Six cores, 3-¾ inches in diameter, were retrieved from the decks. The core sample locations are shown on Figure 2. The locations were evenly distributed along each shoulder of the bridge. The cores were labeled NE-1 through NE-6, and were transferred to FHWA for further analysis.

TABLE 11: Core Dimensions

Sample	NE-1	NE -2	NE -3	NE -4	NE -5	NE -6
Diameter (in.)	3¾	3¾	3¾	3¾	3¾	3¾
Length (in.)	2	1¾	2	3½	3	5

Preliminary Conclusions

The construction of the 120th Street and Giles Road Bridge in Sarpy County, near Omaha, Nebraska was the first use of high performance concrete (HPC) in bridge construction by Nebraska Department of Roads. It was built in the summer of 1995. An important outcome of this project was the initiation of a strategic plan for the implementation of HPC on a statewide basis and improved understanding of design, batching, placing, finishing, and curing of HPC materials.

The visual inspection of the bridge decks as part of our study was performed about eight years after the bridge was opened to traffic. A total of 259 cracks (170 traverse cracks, 25 longitudinal cracks, and 64 diagonal cracks) were recorded during visual survey of the bridge decks (see Figure 2). The sum of crack lengths was 1,664.5 ft over a bridge deck area of 18,450 ft^2. Crack density (total crack length / deck area) for the bridge was calculated to be 0.0902 ft/ft^2. The crack density as compared to other HPC bridge decks is relatively high. Majority of the cracks observed is transverse cracks, which were typically found in the traffic lanes and shoulders.

Compared to results reported by Mr. Milo Cress of FHWA, which were obtained approximately 7 years ago during a bridge survey, the cracks in the shoulder and concrete median (see photo 3) between eastbound and westbound traffic lanes have not changed significantly. In addition, many transverse cracks were observed in the traffic lanes for both eastbound and westbound of the bridge.

All cracks measured are hairline cracks with a width of less than 0.031 in. Typical crack patterns on the bridge decks are shown in photos 4 through 6. The relatively flexible bridge structural system, combined with the heavy ADT on the bridge, might have contributed to the development of some of the cracks.

In general, the top surface of 120th Street and Giles Road Bridge was in good condition, with only hairline cracks found. It shows that HPC designs provide significantly higher strength that can lead to more efficient designs and improved durability. The Sarpy County project has demonstrated that HPC can be mixed, transported, placed, finished and cured with relative ease.

Petrographic examination was performed on six concrete cores that were retrieved from the bridge. The dimension of the concrete cores was 3-¾-in. diameter, 1- to 5-in. long. The identification on the cores was as following: NE#1, NE#2, NE#3, NE#4, NE#5, and NE#6. All of the cores showed evidence of being broken off, and not being drilled all the way through. Two cores (NE#3 and NE#4) were split along the length into halves along the existing crack. Visual inspection of the concrete cores revealed that two cores (NE#2 and NE#5) had cracks along the length of the core.

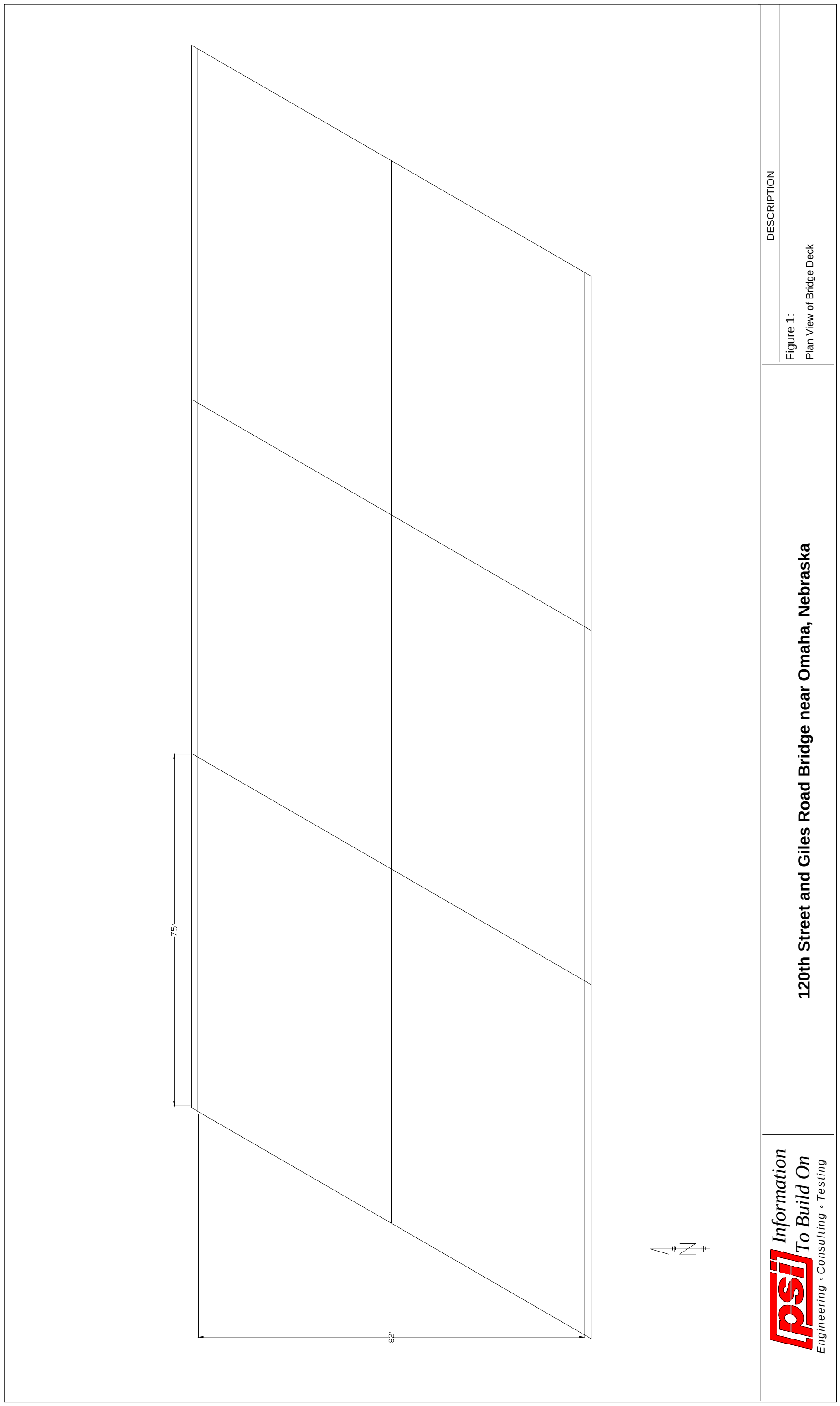
The coarse aggregate in the concrete was predominantly crushed limestone, with a small portion of gravel granite and quartzite. Most coarse aggregate particles were angular, and the maximum size was about 1/2 inch. Preferential orientation of coarse aggregate particles was not observed in this concrete. The natural sand fine aggregate was mainly composed of quartz, with a small portion of feldspar, granite, quartzite, and limestone. The fine aggregate particles appeared rounded to angular.

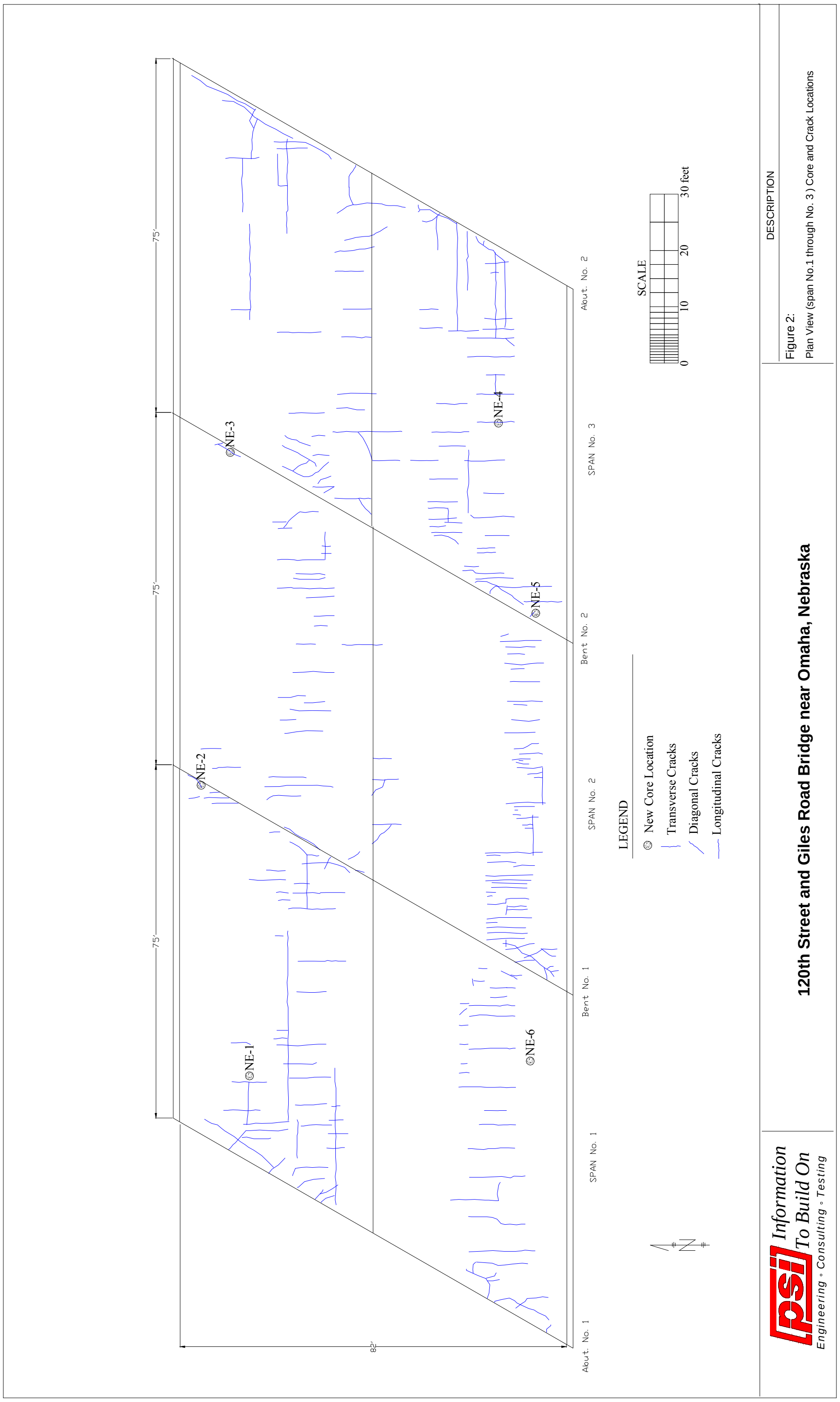
The cement was reasonably hydrated with respect to the age of the concrete. The paste contained some unhydrated cement particles. A small amount of fly ash particles was also present in the paste matrix. The paste/aggregate bond appeared to be good.

The concrete was air entrained, and small, spherical air voids were well distributed in the concrete. Entrapped air voids were also present in the concrete. The amount of entrapped air content was estimated to be slightly higher than normal level.

Cracks of microscopical scale were present in the concrete. They existed in cement paste as well as in the interfacial region between the aggregate and paste. It was speculated that shrinkage may be the cause of the cracking. A network of cracking, formed by several cracks connecting with each other, was also noticed in the examined concrete sample.

Respectfully Submitted,
Professional Service Industries, Inc.
Structural Investigation & Petrography Department


75'
28'
N
Information
To Build On
Engineering • Consulting • Testing
120th Street and Giles Road Bridge near Omaha, Nebraska
DESCRIPTION
Figure 1:
Plan View of Bridge Deck


75'
75'
75'
82'
Abut. No. 1
Bent No. 1
Bent No. 2
Abut. No. 2
SPAN No. 1
SPAN No. 2
SPAN No. 3
NE-1
NE-2
NE-3
NE-4
NE-5
NE-6
LEGEND
New Core Location
Transverse Cracks
Diagonal Cracks
Longitudinal Cracks
SCALE
0 10 20 30 feet
N
DESCRIPTION
Figure 2:
Plan View (span No.1 through No. 3) Core and Crack Locations
120th Street and Giles Road Bridge near Omaha, Nebraska
psi Information To Build On
Engineering • Consulting • Testing

Photographic Documentation

Photo 1: General view of the eastbound bridge facing south.

Photo 2: General view of the westbound bridge at eastern end.

Photo 3: General view of the concrete median between eastbound and west bound traffic lanes.

	Photo 4: Close up view of typical diagonal crack.
	Photo 5: Close up view of typical transverse crack.
	Photo 6: Close up view of typical longitudinal crack.

	Photo 7: Close up view of broken tined surfaces.
	Photo 8: Close up view of polished deck surfaces.
	Photo 9: General overview of underside of the bridge.

Photo 10: Spalling near the joints of second girder north side.

Photo 11: Taking core from a location over a transverse crack.

APPENDIX E – Supplement 1

120th Street and Giles Road Bridge near Omaha, Nebraska
Petrographic Report

PETROGRAPHIC EXAMINATION OF CONCRETE CORES FROM A BRIDGE IN NEBRASKA (NE)

A report submitted to the Structures Laboratory at TFHRC

Rongtang Liu
Petrographic Laboratory
FHWA TFHRC
(Reviewed by Richard Meininger, PE; Concrete Laboratory; printed in color 9-12-2006)

August 15, 2006

1. Introduction

Six concrete cores were received from the Structures Laboratory of the Turner-Fairbank Highway Research Center for petrographic examination. These cores were collected from a concrete bridge in Nebraska.

The dimension of the concrete cores was 3.75-in. diameter, 1- to 5-in. long. The identification on the cores was as following: NE#1, NE#2, NE#3, NE#4, NE#5, and NE#6 (Figure E1-1).

Thin sections, as well as polished slabs, were made from the concrete cores for microscopic examination. Petrographic examination was performed on these samples using optical microscopes.

All of the cores showed evidence of being broken off, and not being drilled all the way through. Two cores (NE#3 and NE#4) were split longitudinally into halves along the existing crack. Visual inspection of the concrete cores revealed that two cores (NE#2 and NE#5) have longitudinal cracks. The findings from microscopic examination indicate that the concrete has entrained air voids, and the air content is estimated to be at a normal level; the hydration of the cement was reasonable; and the presence of unhydrated cement particles was observed in the cement paste; fly ash particles were also found in the concrete; cracks of microscopical scale were observed.

2. Laboratory Procedures

Petrographic examination of the concrete samples was performed in accordance with ASTM C 856, "Standard Practice for Petrographic Examination of Hardened Concrete." Sections were polished and examined using a stereomicroscope at magnifications up to 200×. Small rectangular blocks were cut from concrete samples. One surface of each block was polished, dried, placed on petrographic slide with low−viscosity epoxy resin, and reduced to a thickness of approximately 25 micrometers (0.001 in.). These thin sections were examined using a polarized−light microscope at magnification up to 400×, to determine aggregate mineralogy, paste characteristics, and microstructure.

Two ¾ inch thick slabs were sawn from the concrete cores. The slabs were ground and polished to obtain a smooth and plane surface. They were examined using a stereomicroscope at magnifications up to 200×.

3. Findings

Six thin section samples taken from the received cores were examined using the polarized light microscope. Two polished sections were examined using the stereomicroscope. The findings are summarized as follows:

Aggregate

The coarse aggregate in the concrete is predominantly crushed limestone, with a small portion of gravel granite and quartzite. Most coarse aggregate particles are angular, and the maximum size is about 1/2 inch. Preferential orientation of coarse aggregate particles is not observed in this concrete.

The fine aggregate fraction is natural sand and mainly composed of quartz, with a small portion of feldspar, granite, quartzite, and limestone. The fine aggregate particles appear rounded to angular.

Cement Paste

The cement is reasonably hydrated and the paste contains some unhydrated cement particles as seen under the microscope (Figure E1-2). A small amount of fly ash particles is also present in the paste matrix (Figure E1-3).

Air Voids

Small, spherical air voids are observed in the concrete (Figure E1-4), hence the concrete was air entrained. Entrained air voids are well distributed in the concrete. The air content is estimated to be at a normal level. Entrapped air voids are also present in the concrete, as shown in Figure E1-5. The amount of entrapped air content is estimated to be slightly higher than normal level.

Cement-Aggregate Bonding

The paste/aggregate bond appears to be good.

Cracking

Two cores (NE#3 and NE#4) are split into halves by cracking. Major cracks are also visible in cores NE#2 and NE#5. These cracks run through coarse aggregate particles as well as in the cement paste.

Examination of thin section specimens revealed that cracks of microscopical scale are present in the concrete. Figure E1-6 shows a crack in the cement paste, while Figure E1-7 shows several cracks connecting with each other, forming a network. Figure E1-8 shows a crack along a fine aggregate periphery. Cracks can also be found in the polished concrete samples, as shown in Figure E1-9.

Secondary Deposit

No secondary deposit was found in the concrete.

4. Summary

The concrete is air entrained and the air content is estimated to be at a normal level. The entrained air voids are well distributed in the concrete. The entrapped air content is estimated to be slightly above normal level. Cement was reasonably hydrated and unhydrated cement particles are present in the concrete. Fly ash is also found in the concrete. The bond between the aggregate and paste is good.

Major cracks are visible in two of the six cores, and two cores were split into halves by cracking. Smaller size cracks are also found in the concrete samples. They exist in cement paste as well as at the interfacial region between the aggregate and paste. It is speculated that shrinkage may be the cause of the cracking.

Figure E1-1: Six concrete cores as received.

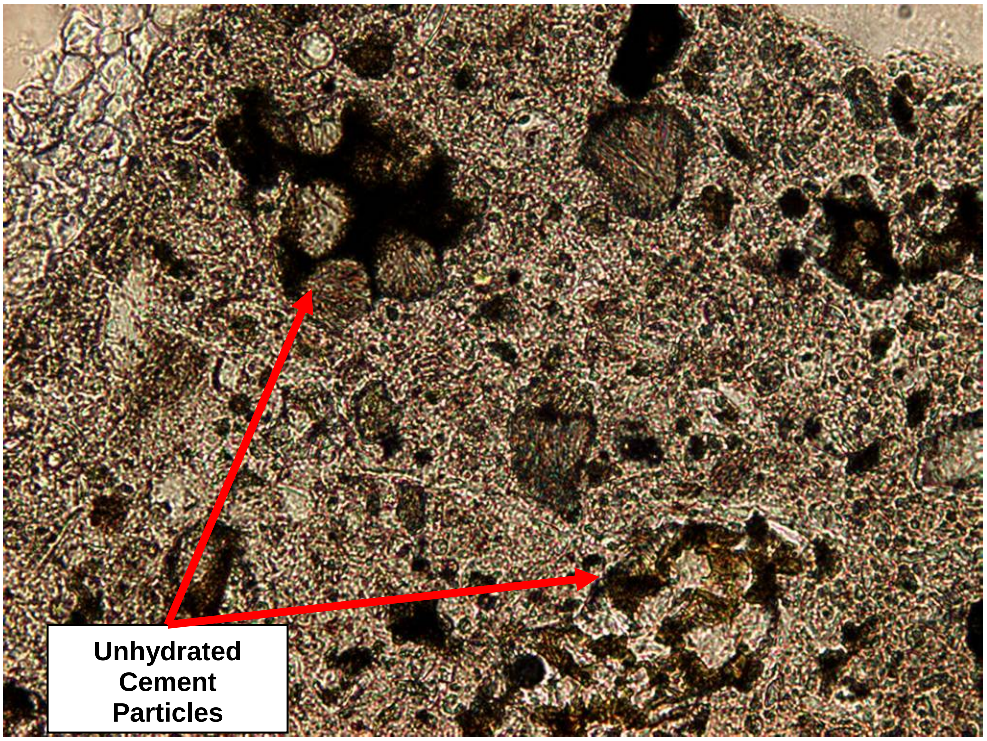


Figure E1-2: Unhydrated cement particles in the paste. Width of field is 0.33 mm, thin section image.

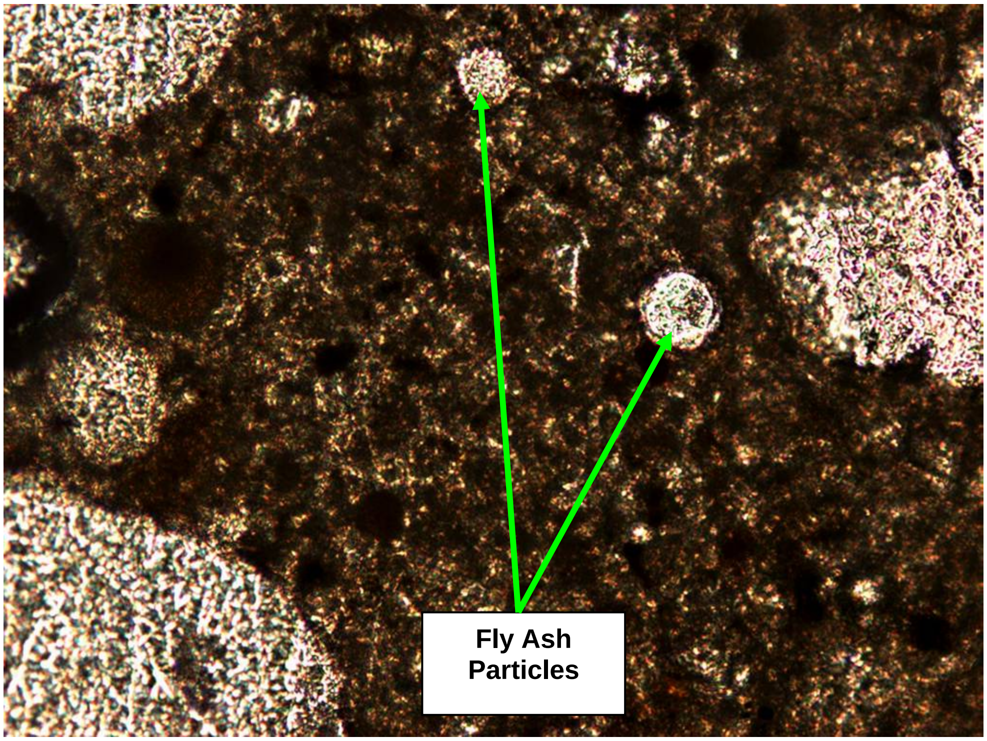


Figure E1-3: Fly ash particles in the cement matrix. Width of field is 0.33 mm, thin section image.

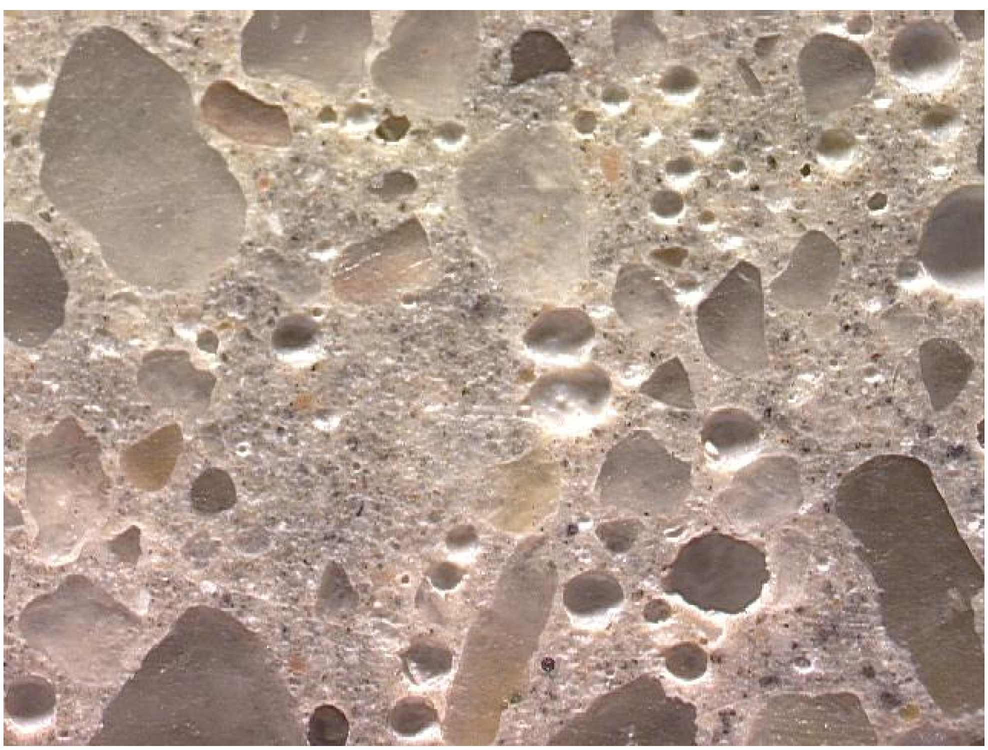


Figure E1-4: Entrained air voids in the concrete. Width of field is 4.0 mm, polished surface image.

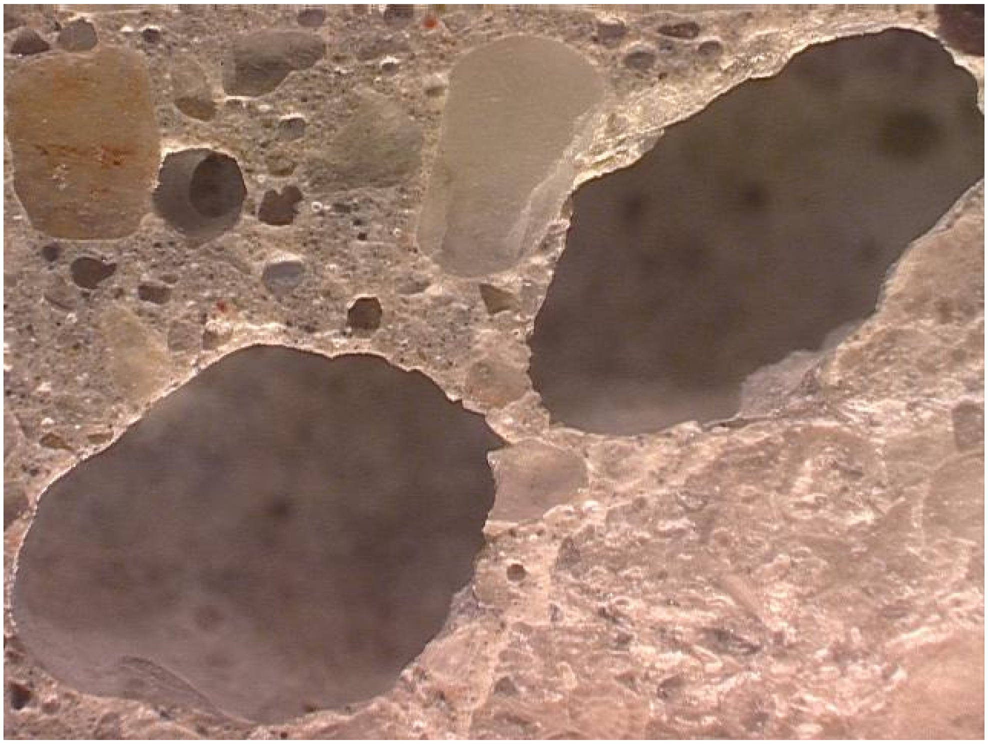


Figure E1-5: Two entrapped air voids. Width of field is 6.5 mm, polished surface image.

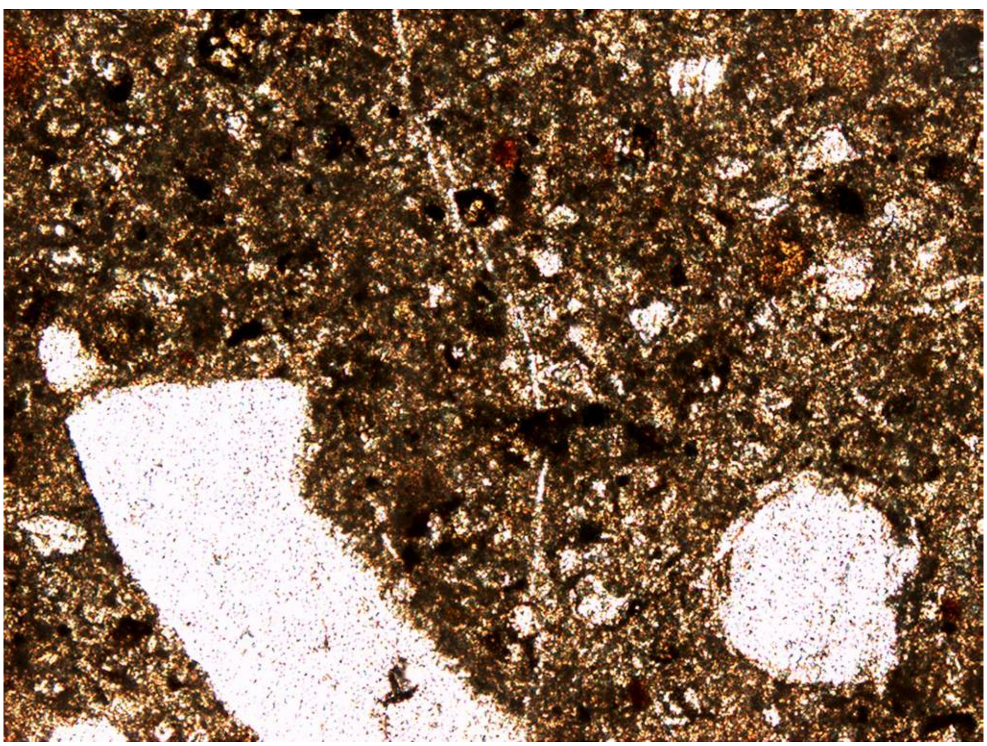


Figure E1-6: A crack in the cement paste. Width of field is 0.65 mm, thin section image.

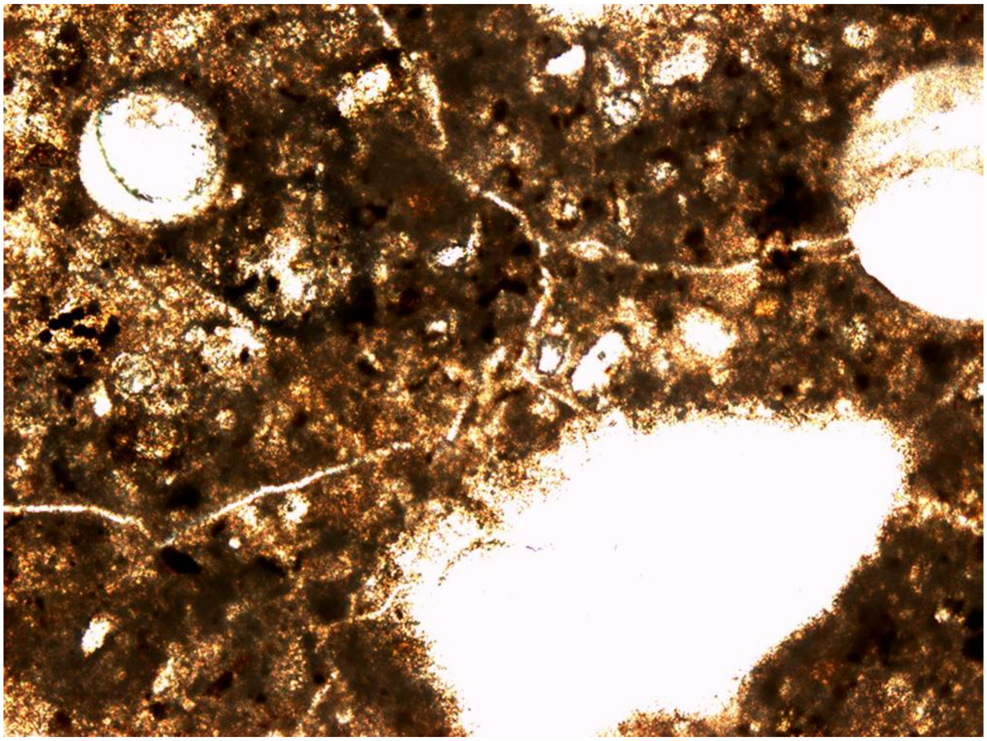


Figure E1-7: Cracks form a network. Width of field is 0.65 mm, thin section image.

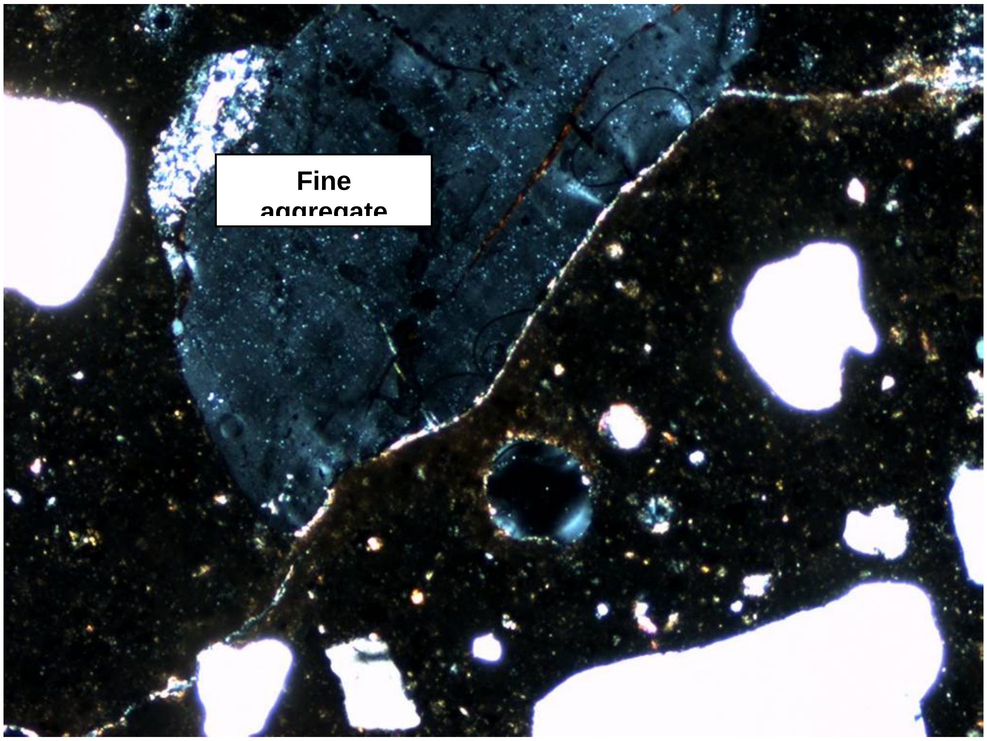


Figure E1-8: A crack at the interfacial region between a fine aggregate and the paste. Width of field is 1.6 mm, thin section image.

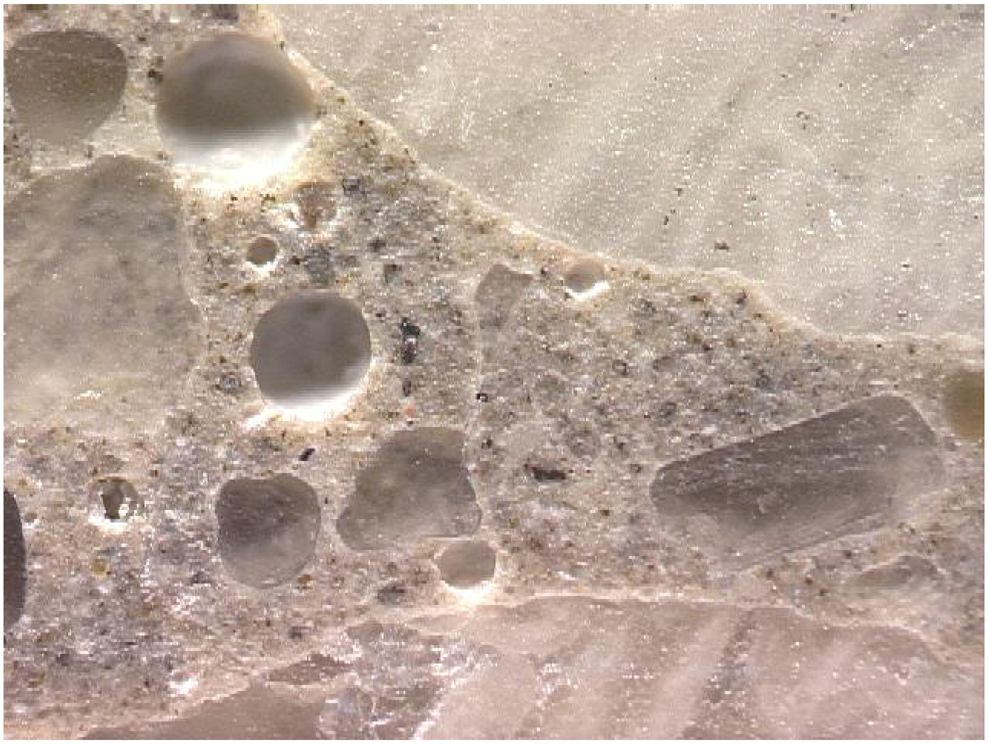


Figure E1-9: A crack in the paste between two aggregate particles. Width of field is 2.0 mm, polished surface image.

APPENDIX E – Supplement 2

120th Street and Giles Road Bridge near Omaha, Nebraska
Survey Checklist

Checklist

The following checklist is adapted from 201.1R-92, and provided to facilitate a thorough survey. The definition of terms and associated photographs in 201.1 R-92, are utilized to standardize the reporting of the condition of the concrete in the structures.

1. Description of structure or pavement
   - 1.1 Name, location, type, and size
     120th Street and Giles Road Bridge in Sarpy County, near Omaha, Nebraska opened to traffic in July 1997. It is a three-span structure 225 ft long. The clear width of the deck is 84.7 ft, including two through-traffic lanes, two shoulders, and a concrete median.
   - 1.2 Owner, project engineer, contractor, when built
     Owner - City of La Vista. This bridge is part of a demonstration project for HPC in bridge structures which were co-sponsored by the Federal Highway Administration (FHWA) and the Nebraska Department of Roads.
   - 1.3 Design
     - 1.3.1 Architect and/or engineer: the Nebraska Department of Roads.
     - 1.3.2 Intended use and history of use: To carry high volume of traffic on 120th street and Giles road. Opened to traffic in July 1996.
     - 1.3.5 Special features: HPC girder with specified strength of 12,000 psi at 56 days was used. Strength of 8,000 psi (55 MPa) at 56-days and a chloride penetration of less than 1800 coulombs at 56 days were specified for bridge deck concrete.
   - 1.4 Construction
     - 1.4.1 Contractor-general Hawkins Construction Company
     - 1.4.2 Subcontractors concrete placement: NA
     - 1.4.3 Concrete Supplier: Ready Mixed Concrete Company of Omaha.
     - 1.4.4 Agency responsible for testing: the Nebraska Department of Roads and University of Nebraska at Omaha Center for Infrastructure Research.
     - 1.4.5 Other subcontractors: NA
   - 1.5 Photographs
     - 1.5.1 General view Photos 1 through 3
     - 1.5.2 Detailed close up of condition of area Photos 4 through 11
   - 1.10 Sketch map-orientation showing sunny and shady and well and poorly drained regions N/A

2. Present condition of structure Date of Evaluation: The week of June 28, 2004
   - 2.1 Overall alignment of structure No signs of misalignment
     - 2.1.1 Settlement
     - 2.1.2 Deflection
     - 2.1.3 Expansion
     - 2.1.4 Contraction

2.2 Portions showing distress (beams, columns, pavement, walls, etc., subjected to strains and pressures) None Observed

2.3 Surface condition of concrete

2.3.1 General (good, satisfactory, poor, dusting, chalking, blisters) Good

2.3.2 Cracks Longitudinal, transverse, and diagonal

2.3.2.1 Location and frequency See Figure 2

2.3.2.8 Type and size (see Definitions) See Figure 2

Transverse Observed

Width (from Crack comparator) Hairline

- Hairline (Less than 1/32 in.)
- Fine (1/32 in. - 1/16 in.)
- Medium (1/16 – 1/8 in.)
- Wide (Greater than 1/8 in.)

Craze N/A

Width (from Crack comparator) ____________

- Hairline (Less than 1/32 in.)
- Fine (1/32 in. - 1/16 in.)
- Medium (1/16 – 1/8 in.)
- Wide (Greater than 1/8 in.)

Map N/A

Width (from Crack comparator) ________________

- Hairline (Less than 1/32 in.)
- Fine (1/32 in. - 1/16 in.)
- Medium (1/16 – 1/8 in.)
- Wide (Greater than 1/8 in.)

D-Cracking N/A

Width (from Crack comparator) ____________

- Hairline (Less than 1/32 in.)
- Fine (1/32 in. - 1/16 in.)
- Medium (1/16 – 1/8 in.)
- Wide (Greater than 1/8 in.)

Diagonal NA

Width (from Crack comparator) NA

- Hairline (Less than 1/32 in.)
- Fine (1/32 in. - 1/16 in.)
- Medium (1/16 – 1/8 in.)
- Wide (Greater than 1/8 in.)

2.3.2.9 Leaching, stalactites N/A

2.3.3 Scaling N/A

2.3.3.1 Area, depth ____________________

2.3.3.5 Type (see Definitions) ____________________

- Light (Less than 1/8 in.)
- Medium (1/8 in. – 3/8 in.)
- Severe (3/8 in. – 3/4 in.)
- Very Severe (Greater than 3/4 in.)

2.3.4 Spalls and popouts None observed
- 2.3.4.1 Number, size, and depth NA
- 2.3.4.5 Type (see Definitions) NA
  - Spalls
    - Small (Less than 3/4 in. depth)
    - Large (Greater than 3/4 in. depth)
  - Popouts
    - Small (Less than 3/8 in. diameter)
    - Medium (3/8 in. – 2 in. diameter)
    - Large (Greater than 2 in. diameter)

2.3.5 Extent of corrosion or chemical attack, abrasion, impact, cavitation N/A

2.3.6 Stains, efflorescence None observed

2.3.7 Exposed reinforcement None

2.3.8 Curling and warping N/A

2.3.9 Previous patching or other repair N/A

2.3.10 Surface coatings N/A
- 2.3.10.1 Type and thickness N/A
- 2.3.10.2 Bond to concrete N/A
- 2.3.10.3 Condition N/A

2.3.11 Abrasion N/A

2.3.12 Penetrating sealers ___
- 2.3.12.1 Type N/A
- 2.3.12.2 Effectiveness N/A
- 2.3.12.6 Discoloration N/A

2.4 Interior condition of concrete (in situ and samples) N/A

2.4.1 Strength of cores
2.4.2 Density of cores
2.4.3 Moisture content
2.4.4 Evidence of alkali-aggregate or other reactions N/A
2.4.5 Bond to aggregate, reinforcing steel, joints N/A
2.4.6 Pulse velocity
2.4.7 Volume change
2.4.8 Air content and distribution
2.4.9 Chloride-ion content
2.4.10 Cover over reinforcing steel
2.4.11 Half-cell potential to reinforcing steel.
2.4.12 Evidence of reinforcement corrosion
2.4.13 Evidence of corrosion of dissimilar metals
2.4.17 Delaminations N/A
- 2.4.17.1 Location N/A
- 2.4.17.2 Number, and size N/A

2.4.15 Depth of carbonation
2.4.16 Freezing and thawing distress (frost damage)
2.4.17 Extent of deterioration
2.4.21 Aggregate proportioning, and distribution

3. Nature of loading and detrimental elements
   3.1 Exposure
      3.1.1 Environment (arid, subtropical, marine, freshwater, industrial, etc.) Industrial
      3.1.2 Weather-(July and January mean temperatures, Max 88ºF and min. 11ºF
      mean annual rainfall and 29.3-in
      months in which 60 percent of it occurs) May
      3.1.3 Freezing and thawing Significant
      3.1.4 Wetting and drying Significant
      3.1.8 Drying under dry atmosphere N/A
      3.1.6 Chemical attack-sulfates, acids, chloride N/A
      3.1.7 Abrasion, erosion, cavitation, impact N/A
      3.1.8 Electric currents N/A
      3.1.9 Deicing chemicals which contain chloride ions N/A
      3.1.10 Heat from adjacent sources N/A
   3.2 Drainage N/A
      3.2.1 Flashing ____
      3.2.2 Weepholes ____
      3.2.3 Contour ____
      3.2.4 Elevation of drains ____
   3.3 Loading Research Test Data Available in Compilation CD Version 3
      3.3.1 Dead ____
      3.3.2 Live ____
      3.3.3 Impact ____
      3.3.4 Vibration ____
      3.3.5 Traffic index ____
      3.3.6 Other ____
   3.4 Soils (foundation conditions) ____
      3.4.1 Compressibility
      3.4.2 Expansive soil
      3.4.3 Settlement
      3.4.4 Resistivity
      3.4.5 Evidence of pumping
      3.4.6 Water table (level and fluctuations)

4. Original condition of structure Good
   4.1 Condition of formed and finished surfaces Good
      4.1.1 Smoothness ____
      4.1.2 Air pockets ("bugholes") ____
      4.1.3 Sand streaks ____
      4.1.4 Honeycomb ____
      4.1.5 Soft areas (retarded hydration) ____
      4.1.6 Cold joints ____
      4.1.13 Staining ____

4.1.14	Sand pockets		________
4.2	Defects		N/A
4.2.1	Cracking		________
4.2.1.1		Plastic shrinkage	________
4.2.1.2		Thermal shrinkage	________
4.2.1.3		Drying shrinkage	________
4.2.5	Curling		________

5. Materials of Construction — See Table 1

6. Construction Practices — See Report pg. 3, 7, 8

APPENDIX F

Route 104 Bridge, New Hampshire

HIGH PERFORMANCE CONCRETE BRIDGE DECK INVESTIGATION

The Route 104 Bridge in Bristol, New Hampshire

I. BACKGROUND

The Route 104 Bridge over the Newfound River in Bristol, New Hampshire, was the first HPC bridge deck project built in New Hampshire. It was completed in summer 1996 and opened to traffic thereafter. HPC was used for the girders and the cast-in-place deck in the Route 104 Bridge.

The Route 104 Bridge is a simple-span structure about 65-ft long. The clear width of the deck is 57.5 ft, including two through-traffic lanes, a shoulder, and a right-turn lane. The 9-in. thick cast-in-place deck is supported by five prestressed Type III AASHTO I-girders separated at 12.5 ft on center. For the beams at transfer, the design strength was 8000 psi at 28 days. The designed deck concrete was specified to have strength of 6000 psi at 28 days.

The Route 104 Bridge is part of a demonstration project for HPC in bridge structures, which are co-sponsored by the Federal Highway Administration (FHWA) and the New Hampshire Department of Transportation (NHDOT). University of New Hampshire undertook the research project to monitor the long-term behavior of HPC bridge. This bridge was built with many instruments within the concrete to measure concrete temperatures, elastic shortening, creep, shrinkage, and stresses of live and dead loads in the deck and girders.

The Route 104 Bridge is expected to demonstrate that HPC provides greater economy and greater durability with reduced long-term maintenance. Following the success of the Route 104 Bridge, New Hampshire Department of Transportation (NHDOT) decided to construct another HPC bridge – the Route 3A Bridge over the Newfound River in Bristol, New Hampshire, about one mile away from the Route 104 Bridge.

II. SCOPE OF SERVICES

Professional Service Industries (PSI) is under contract with the Federal Highway Administration (FHWA) to conduct bridge deck inspections for HPC bridges. Our scope of services on this bridge included a series of tasks and sub-tasks, which are described as follows:

- Collect all available information relevant to the construction of each bridge, including:
    - Deck Concrete Properties
    - Specified Deck Concrete Construction Procedures
    - Approved Concrete Mix Proportions
    - Measured Properties from QC

    - Other Measured Properties
    - Actual Method of Deck Placement
    - Average Daily Traffic (ADT)
    - Exposure Condition of the Bridge
    - Any Performed Maintenance
    - Any Inspection Reports
- Visually inspect the bridge, and obtain the following:
    - General condition of the deck top surface
    - Determination of the maximum crack width
    - General condition of the deck underside
    - General condition of the girders
    - Photograph areas of significant deterioration
    - Prepare drawings locating defects
    - Extract 6 concrete core samples

III. COMPILATION OF BRIDGE CONSTRUCTION INFORMATION

Information sources used for this report include bridge drawings, inspection reports, bridge summary data sheets, and technical information contained in FHWA's "Compilation and Evaluation of Results from High Performance Concrete Bridge Projects" version 3.

Deck Concrete Properties

The bridge deck concrete had a specified compressive strength of 6000 psi (41 MPa) at 28 days. Maximum water-to-cementitious materials ratio of 0.38 was specified. For adequate protection against the likelihood of freeze-thaw cycles, the air content was specified to be 6-9%. Table 1 lists the specified concrete properties for the cast-in-place decks.

TABLE 1: Specified Concrete Properties

Property	Deck
Cement type:	Type II
Minimum Cementitious Materials Content:	658 lb/yd^3
Max. Water/Cementitious Materials Ratio:	0.38
Min. Percentage of Silica Fume:	7.5%
Air Content:	6-9%
Slump:	2-3 in.
Compressive Strength - Design:	6000 psi @ 28 days
Chloride Permeability:	≤1000 Coulombs at 56 days
Other:	Corrosion inhibitor at 4 gal/yd^3 in deck Type II cement

Specified Deck Concrete Construction Procedures

For the Route 104 Bridge, the concrete supplier was required to mix several trial batches of concrete to determine an acceptable mixture design. Each trial batch was tested for slump, air content, concrete temperature, and unit weight. Research conducted by University of New Hampshire found that one of the three trial batches gave superior durability performance in terms of freeze-thaw, scaling, abrasion, and moment capacity.

Once the NHDOT approved the mixture design, a 5 yd^3 trial pour was initiated so that the actual placing, finishing, and curing conditions could be evaluated. Such procedures allowed for the fine-tuning of the admixture dose and for the testing of the equipment needed for placement. Typically, NHDOT specifies protecting concrete decks with a barrier membrane and an asphalt overlay. However, for the Route 104 Bridge, a corrosion inhibitor was required and no asphalt overlay was used. The deck was specified to be wet-cured with cotton mats for four days.

Approved Concrete Mix Proportions

Deck

The approved proportions for the cast-in-place deck are shown in Table 2. Note that the selected mix design was chosen based on performance testes during trial batching.

TABLE 2: Approved Mix Proportions for the Route 104 Bridge

Mix Parameters	Cast-in-Place Deck
Cement Brand:	Ciment Quebec
Cement Type:	II
Cement Quantity:	660 lb/yd^3 *2
Silica Fume Brand:	Ciment Quebec
Silica Fume Quantity:	8 %
Fine Aggregate Quantity:	1190 lb/yd^3
Fine Aggregate FM:	2.8
Fine Aggregate SG:	2.66
Coarse Aggregate, Max. Size:	¾ in.
Coarse Aggregate Type:	No. 67 stone
Coarse Aggregate SG:	2.69
Coarse Aggregate Quantity:	1815 lb/yd^3
Water:	253 lb/yd^3
Water Reducer Brand:	WRDA with HYCOL
Water Reducer Type:	A
Water Reducer Quantity:	20 fl oz /yd^3
High Range Water Reducer Brand:	Daracem 100
High Range Water Reducer Type:	F and G
High Range Water Reducer Quantity:	79 fl oz /yd^3
Air Entrainment Brand:	Daravair 1000
Air Entrainment Type:	Saponified rosin
Air Entrainment Quantity:	6 fl oz /yd^3
Corrosion Inhibitor Brand:	DCI S
Corrosion Inhibitor Type:	Calcium nitrate
Corrosion Inhibitor Quantity:	4 gal/yd^3
Water/Cementitious Materials Ratio:	0.38

NOTES:
1. Mix designs were recommended by the University of New Hampshire. Minor changes were made for the approved mix.
2. Cement and silica fume were pre-blended. Total cementitious materials are 660 lb/yd^3.

Measured Properties of the Approved Concrete Mix

The approved concrete mix had a slump of 5-7 in. and an air content of 6-9%, as indicated in Table 3.

TABLE 3. Measured Properties of the Approved Concrete Mix for the Route 104 Bridge

Property	Value
Slump	5-7 in.
Air Content	6-9%

Measured Properties from QC Tests of Production Concrete

Cast-in-Place Deck

Measured properties of the deck concrete mixture from QC tests are summarized in Table 4. Air content and slump are somewhat lower than those values for the approved concrete mix (Table 3).

TABLE 4: Measured Properties of QC Tests of the Production Concrete Mixes for the Cast-in-Place Deck

Property	Value
Actual curing procedure for the deck:	Dry cotton mats were placed within 15 minutes of the burlap drag. The mats were then wetted down and the deck wet cured for about 136 hours
Slump:	3-5 in.
Air Content:	4.0-5.8 %
Unit Weight:	144-147 lb/ft^3
Compressive Strength:	5700 psi at 3 days 6890 psi at 7 days 7060 psi at 14 days 7810 psi at 21 days 9020 psi at 28 days 9600 psi at 56 days
Cylinder Size:	6x12 in.

Table 5 lists the composition of the Type II cement and the Type II cement with silica fume used in the mixture. Note that the addition of silica fume increases the percentage of silicon dioxide in the cementitious materials, as expected.

TABLE 5: Composition of Cement Used in QC Tests of the Production Concrete for the Deck

Component	Deck	
	w/o Silica Fume (%)	w/ Silica Fume (%)
Silicon dioxide (SiO_2)	21.5	27.18
Aluminum oxide (Al_2O_3)	4.9	4.40
Ferric oxide (Fe_2O_3)	3.1	2.67
Calcium oxide (CaO), Total	63.7	59.18
Calcium oxide (CaO), Free	0.7	—
Sulfur trioxide (SO_3)	2.9	2.96
Magnesium oxide (MgO)	2.4	2.18
Alkali equivalent (Na_2O)	0.8	—
Potassium monoxide (K_2O)	—	0.98
Strontium oxide (SrO)	—	0.18
Manganese sesquioxide (Mn_2O_2)	—	0.04
Zinc oxide (ZnO)	—	0.05
Chromium sesquioxide (Cr_2O_3)	—	0.02
Loss on ignition	0.7	0.84
Insoluble residue	0.3	3.44
Tricalcium aluminate (C_3A)	7.6	7.15
Tetracalcium aluminoferrite (C_4AF)	9.6	—
Tricalcium silicate (C_3S)	50.9	—
Dicalcium silicate (C_2S)	23.1	—

Measured Properties from Research Tests of Production Concrete for the Deck

Compressive Strength and Modulus of Elasticity

The compressive strength tests on the production concrete showed that the concrete used for the Route 104 Bridge had 28-day strengths greater than 8000 psi (Table 6), which are well above the specified strength of 6000 psi.

Freeze-Thaw Resistance and Chloride Permeability

The mixture showed excellent freeze-thaw resistance and chloride permeability, much lower than the specified value of 1000 coulombs (Table 7).

TABLE 6: Measured Compressive Strength and Modulus of Elasticity from Research Tests of Production Concrete for the Deck

Source	Age, days	Compressive Strength, psi	Modulus of Elasticity, ksi
UNH [1]	1.26	3360	—
	3.18	5700	—
	7.30	6430	3750
	14.25	7590	4200
	28.1	8580	4250
	56.1	9380	4150
	122.1	9750	4500
	365	9850	4450
Contractor [2]	7	6510	—
	14	7340	—
	28	8310	—

NOTES:
(1) UNH specimens were 4x8-in cylinders cured in accordance with ASTM C 31 Standard Cure.
(2) Contractor's specimens were 6x12-in cylinders.

TABLE 7: Measured Freeze-Thaw Resistance and Chloride Permeability from Research Tests of Production Concrete for the Deck

Sample	Freeze-Thaw Resistance [1], %	Chloride Permeability [2], coulombs
1	99	609
2	97	896
3	97	—
4	96	—
Average	97	753

NOTES:
(1) Test followed AASHTO T 161 Procedure A, tested at a concrete age of 140 days.
(2) Test followed ASTM C 1202, tested at 56 days on cores from the deck.

Deicer Scaling

Scaling specimens were cured in a manner identical to deck curing. The tests began at concrete age of 30 days and continued to 100 cycles. No scaling was found up to 50 cycles. The deicer scaling resistance was rated as 0 to 1.

Actual Method of Deck Placement

Construction of the Route 104 Bridge decks occurred in 1996. Concrete was delivered by truck and was pumped into the forms for easy placement. The end of the hose on the pump was placed horizontally during pumping to limit the loss of air content. A standard, self-propelled finishing machine was implemented to strike off the top surface. In areas adjacent to the curb line, hand-finishing was performed. Attached behind a screed were a finishing pan and burlap drag, they were used to simultaneously finish and texture the surface. Specifications strongly discouraged over-finishing and bull floating.

Within 15 minutes after finishing and texturing, a section was covered with dry cotton mats and then wetted. The mats were kept wet for four days. The rapid placement of the mats reduced surface evaporation and eliminated shrinkage cracking. The specifications regarding evaporation at the time of placement were strictly enforced. If the evaporation rate was greater than 0.1 lb/ft^2/hr or if the ambient temperature was above 85^{o}F (29^{o}C), no placing of concrete was allowed. The hardened finish of the deck was transversely saw-cut on 1.5 in. centers with 0.125-in. wide and 0.25-in. deep grooves.

While every effort was made to ensure consistency and uniformity in the fresh concrete properties, there were some difficulties in maintaining the required air content and a consistent slump. In fact, more superplasticizer was added to the mixture on site to obtain the desired workability. It is possible that an interaction between the corrosion inhibitor and the other admixtures produced the inconsistent air content and slump measurements.

Average Daily Traffic (ADT) and Average Daily Truck Traffic (ADTT)

Average daily traffic for both eastbound and westbound lanes was calculated based on a count of all vehicles crossing the bridge during a 15 minute period beginning at 1413 hrs on May 26, 2004. The westbound ADT was 4,608, including 4,320 cars and 288 trucks per day. The eastbound ADT was 5,568, including 5,280 cars and 288 trucks per day. The estimation of traffic flow was made while the PSI inspection crew was on-site.

Exposure Condition of the Bridge

Based on climatology, the Route 104 Bridge in Bristol, NH experiences a wide range of conditions throughout the year. The mean daily maximum temperatures for Concord, NH (about 37 miles south of Bristol) range from a low of 29.8^{o}F in January to a high of 82.4^{o}F in July. Mean daily minimum temperatures in Concord vary between 7.4^{o}F in January and 56.5^{o}F in July. In Lebanon, NH, 37 miles to the west of Bristol, the mean daily maximum temperatures range from 28.0^{o}F in January to 81.2^{o}F in July. Lebanon's mean daily minimum temperatures range from 5.7^{o}F in January to 56.8^{o}F in July. The Bristol area in central NH experiences about 173 days per year in which air temperatures drop below 32^{o}F, implying a considerable number of freeze-thaw cycles. Direct measurements of freeze-thaw cycles (defined as a drop in temperature below 28^{o}F followed by a rise above 32^{o}F) from November 1996 through April 1997 showed an average of 8 freeze-thaw cycles per month, with a maximum of 15 in March 1997. The possibility of below freezing temperatures and the fact that Concord and Lebanon receive on average about 64 and 76 inches of snow per year, respectively, suggests that the roads are treated for ice and snow. Central New Hampshire receives an average monthly precipitation total of between 2.4 inches in January and 3.7 inches in July, with an annual average total of about 36 inches. Considerable precipitation throughout the year implies that the bridge experiences many wet/dry cycles.

Performed Maintenance

No documents were found which would indicate any maintenance had been performed since the bridge was constructed.

Inspection Reports

University of New Hampshire and the New Hampshire Department of Transportation (NHDOT) undertook the research project to monitor the long-term behavior of Route 104 HPC Bridge. Several reviews of the bridge performance until 2000 showed some microscopic longitudinal flexural cracks over the girder lines, but no transverse or shrinkage cracks were found. Also, there was no scaling and no freeze-thaw damage.

The bridge was tested with a live load just before opening. The truck weighed 88,000 lb and deflections at various locations on the bridge were measured. No reports of conclusions from these tests have been found.

IV. BRIDGE DECK INSPECTION

PSI personnel performed a visual inspection of the bridge decks during the week of May 24, 2004. The results of the inspection are summarized as follows.

General Condition of the Deck Top Surface

Figure 1 shows the general layout of the decks for the Route 104 Bridge. Results of visual inspection of the decks are shown in Figure 2. Surface defects observed and documented during visual inspection primarily are longitudinal cracks (see photo 4). Apparent signs of other serious damages such as freeze-thaw, D-cracking, alkali-silica reaction, and alkali-aggregate reaction were not observed.

Transverse Cracks: No transverse cracks were observed on the deck.

Diagonal Cracks: No diagonal cracks were observed on the deck.

Longitudinal Cracks: A total of 2 longitudinal cracks were recorded during visual survey of the bridge decks (see Figure 2). The sum of crack lengths was 10 ft over a bridge deck area of 3,217.5 ft^2. Crack density (total crack length / deck area) for the eastbound and westbound bridges combined was calculated to be 0.003 ft/ft^2.

It is noted that the longitudinal cracks are hairline crack with a width of less than 0.020 in. (0.5 mm). Longitudinal crack pattern is shown in photo 4. Cracks were typically limited at span ends. Figure 2 also illustrates the locations of drilled cores. Table 8 lists the details of measured longitudinal cracks.

TABLE 8: Measured Cracks on the Bridge Decks

Crack Type	Count	Length Range (feet)	Mean Length of Cracks (feet)	Median Length of Cracks (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Longitudinal	2	4 to 6	5	5	10	3217.5	0.003

Maximum Crack Width

The maximum width of transverse cracks was measured to be 0.020 in. According to ACI 201, these crack widths are classified as hairline cracks.

General Condition of the Deck Underside

The underside of the deck exhibits no signs of distress, as illustrated by photos 5 and 6.

General Condition of the Girders

The girders were inspected without the aide of any access equipment. No signs of distress were noted on any of the girders.

Concrete Core Samples

Six cores, 3-¾ inches in diameter, were retrieved from the decks. The core sample locations are shown on Figure 2. The locations were evenly distributed along each shoulder of the bridge. The cores were labeled NH104-1 through NH104-6 and were transferred to FHWA for further analysis.

TABLE 9: Core Dimensions

Sample	NH104-1	NH104-2	NH104-3	NH104-4	NH104-5	NH104-6
Diameter (in.)	3¾	3¾	3¾	3¾	3¾	3¾
Length (in.)		3		3	3	3½

Preliminary Conclusions

The construction of the Route 104 Bridge is part of a demonstration project for HPC in bridge structures. Following the success of the Route 104 Bridge in Bristol, NH, New Hampshire Department of Transportation (NHDOT) decided to construct another HPC bridge – the Route 3A Bridge over the Newfound River in Bristol, New Hampshire, about one mile away from the Route 104 Bridge.

Researchers from University of New Hampshire performed material testing, bridge instrumentation, and bridge monitoring throughout this project. It was reported that several inspections have been conducted. Until year 2000 only some microscopic longitudinal flexural cracks over the girder lines were observed, but no transverse or shrinkage cracks were found. Also, there was no scaling and no freeze-thaw damage.

The visual inspection of the bridge decks as part of our study was performed about eight years after the bridge opened to traffic. Only 2 longitudinal cracks were recorded on the bridge with a combined total crack length of 10 ft over a bridge deck area of 3,217.5 ft^2. Crack density (total crack length / deck area) for the eastbound and westbound bridges combined was calculated to be 0.003 ft/ft^2. All cracks on the bridge were hairline cracks with a width of less than 0.031 in. No major distresses were observed in our bridge survey. Compared to data reported by the University of New Hampshire, which mentioned microscopic longitudinal cracks, it is believed that more cracks have not occurred to the Route 104 Bridge. Considering the heavy ADT on the bridge, the Route 104 Bridge was in excellent condition. HPC designs provide significantly higher strength that can lead to more efficient designs and improved durability.

Petrographic examination was performed on five concrete cores that were retrieved from the bridge. The identification on the cores is as follows: NH-104-1, NH-104-2, NH-104-4, NH-104-5, and NH-104-6. All of the cores showed evidence of being broken off, and not being drilled all the way through. Visual inspection of the concrete cores revealed no defects in the cores.

The coarse aggregate in the concrete was gravel, which was composed of granite, quartzite, andesite, and basalt. Coarse aggregate particles were rounded to angular, and the maximum size, measured from the examined concrete samples, was about 1 inch. Preferential orientation of coarse aggregate particles was not observed in this concrete, nor was segregation. The fine aggregate fraction was composed of quartz, quartzite, granite, feldspar, mica, and sandstone. The fine aggregate was from natural sand and the particles appeared rounded to angular.

The cement was well hydrated with respect to the age of the concrete. The cement paste contained some unhydrated cement particles. Ground granulated blast-furnace slag (GGBFS) particles were also present in the cement matrix. In general, bonding between the cement paste and the aggregate was strong.

The concrete was air entrained. Small, spherical air voids were well distributed in the concrete. The air content was estimated at a normal level. Ettringite crystals were observed in some air voids in the concrete.

Occasionally cracks were observed in the cement paste. Cracks were also found at the paste-aggregate interface. These cracks were mostly found in the surface region of about 1 in. from the exposed surface. It was speculated that the cracks were probably due to drying shrinkage, although other mechanisms might also contribute to the distress.

Respectfully Submitted,
Professional Service Industries, Inc.
Structural Investigation & Petrography Department

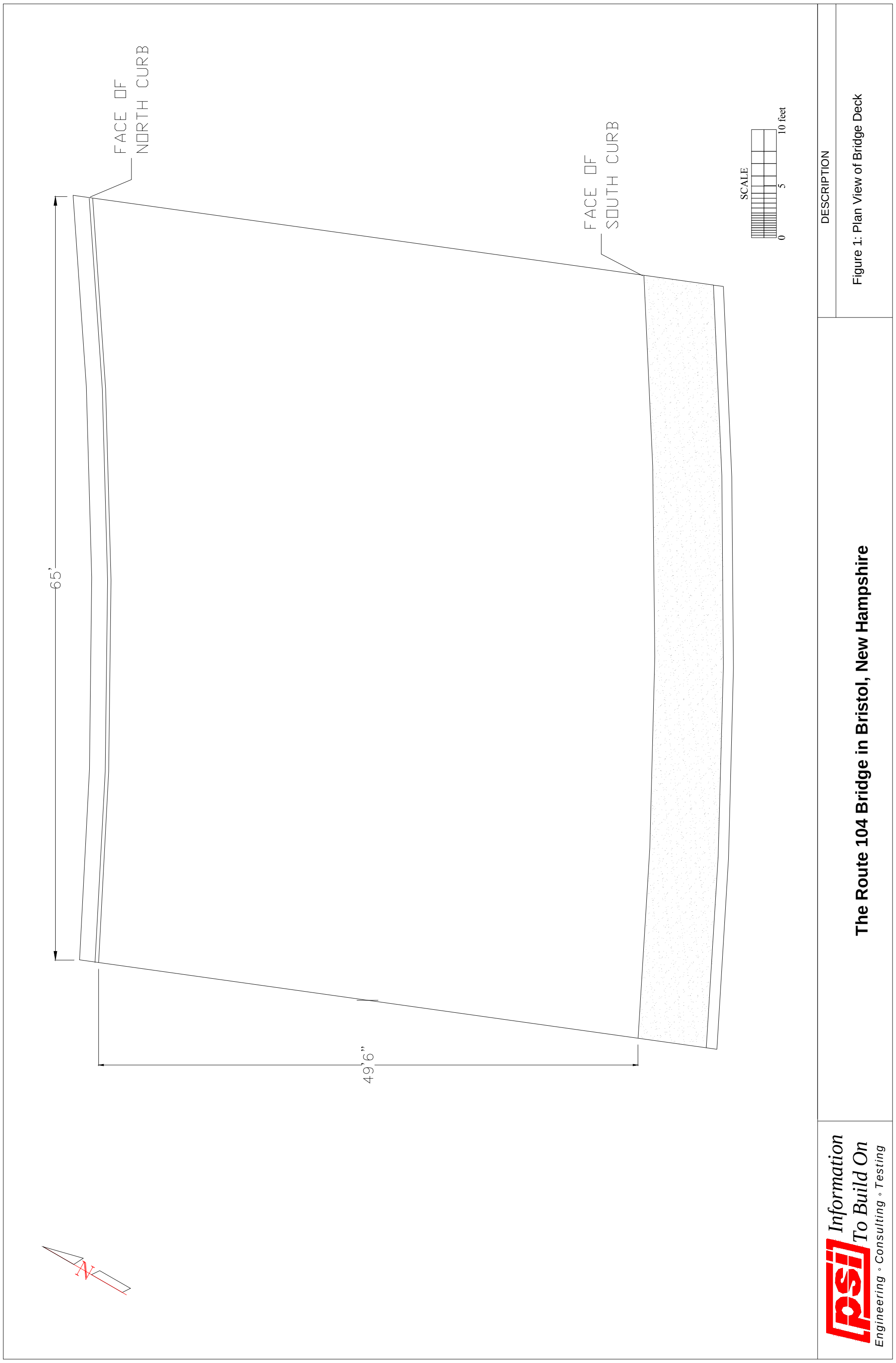
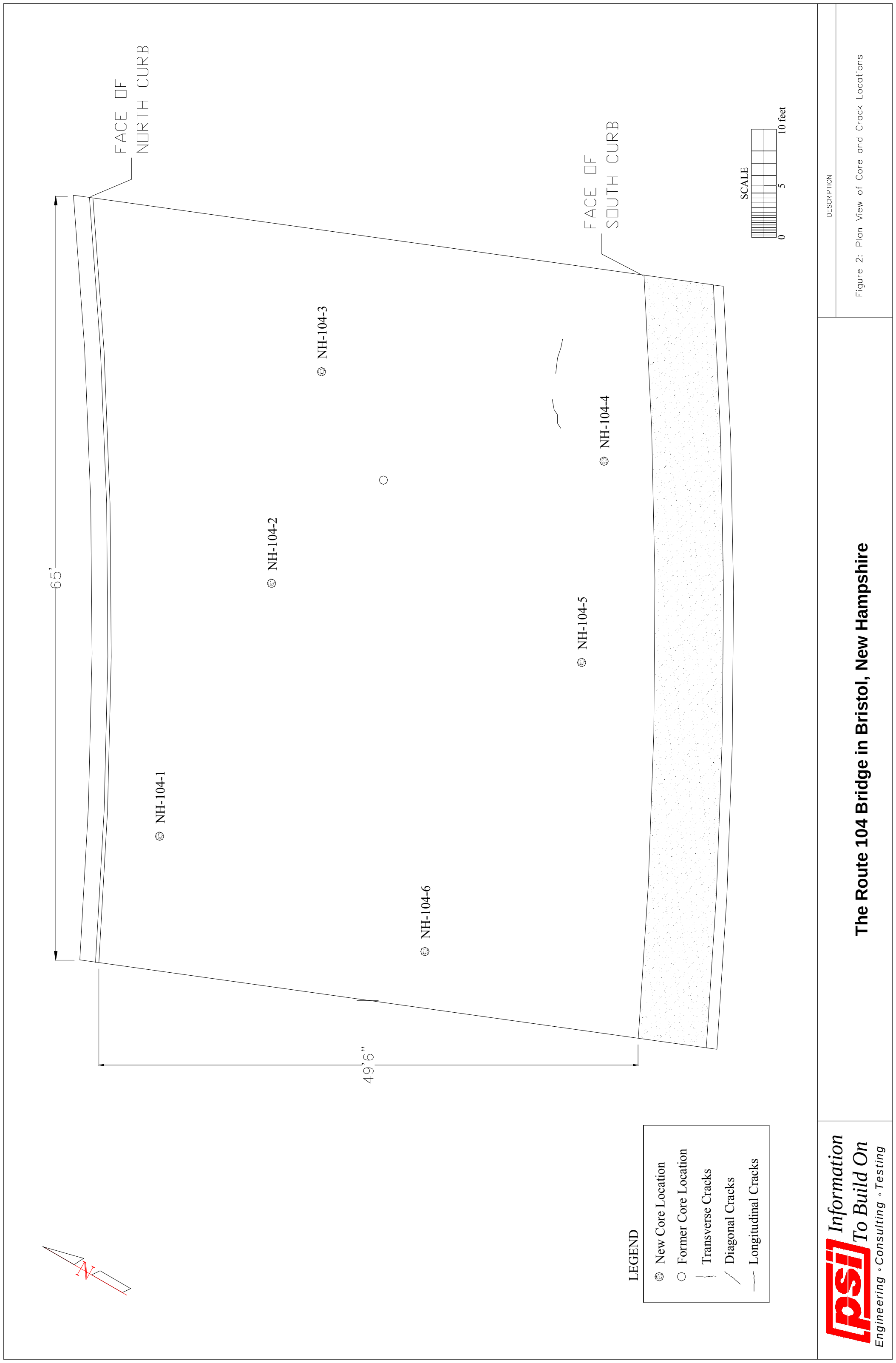


Figure 1: Plan View of Bridge Deck


FACE OF NORTH CURB
FACE OF SOUTH CURB
65'
49'6"
NH-104-1
NH-104-2
NH-104-3
NH-104-4
NH-104-5
NH-104-6
N
LEGEND
New Core Location
Former Core Location
Transverse Cracks
Diagonal Cracks
Longitudinal Cracks
SCALE
0
5
10 feet
DESCRIPTION
Figure 2: Plan View of Core and Crack Locations
The Route 104 Bridge in Bristol, New Hampshire
psi
Information To Build On
Engineering • Consulting • Testing

Photographic Documentation

	Photo 1: View looking south from the northern end of east facing bridge.
	Photo 2: View looking north from the southern end of east facing bridge.
	Photo 3: View looking north from the northern end of east facing bridge. Note the right-turn lane on bridge deck.

Photo 4: General view of longitudinal crack on bridge deck.

Photo 5: Close up view of bridge deck underside inspection using a boat.

Photo 6: Close up view of bridge deck underside pipe lines.

APPENDIX F – Supplement 1

Route 104 Bridge, New Hampshire
Petrographic Examination

PETROGRAPHIC EXAMINATION OF CONCRETE CORES FROM A BRIDGE IN NEW HAMPSHIRE (NH104)

A report submitted to the Structures Laboratory at TFHRC

Rongtang Liu
Petrographic Laboratory
FHWA TFHRC
(Reviewed by Richard Meininger, PE; Concrete Laboratory; printed in color 9-6-2006)

August 17, 2006

1. Introduction

Five concrete cores were received from the Structures Laboratory of the Turner-Fairbank Highway Research Center for petrographic examination. These cores were collected from a concrete bridge in New Hampshire. The identification on the cores is as follows: NH-104-1, NH-104-2, NH-104-4, NH-104-5, and NH-104-6.

Thin sections, as well as polished slabs, were made from the concrete cores for microscopic examination. Petrographic examination was performed on these samples using optical microscopes.

All of the cores showed evidence of being broken off, and not being drilled all the way through (Figure F1-1). Visual inspection of the concrete cores revealed no defects in the cores. The findings from microscopic examination indicate that the concrete has entrained air voids, and the air content is estimated as at a normal level; the hydration of the cement was reasonable; the presence of some unhydrated cement particles was observed in the cement paste; ground granulated blast-furnace slag was added as a supplementary cementitious material; cracks existed in the paste as well as in the aggregate peripheral zone; occasionally, ettringite as a secondary deposit formed in some of the air voids.

2. Laboratory Procedures

Petrographic examination of the concrete samples was performed in accordance with ASTM C 856, "Standard Practice for Petrographic Examination of Hardened Concrete." Sections were polished and examined using a stereomicroscope at magnifications up to 200×. Small rectangular blocks were cut from concrete samples. One surface of each block was polished, dried, placed on a petrographic slide with low–viscosity epoxy resin, and reduced to a thickness of approximately 25 micrometers (0.001 in.). These thin sections were examined using a polarized–light microscope at magnification up to 400×, to determine aggregate mineralogy, paste characteristics, and microstructure.

Two ¾ inch thick slabs were sawn from the concrete cores. The slabs were ground and polished to obtain a smooth and plane surface. They were examined using a stereomicroscope at magnifications up to 200×.

Findings

Six (6) thin section samples taken from the received cores were examined using the polarized light microscope. Two polished sections were examined using the stereomicroscope. The findings are summarized as follows:

Aggregates

The coarse aggregate in the concrete is gravel, which is composed of granite, quartzite, andesite, and basalt. Coarse aggregate particles are rounded to angular, and the maximum size is about 1 inch. Preferential orientation of coarse aggregate particles is not observed in this concrete, nor is segregation.

The fine aggregate fraction is composed of quartz, quartzite, granite, feldspar, mica, and sandstone. The fine aggregate is from natural sand and the particles appear rounded to angular.

Cement Paste

The cement is well hydrated and the paste contains some unhydrated cement particles as seen under the microscope (Figure F1-2). Silica fume particles, as shown in Figure F1-3, are present in the cement matrix.

Air Voids

Small, spherical air voids are observed in the concrete (Figure F1-4), hence the concrete was air entrained. Entrained air voids are well distributed in the concrete. The air content is estimated as at a normal level.

Cement-Aggregate Bonding

In general, bonding between the cement paste and the aggregate is strong, as shown in Figure F1-5.

Secondary Deposit

Ettringite is sporadically observed in some air voids in the concrete (Figure F1-6).

Cracking

Occasionally cracks are present in the cement paste, as shown in Figure F1-7 and Figure F1-8. Cracks are also found at the paste-aggregate interface (Figure F1-9). These cracks are mostly found in the surface region of about 1-in. from the exposed surface. It is speculated that the cracks are probably due to drying shrinkage, although other mechanisms may also contribute to the distress.

3. Summary

The concrete was air entrained, and the entrained air voids were well distributed in the concrete. The cement was well hydrated. Unhydrated cement particles, as well as ground granulated blast-furnace slag particles, are present in the paste. In general, the bond between the aggregate and the paste appears strong. Cracks exist in the cement paste as well as in the interfacial region between the paste and aggregate. It is speculated that shrinkage, among other mechanisms, may be the major cause of the cracking. Ettringite crystals have formed sporadically in air voids. It is common to see ettringite as secondary deposit in concrete.

Figure F1-1: Five cores as received.

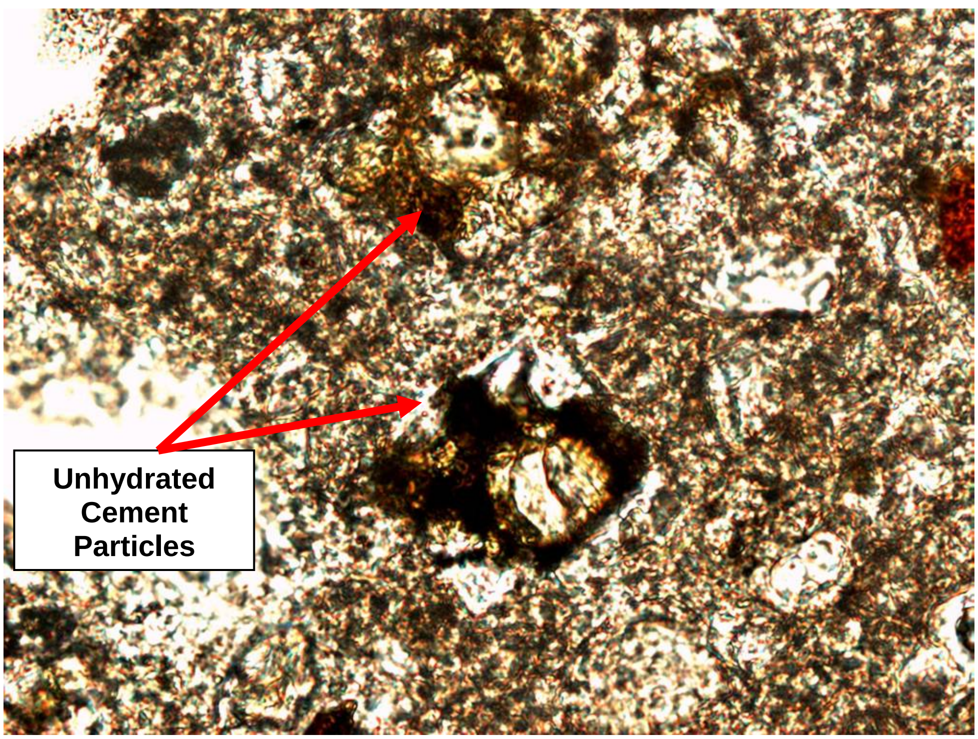


Figure F1-2: Unhydrated cement particles in paste. Width of field is 0.165 mm. Thin section image.

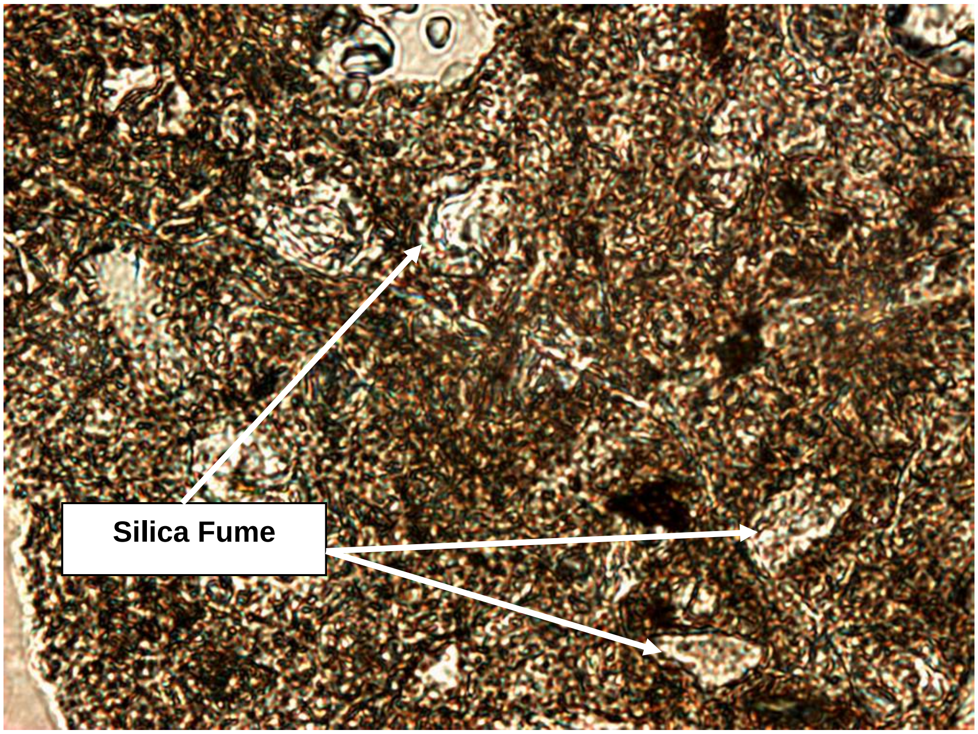


Figure F1-3: Silica Fume particles in the cement matrix. Width of field is 0.165mm. Thin section image.

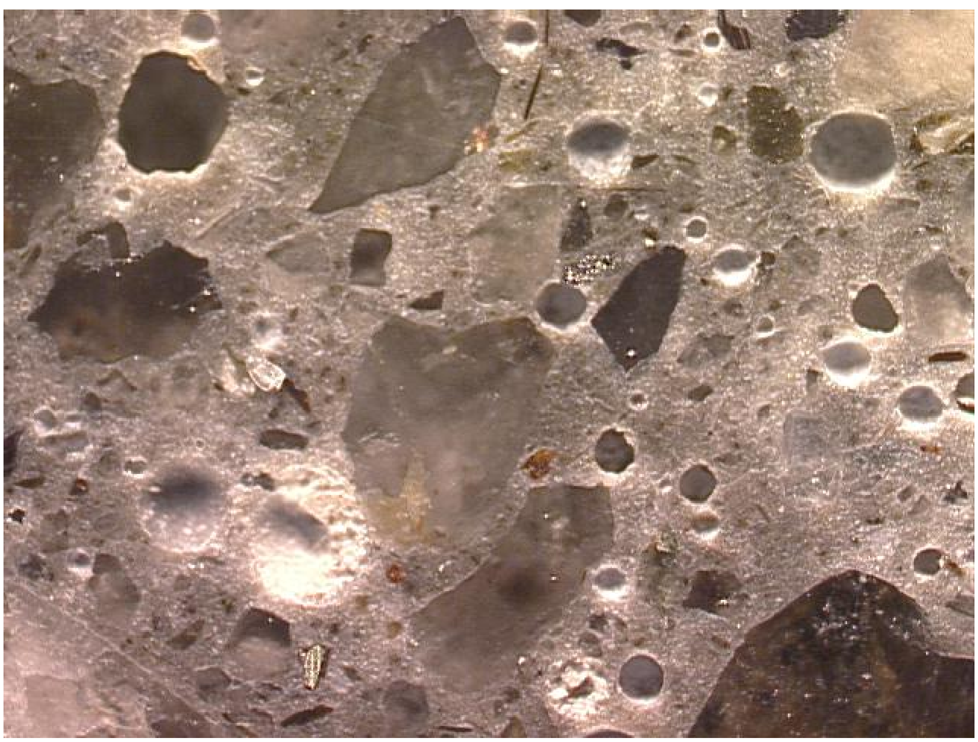


Figure F1-4: Entrained air voids in the concrete. Width of field is 4.0 mm. Polished surface image.

Figure F1-5: The bonding between aggregate and cement paste is strong. Width of field is 2.0 mm. Polished surface image.

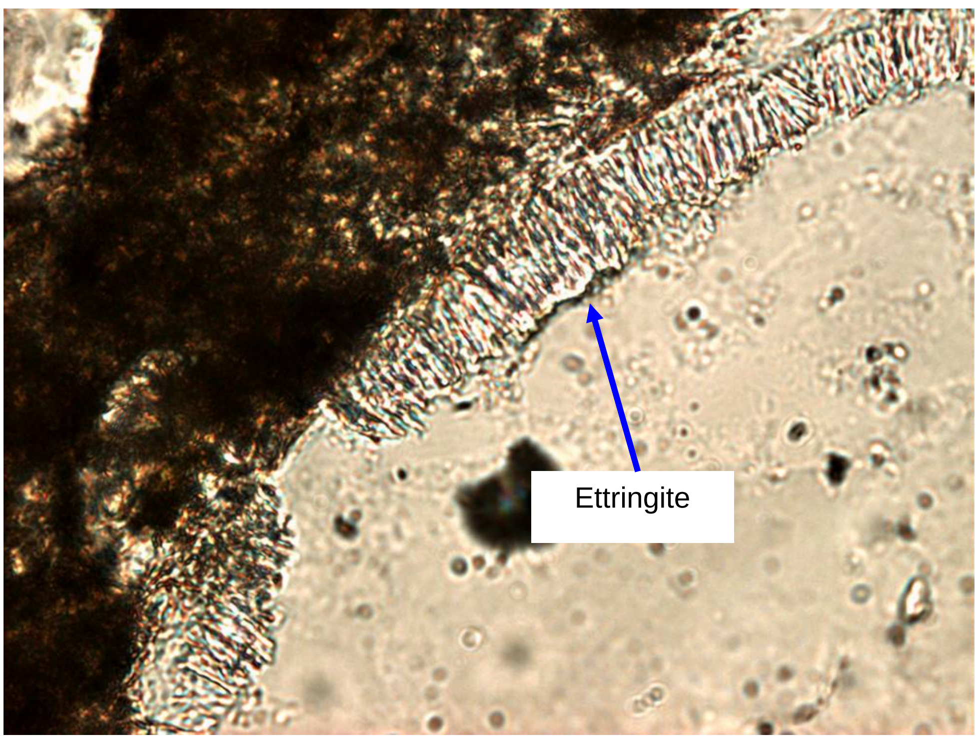


Figure F1-6: Ettringite in an air void. Width of field is 0.165 mm. Thin section image 0.165 mm.

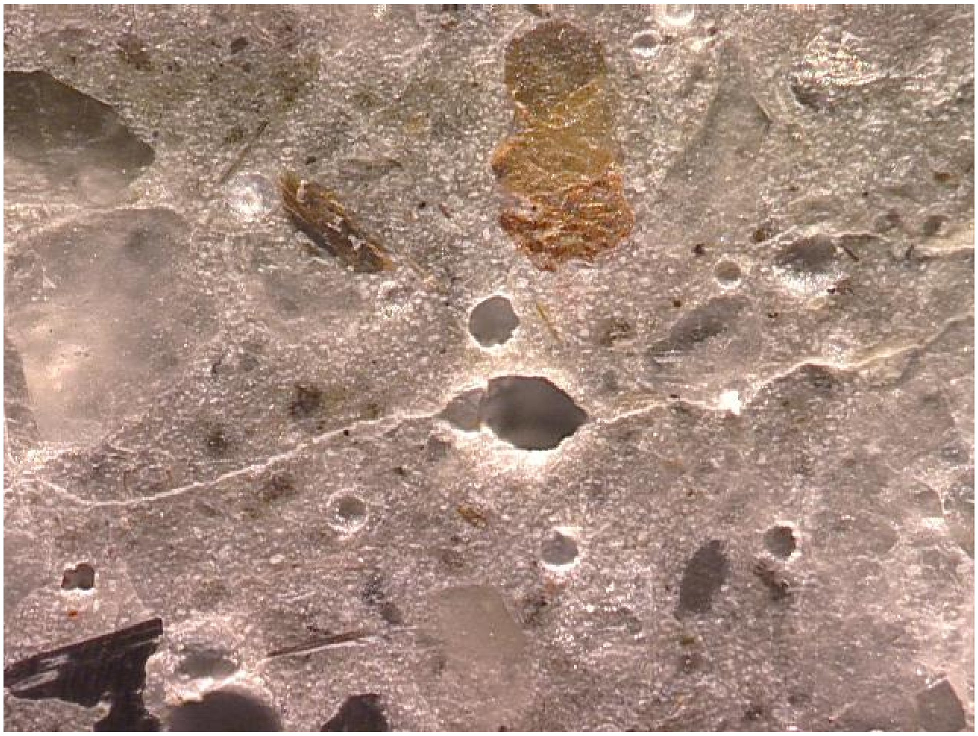


Figure F1-7: A crack in the paste. Width of field is 2.0 mm. Polished surface image.

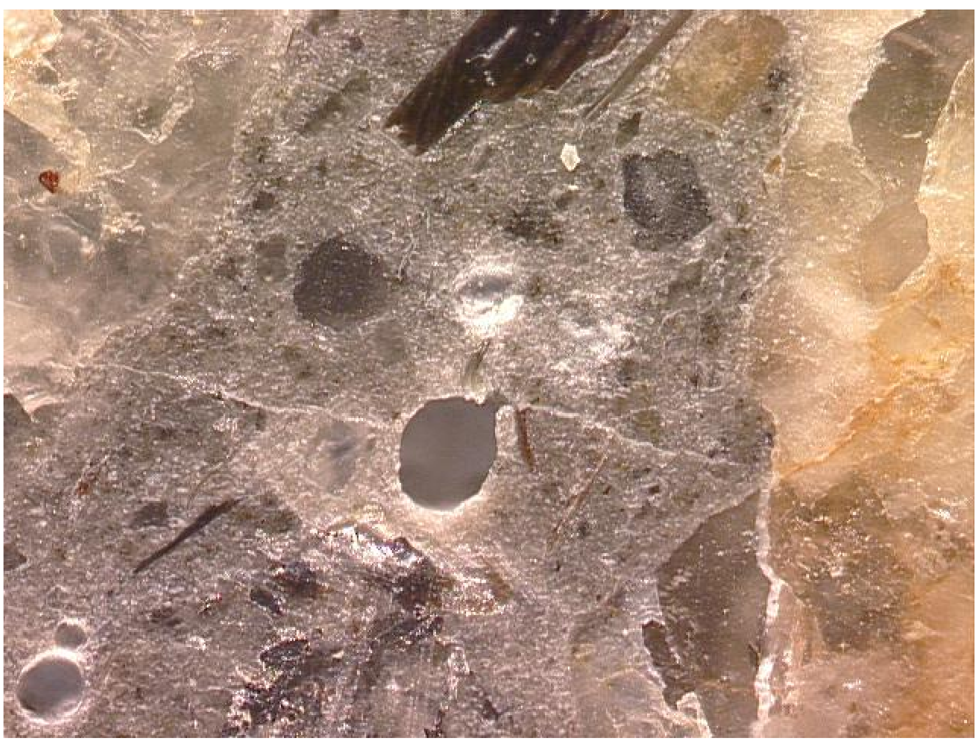


Figure F1-8: A crack connecting two aggregate pieces. Width of field is 2.0 mm. Polished surface image.

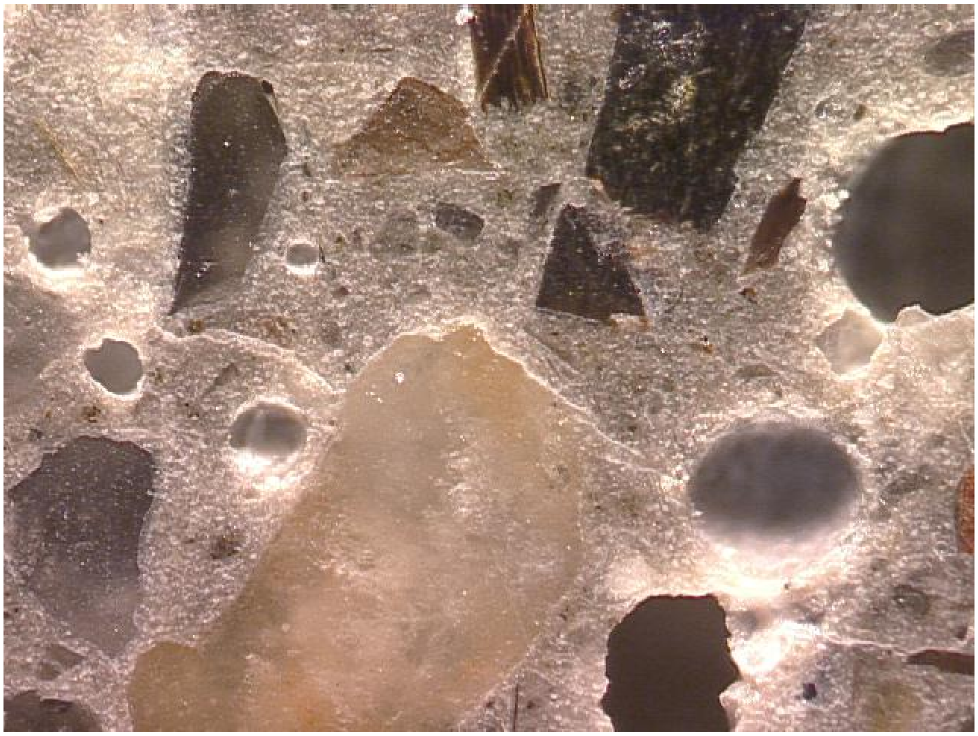


Figure F1-9: Cracks in the aggregate peripheral zone. Width of field is 2.0 mm. Polished surface image.

APPENDIX F – Supplement 2

Route 104 Bridge, New Hampshire
Survey Checklist

Checklist

The following checklist is adapted from 201.1R-92, and provided to facilitate a thorough survey. The definition of terms and associated photographs in 201.1 R-92, are utilized to standardize the reporting of the condition of the concrete in the structures.

1. Description of structure or pavement
   - 1.1 Name, location, type, and size: The Route 104 Bridge over the Newfound River in Bristol, New Hampshire was completed in summer 1996 and opened to traffic. It is a simple-span structure about 65 ft long. The clear width of the deck is 57.5 ft, including two through-traffic lanes, a shoulder, and a right-turn lane.
   - 1.2 Owner, project engineer, contractor, when built
     Owner-New Hampshire Department of Transportation. This bridge is part of a demonstration project for HPC in bridge structures which were co-sponsored by the Federal Highway Administration (FHWA) and the New Hampshire Department of Transportation (NHDOT). The bridge was constructed in summer 1996.
   - 1.3 Design
     - 1.3.1 Architect and/or engineer: the New Hampshire Department of Transportation (NHDOT)
     - 1.3.2 Intended use and history of use: To carry high volume of traffic over the Route 104. Opened to traffic in summer 1996.
     - 1.3.6 Special features: Bridge consists of one spans (65-ft long). The clear width of the deck is 57.5 ft, including two through-traffic lanes, a shoulder, and a right-turn lane. AASHTO Type III girders were used. HPC with specified strength of 6000 psi at 28 days was used in cast-in-place deck panels
   - 1.4 Construction
     - 1.4.1 Contractor-general, Weaver Brother Construction Company Inc. Concord, NH
     - 1.4.2 Subcontractors concrete placement: Beck and Belucci Inc. Franklin, NH
     - 1.4.3 Concrete Supplier: Persons Concrete Inc. of Winnisquam, Camton Plant, NH.
     - 1.4.4 Agency responsible for testing: NHDOT and University of New Hampshire
     - 1.4.5 Other subcontractors: Unistress Inc. Pittsfield, MA as beam fabricator
   - 1.5 Photographs
     - 1.5.1 General view — Photos 1 through 3
     - 1.5.2 Detailed close up of condition of area — Photos 4 through 6
   - 1.11 Sketch map-orientation showing sunny and shady and well and poorly drained regions N/A

2. Present condition of structure Date of Evaluation: The week of May 24, 2004

2.1 Overall alignment of structure No signs of misalignment
2.1.1 Settlement
2.1.2 Deflection
2.1.3 Expansion
2.1.4 Contraction
2.2 Portions showing distress (beams, columns, pavement, walls, etc., subjected to strains and pressures) None Observed
2.3 Surface condition of concrete
2.3.1 General (good, satisfactory, poor, dusting, chalking, blisters) Good
2.3.2 Cracks Longitudinal
2.3.2.1 Location and frequency See Figure 2
2.3.2.10 Type and size (see Definitions) See Figure 2

Transverse None
Width (from Crack comparator) NA
Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)

Craze N/A
Width (from Crack comparator) ____________
Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)

Map N/A
Width (from Crack comparator) ____________
Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)

D-Cracking N/A
Width (from Crack comparator) ____________
Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)

Diagonal NA
Width (from Crack comparator) NA
Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)

2.3.2.11 Leaching, stalactites N/A
2.3.3 Scaling N/A
2.3.3.1 Area, depth ____________

2.3.3.6	Type (see Definitions)		______
		Light	(Less than 1/8 in.)
		Medium	(1/8 in. – 3/8 in.)
		Severe	(3/8 in. – 3/4 in.)
		Very Severe	(Greater than 3/4 in.)
2.3.4	Spalls and popouts	None observed	
2.3.4.1	Number, size, and depth		NA
2.3.4.6	Type (see Definitions)		NA
	Spalls		
		Small	(Less than 3/4 in. depth)
		Large	(Greater than 3/4 in. depth)
	Popouts		
		Small	(Less than 3/8 in. diameter)
		Medium	(3/8 in. – 2 in. diameter)
		Large	(Greater than 2 in. diameter)
2.3.5	Extent of corrosion or chemical attack, abrasion, impact, cavitation		N/A
2.3.6	Stains, efflorescence		None observed
2.3.7	Exposed reinforcement		None
2.3.8	Curling and warping		N/A
2.3.9	Previous patching or other repair		N/A
2.3.10	Surface coatings		N/A
2.3.10.1	Type and thickness		N/A
2.3.10.2	Bond to concrete		N/A
2.3.10.3	Condition		N/A
2.3.11	Abrasion		N/A
2.3.12	Penetrating sealers ___		
2.3.12.1	Type		N/A
2.3.12.2	Effectiveness		N/A
2.3.12.7	Discoloration		N/A
2.4	Interior condition of concrete (in situ and samples)		N/A
2.4.1	Strength of cores		
2.4.2	Density of cores		
2.4.3	Moisture content		
2.4.4	Evidence of alkali-aggregate or other reactions		N/A
2.4.5	Bond to aggregate, reinforcing steel, joints		N/A
2.4.6	Pulse velocity		
2.4.7	Volume change		
2.4.8	Air content and distribution		
2.4.9	Chloride-ion content		
2.4.10	Cover over reinforcing steel		
2.4.11	Half-cell potential to reinforcing steel.		
2.4.12	Evidence of reinforcement corrosion		
2.4.13	Evidence of corrosion of dissimilar metals		
2.4.18	Delaminations		N/A
2.4.18.1	Location		N/A

2.4.18.2 Number, and size N/A
2.4.15 Depth of carbonation
2.4.16 Freezing and thawing distress (frost damage)
2.4.17 Extent of deterioration
2.4.22 Aggregate proportioning, and distribution

3. Nature of loading and detrimental elements
3.1 Exposure
3.1.1 Environment (arid, subtropical, marine, freshwater, industrial, etc.) N/A
3.1.2 Weather-(July and January mean temperatures, 81ºF and 28ºF
mean annual rainfall and 36-in
months in which 60 percent of it occurs) July
3.1.3 Freezing and thawing Significant amount of exposure
3.1.4 Wetting and drying Significant amount of exposure
3.1.9 Drying under dry atmosphere N/A
3.1.6 Chemical attack-sulfates, acids, chloride N/A
3.1.7 Abrasion, erosion, cavitation, impact N/A
3.1.8 Electric currents N/A
3.1.9 Deicing chemicals which contain chloride ions N/A
3.1.10 Heat from adjacent sources N/A
3.2 Drainage N/A
3.2.1 Flashing
3.2.2 Weepholes
3.2.3 Contour
3.2.4 Elevation of drains
3.3 Loading Research Test Data Available in Compilation CD Version 3
3.3.1 Dead
3.3.2 Live
3.3.3 Impact
3.3.4 Vibration
3.3.5 Traffic index
3.3.6 Other
3.4 Soils (foundation conditions)
3.4.1 Compressibility
3.4.2 Expansive soil
3.4.3 Settlement
3.4.4 Resistivity
3.4.5 Evidence of pumping
3.4.6 Water table (level and fluctuations)

4. Original condition of structure Good
4.1 Condition of formed and finished surfaces Good
4.1.1 Smoothness
4.1.2 Air pockets ("bugholes")
4.1.3 Sand streaks

	4.1.4	Honeycomb	______
	4.1.5	Soft areas (retarded hydration)	______
	4.1.6	Cold joints	______
	4.1.15	Staining	______
	4.1.16	Sand pockets	______
4.2	Defects		N/A
	4.2.1	Cracking	______
		4.2.1.1 Plastic shrinkage	______
		4.2.1.2 Thermal shrinkage	______
		4.2.1.3 Drying shrinkage	______
	4.2.6	Curling	______

5. Materials of Construction — See Table 2

6. Construction Practices — See Report pg. 3 and 7

APPENDIX G

Route 3A Bridge, New Hampshire

HIGH PERFORMANCE CONCRETE BRIDGE DECK INVESTIGATION

The Route 3A Bridge in Bristol, New Hampshire

I. BACKGROUND

Following the success of the Route 104 Bridge in Bristol, New Hampshire Department of Transportation (NHDOT) decided to construct another HPC bridge – the Route 3A Bridge over the Newfound River in Bristol, New Hampshire, about one mile away from the Route 104 Bridge. HPC was used for the girders, the precast prestressed deck panels, and the cast-in-place deck in the Route 3A Bridge. The Route 3A Bridge opened to traffic on June 25, 1999.

The Route 3A Bridge is a simple-span structure about 60 feet long. There are two traffic lanes and two shoulders for a clear deck width of 31.5 ft (9.1 m). The superstructure contains four New England Bulb-Tee (NEBT) prestressed concrete girders, spaced at 11.5 ft apart on center. The HPC girders also contain 0.6-in. diameter low-relaxation prestressing strands. The use of HPC allowed the designers to reduce the number of girders from five to four, resulting in substantial cost savings. The deck of Route 3A Bridge is composed of twenty-one 3.5-in. thick precast prestressed deck panels covered with 5.5 in. of cast-in-place concrete.

The Route 3A Bridge is part of a demonstration project for HPC in bridge structures, which are co-sponsored by the Federal Highway Administration (FHWA) and the New Hampshire Department of Transportation (NHDOT). University of New Hampshire undertook the research project to monitor the long-term behavior of HPC bridge. Many instruments were built within the bridge to measure concrete temperatures, elastic shortening, creep, shrinkage, and stresses of live and dead loads in the deck and girders. The temperature measurements provide information about the heat development during peak hydration and temperature gradients at later time periods. The measurements were related to the other measured concrete properties.

II. SCOPE OF SERVICES

Professional Service Industries (PSI) is under contract with the Federal Highway Administration (FHWA) to conduct bridge deck inspections for HPC bridges. Our scope of services on this bridge included a series of tasks and sub-tasks, which are described as follows:

- Collect all available information relevant to the construction of each bridge, including;
    - Deck Concrete Properties
    - Specified Deck Concrete Construction Procedures
    - Approved Concrete Mix Proportions
    - Measured Properties from QC

- Other Measured Properties
- Actual Method of Deck Placement
- Average Daily Traffic (ADT)
- Exposure Condition of the Bridge
- Any Performed Maintenance
- Any Inspection Reports
- Visually inspect the bridge, and obtain the following:
  - General condition of the deck top surface
  - Determination of the maximum crack width
  - General condition of the deck underside
  - General condition of the girders
  - Photograph areas of significant deterioration
  - Prepare drawings locating defects
  - Extract 5 concrete core samples

III. COMPILATION OF BRIDGE CONSTRUCTION INFORMATION

Information sources used for this report include bridge drawings, inspection reports, bridge summary data sheets, and technical information contained in FHWA's "Compilation and Evaluation of Results from High Performance Concrete Bridge Projects" version 3.

Deck Concrete Properties

The cast-in-place bridge deck had a specified concrete compressive strength of 6000 psi (41 MPa) at 28 days. Maximum water-to-cementitious materials ratio of 0.38 was specified for the cast-in-place deck concrete. For adequate protection against the likelihood of freeze-thaw cycles, the air content was specified to be 5-9%. Table 1 lists the specified concrete properties for the deck.

TABLE 1: Specified Concrete Properties

Property	Deck	Precast Deck Panels
Cement type:	Type II	---
Min. Cementitious Materials Content:	658 lb/yd^3	---
Max. W/CM Ratio:	0.38	---
Min. Percentage of Silica Fume:	7.5%	---
Max. Percentage of Silica Fume:	7.5%	---
Air Content:	5-9%	5-8%
Slump:	2-3 in.	5-7 in.
Compressive Strength - Design:	6000 psi @ 28 days	6000 psi @ 28 days
Chloride Permeability:	≤1000 Coulombs at 56 days	≤ 1500 coulombs at 56 days
Other:	Corrosion inhibitor at 4 gal/yd^3 only in deck	–––

Specified Deck Concrete Construction Procedures

For the Route 3A Bridge, the concrete supplier was required to mix several trial batches of concrete to determine an acceptable mixture design. Each trial batch was tested for slump, air content, concrete temperature, and unit weight. Once the NHDOT approved the mixture design, a 5 yd^3 trial pour was initiated so that the actual placing, finishing, and curing conditions could be evaluated. Such procedures allowed for the fine-tuning of the admixture dose and for the testing of the equipment needed for placement. Cotton mats were to be placed on the fresh concrete within 10 minutes of finishing and the deck was to be wet-cured for seven days.

Approved Concrete Mix Proportions

The approved proportions for cast-in-place deck panels are shown in Table 2. Note that the selected mixture design was chosen based on performance during trial batching.

TABLE 2: Approved Mix Proportions for the Route 104 Bridge

Mix Parameters	Cast-in-Place Deck	Precast Deck Panels
Cement Brand:	Ciment Quebec	Blue Circle
Cement Type:	Blended (1)	II
Cement Quantity:	660 lb/yd^3 (1)	550 lb/yd^3
Silica Fume Brand:	Ciment Quebec	Rheomax SF100
Silica Fume Quantity:	52 lb/yd^3 (1)	50 lb/yd^3
Slag Cement Brand, Quantity:		Newcem, 200 lb/yd^3
Fine Aggregate Quantity:	1190 lb/yd^3	1200 lb/yd^3
Fine Aggregate FM:	2.8	2.7
Fine Aggregate SG:	2.69	2.65
Coarse Aggregate, Max. Size:	¾ in.	¾ in.
Coarse Aggregate SG:	2.69	2.63
Coarse Aggregate Quantity:	1815 lb/yd^3	1750 lb/yd^3
Water:	253 lb/yd^3	242 lb/yd^3
Water Reducer Brand, Type:	Daracem 65, A	
Water Reducer Quantity:	19.8 fl oz /yd^3	
High Range Water Reducer Brand:	Daracem 100	Rheobuild 3000 FC
High Range Water Reducer Type:	F and G	A and F
High Range Water Reducer Quantity:	105.6 fl oz /yd^3	80 fl oz/yd^3
Air Entrainment Brand:	Daravair 1000	Darex II
Air Entrainment Type:	Saponified rosin	Organic acid salts
Air Entrainment Quantity:	4.5 fl oz /yd^3	5 fl oz/yd^3
Corrosion Inhibitor Brand:	DCI S	DCI S
Corrosion Inhibitor Type:	Calcium nitrate	Calcium nitrate
Corrosion Inhibitor Quantity:	4 gal/yd^3	4 gal/yd^3
Water/Cementitious Materials Ratio:	0.38	0.30

NOTES:
1. Mix designs recommended by the Univ. of New Hampshire. Minor changes made for approved mix.
2. Cement and silica fume were pre-blended. Total cementitious materials are 660 lb/yd^3.

Measured Properties from QC Tests of Production Concrete

Deck

Measured properties of the deck concrete mix from QC tests are summarized in Table 3. Air content, slump, and compressive strengths meet the specifications (Table 2).

TABLE 3: Measured Properties of QC Tests of the Production Concrete Mixes For the Cast-in-Place Deck and Precast Deck Panels

Property	Cast-in-Place Deck	Precast Deck Panels
Actual curing procedure:	Dry cotton mats were placed within 10 minutes of surface finishing. The mats were then wetted down and the deck wet cured for about seven days.	120 – 140 ^{o}F until a concrete strength of 4000 psi obtained
Average Slump:	5.25 in.	7.25 in.
Average Air Content:	6 %	5.7%
Average Unit Weight:	147.4 lb/ft^3	144.1 lb/ft^3
Compressive Strength:	5800 psi at 4 days 7100 psi at 7 days 9004 psi at 28 days	9400 psi at 28 days

Table 4 lists the composition of the cement and the cement blended with silica fume that were used in the mixture. Note that the addition of silica fume increases the percentage of silicon dioxide in the cementitious materials, as expected.

TABLE 4: Composition of Cement Used in the Production Concrete for the Deck

Component	Deck	
	w/o Silica Fume (%)	w/ Silica Fume (%)
Silicon dioxide (SiO_2)	21.5	27.18
Aluminum oxide (Al_2O_3)	4.9	4.40
Ferric oxide (Fe_2O_3)	3.1	2.67
Calcium oxide (CaO), Total	63.7	59.18
Calcium oxide (CaO), Free	0.7	—
Sulfur trioxide (SO_3)	2.9	2.96
Magnesium oxide (MgO)	2.4	2.18
Alkali equivalent (Na_2O)	0.8	—
Potassium monoxide (K_2O)	—	0.98
Strontium oxide (SrO)	—	0.18
Manganese sesquioxide (Mn_2O_2)	—	0.04
Zinc oxide (ZnO)	—	0.05
Chromium sesquioxide (Cr_2O_3)	—	0.02
Loss on ignition	0.7	0.84
Insoluble residue	0.3	3.44
Tricalcium aluminate (C_3A)	7.6	7.15
Tetracalcium aluminoferrite (C_4AF)	9.6	—
Tricalcium silicate (C_3S)	50.9	—
Dicalcium silicate (C_2S)	23.1	—

Measured Properties from Research Tests of Production Concrete for the Deck

Research tests of the production concrete showed that the compressive strength of the Route 3A Bridge had 28-day strengths greater than 8000 psi (Table 5), well above the specified 6000 psi. However, the chloride permeability was slightly higher than the specified value of 1000 Coulombs at 56 days.

TABLE 5: Measured Properties from Research Tests of Production Concrete for the Cast-in-Place Deck

Property	Value
Slump:	5 in.
Air Content:	6.1 %
Unit Weight:	142.7 lb/ft^3
Chloride Permeability (ASTM C1202):	1083 and 1036 at 56 days
Compressive Strength (ASTM C 39):	5759 psi at 4 days 7001 psi at 7 days 7822 psi at 14 days 8506 psi at 28 days 9120 psi at 56 days

NOTE: 4 in. x 8 in. cylinders were used for the compressive strength test.

Actual Method of Deck Placement

Construction of the Route 3A Bridge began in late fall of 1998, with the contract specifying that the bridge be opened to two lanes of traffic by July 4th of the following year. The first step of the construction was to divert traffic with the use of a temporary Acrow bridge. This bridge was set-up down stream of the bridge to be replaced and carried alternating one way traffic. The contractor then erected the abutments and wing-walls.

The four NEBT 60-foot girders were then placed with a crane and the 8.5 ft x 8 ft precast prestressed concrete deck panels were placed on the girders. The panels contained adjustable screw jacks in their corners. The screws were used to adjust the height of the panels. Once the panels were grouted into place, the screws were backed out through the top surface.

After the cast-in-place deck was poured, a standard, self-propelled finishing machine was implemented to strike off the top surface. Attached behind a screed were a finishing pan and burlap drag, used simultaneously to finish and texture the surface. Specifications strongly discouraged bull floating. Within 10 minutes after finishing and texturing, a section was covered with dry cotton mats and then wetted. The mats were kept wet for seven days.

The rapid placement of the mats reduced surface evaporation and eliminated shrinkage cracking. The specifications regarding evaporation at the time of placement were strictly enforced. If the evaporation rate was greater than 0.1 lb/ft^2/hr or if the ambient temperature was above 85^{o}F (29^{o}C) or below 50^{o}F, no placing of concrete was allowed. If the ambient temperatures dropped below 50^{o}F, the contractor was to provide provisions so that the concrete temperatures did not fall below 45^{o}F. Concrete temperatures at the time of placement were not to be higher than 90^{o}F. The hardened finish of the deck was transversely saw-cut on 1.5 in. centers with 0.125-in. wide and 0.25-in. deep grooves.

A slow moving truck weighing 16,000 lb in the front and 60,000 lb in the back was slowly driven across the completed bridge deck. Strain measurements showed that the deck system behaved the same in the middle of a deck panel as in the joints between panels. Furthermore, the data suggested that the deck acts more as a simply supported beam than a continuous beam.

Average Daily Traffic (ADT) and Average Daily Truck Traffic (ADTT)

Average daily traffic for both westbound and eastbound lanes was calculated based on a count of all vehicles crossing the bridge during a 10 minutes period beginning at 0935 hrs on May 26, 2004. The westbound ADT was 5,184, including 4,752 cars and 432 trucks. The eastbound ADT was 5,472, including 4,896 cars per day and 576 trucks. The estimation of traffic flow was made while the PSI inspection crew was on-site.

Exposure Condition of the Bridge

Based on climatology, the Route 3A Bridge in Bristol, NH experiences a wide range of conditions throughout the year. The mean daily maximum temperatures for Concord, NH (about 37 miles south of Bristol) range from a low of 29.8^{o}F in January to a high of 82.4^{o}F in July. Mean daily minimum temperatures in Concord vary between 7.4^{o}F in January and 56.5^{o}F in July. In Lebanon, NH, 37 miles to the west of Bristol, the mean daily maximum temperatures range from 28.0^{o}F in January to 81.2^{o}F in July. Lebanon's mean daily minimum temperatures range from 5.7^{o}F in January to 56.8^{o}F in July. The Bristol area in central NH experiences about 173 days per year in which air temperatures drop below 32^{o}F, implying a considerable number of freeze-thaw cycles. The possibility of below freezing temperatures and the fact that Concord and Lebanon receive on average about 64 and 76 inches of snow per year, respectively, suggests that the roads are treated for ice and snow. However, no specific information regarding the maintenance of this bridge has been located. Central NH receives an average monthly precipitation total of between 2.4 inches in January and 3.7 inches in July, with an annual average total of about 36 inches. Considerable precipitation throughout the year implies that the bridge experiences many wet/dry cycles.

Performed Maintenance

No documents were found that would indicate any maintenance had been performed since the bridge was constructed.

Inspection Reports

An inspection of the bridge was made by University of New Hampshire (UNH) researchers approximately one year after the cast-in-place deck was placed. As of fall 2001, the top surface was in excellent condition, with only five hairline cracks found. On the underside of the bridge, transverse cracks were observed in two of the 21 deck panels. On the top surface of the deck, five longitudinal cracks were observed. Four of these cracks were located at the ends of the bridge above the abutments. One crack was located towards mid-span. Other than numerous small chips observed near the saw-cut grooves in the deck, the bridge deck was reported to be in excellent condition.

IV. BRIDGE DECK INSPECTION

PSI personnel performed a visual inspection of the bridge decks during the week of May 24, 2004. The results of the inspection are summarized as follows.

General Condition of the Deck Top Surface

Figure 1 shows the general layout of the decks for the Route 3A Bridge. Results of visual inspection of the decks are shown in Figure 2. Surface defects observed and documented during visual inspection primarily included transverse cracks and longitudinal cracks (see photos 4 and 5). There are numerous small chips observed near the saw-cut grooves in the deck. Apparent signs of other serious damages such as freeze-thaw, D-cracking, alkali-silica reaction, and alkali-aggregate reaction were not observed. Longitudinal cracks were observed at span ends of cast-in-place deck, extended from the abutments to the approach slabs.

Transverse Cracks: A total of 5 transverse cracks were recorded during the visual survey of the bridge decks (see Figure 2). The sum of the transverse crack lengths was 18.5 ft over the bridge deck area of 1,890 ft^2. The transverse crack density (total crack length / deck area) for the eastbound and westbound bridges combined was calculated to be 0.010 ft/ft^2.

Diagonal Cracks: No diagonal cracks were observed on the bridge deck.

Longitudinal Cracks: A total of 2 longitudinal cracks were recorded during the visual survey of the bridge decks (see Figure 2). The sum of the longitudinal crack lengths was 12 ft over the bridge deck area of 1,890 ft^2. The longitudinal crack density (total crack length / deck area) for the eastbound and westbound bridges combined was calculated to be 0.006 ft/ft^2. These cracks occurred in the approach slab and were not on the deck.

A total of 7 cracks (5 traverse cracks and 2 longitudinal cracks) were recorded during visual survey of the bridge decks (see Figure 2). The sum of crack lengths was 30.5 ft over a bridge deck area of 1,890 ft^2. Crack density (total crack length / deck area) for the eastbound and westbound bridges combined was calculated to be 0.016 ft/ft^2.

It is noted that the transverse cracks mainly located on the eastbound bridge traffic lane. All cracks measured are hairline crack with a width of less than 0.031 in. Typical transverse crack and longitudinal crack are shown in photos 4 and 5, respectively. Photo 6 shows the joint pattern at the adjacent bridge decks.

TABLE 8: Measured Cracks on the Bridge Decks

Crack Type	Count	Length Range (feet)	Mean Length of Cracks (feet)	Median Length of Cracks (feet)	Total Length of Cracks (feet)	Total Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Transverse	5	1.5 to 6	3.7	4	18.5	1890	0.010

Maximum Crack Width

The maximum width of transverse cracks was measured to be 0.016 in. According to ACI 201, these crack widths are classified as hairline cracks.

General Condition of the Deck Underside

The underside of the deck exhibits no signs of distress. Photo 7 shows a general view of the underside of the deck.

General Condition of the Girders

The girders were inspected without the aide of any access equipment. No signs of distress were noted on any of the girders.

Concrete Core Samples

Five cores, 3-¾ inches in diameter, were retrieved from the decks. The core sample locations are shown on Figure 2. The locations were evenly distributed along each shoulder of the bridge. The cores were labeled NH3A-1 through NH3A-5 and were transferred to FHWA for further analysis.

TABLE 9: Core Dimensions

Sample	NH3A-1	NH3A-2	NH3A-3	NH3A-4	NH3A-5
Diameter (in.)	3¾	3¾	3¾	3¾	3¾
Length (in.)	3½	3½	3½	3½	2¾

Preliminary Conclusions

The construction of the Route 3A Bridge, following the success of the Route 104 Bridge in Bristol, NH, is the second showcase HPC bridge project by New Hampshire Department of Transportation (NHDOT) and the FHWA. The Route 3A Bridge opened to traffic on June 25, 1999. It was selected as the 2000 Precast/Prestressed Concrete Institute (PCI) Design Award winner for “Best Bridge, Spans under 65 Feet”.

Researchers from University of New Hampshire performed material testing, bridge instrumentation, and bridge monitoring throughout this project. It was reported that as of Fall 2001, five longitudinal cracks were observed. Four of these cracks were located at the ends of the bridge above the abutments. One crack was located towards mid-span.

The visual inspection of the bridge decks as part of our study was performed about two and half years after the bridge was inspected by the researchers at University of New Hampshire. A total of 7 cracks (5 traverse cracks and 2 longitudinal cracks) were recorded during visual survey of the bridge decks (see Figure 2). Two longitudinal cracks on the bridge were above the abutments, having crack width of 0.02 in. (0.5 mm). Compared to data reported by the University of New Hampshire, which mentioned 5 longitudinal cracks, it is suspected that some hairline cracks may have gone through the self-healing process and became invisible. However, it should also be noted that the bridge inspection was performed on a raining day. It is possible that smaller cracks may not be visible in such weather condition. In addition, 5 traverse cracks were reported from our inspection.

The longitudinal cracks at span ends above the abutment may be attributed to the different support conditions. The relatively flexible bridge structural system combined with the heavy ADT on the bridge might have contributed to the development and widening of some cracks.

In general, the top surface of Route 3A Bridge was in excellent condition, with only very limited hairline cracks found, showing that HPC designs provide significantly higher strength that can lead to more efficient designs and improved durability.

Petrographic examination of the concrete samples was performed on five concrete cores that were retrieved from the bridge. The identification on the cores was as follows: NH-3A-1, NH-3A-2, NH-3A-3, NH-3A-4, and NH-3A-5. All of the cores showed evidence of being broken off, and not being drilled all the way through. Visual inspection of the concrete cores revealed that core NH-3A-4 had a crack extending down about 2 in. from the exposed surface. A crack ran the full length of core NH-3A-1.

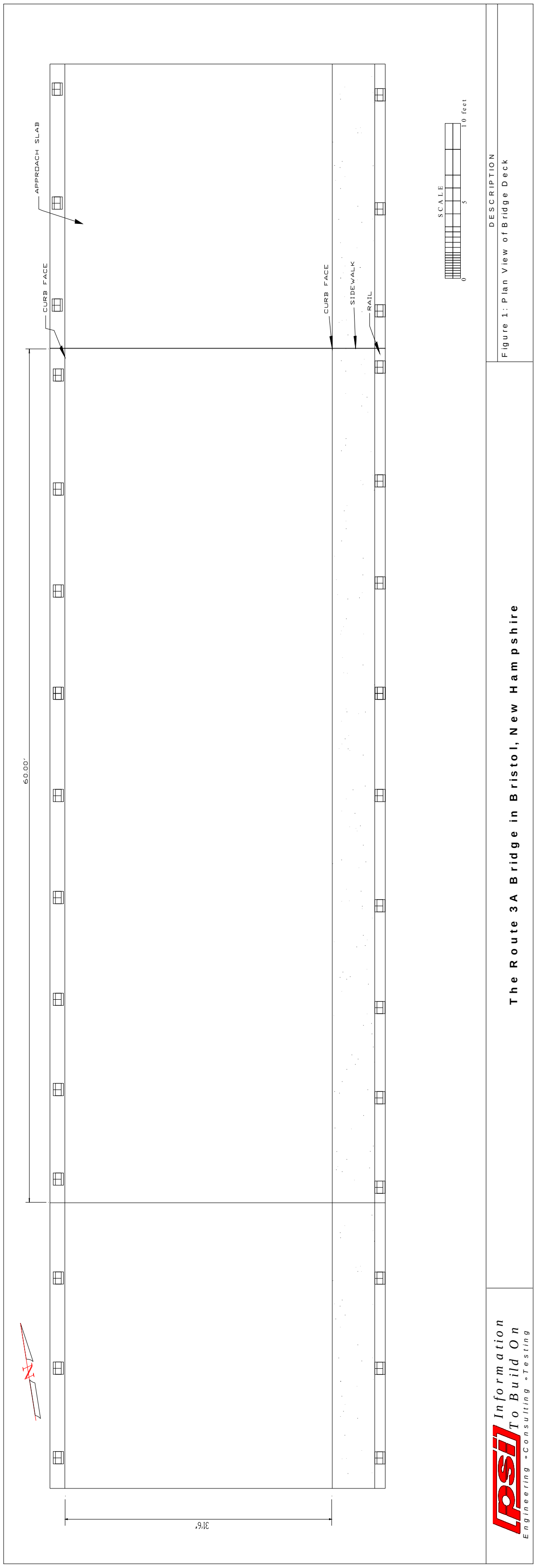
The gravel coarse aggregate in the concrete was composed of granite, quartzite, andesite, and basalt. Coarse aggregate particles were rounded to angular, and the maximum size, measured from the examined samples, was about 1 inch. Preferential orientation of coarse aggregate particles was not observed in this concrete, nor was segregation. The natural sand fine aggregate was composed of quartz, quartzite, granite, feldspar, mica, and andesite. The particles appeared rounded to angular.

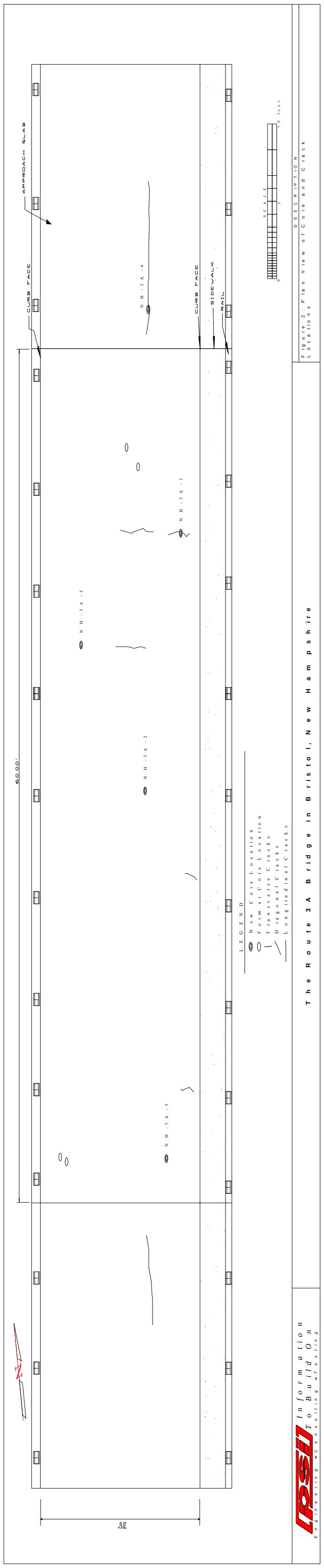
The cement was well hydrated with respect to the age of the concrete. The cement paste contained some unhydrated cement particles. Ground granulated blast-furnace slag particles were also present in the cement matrix. In general, bonding between the cement paste and the aggregate was strong.

The concrete was air entrained. Small, spherical entrained air voids were well distributed in the concrete. A small amount of entrapped air voids was also present in the concrete. Ettringite was sporadically observed in some air voids.

Occasionally, cracks were observed under the microscope. Micro-cracking was present in the cement paste. Cracking was also found in the peripheral zone between the paste and aggregate. Cracks that were adjacent to the exposed surface were much larger than those found in the bulk of the concrete. Large cracks were mostly found in the region of about 1 in. from the exposed surface. It was speculated that cracking was probably due to drying shrinkage, although other mechanisms might also contribute to the distress.

Respectfully Submitted,
Professional Service Industries, Inc.
Structural Investigation & Petrography Department


APPROACH SLAB
CURB FACE
CURB FACE
SIDEWALK
RAIL
60.00'
N
SCALE
0
5
10 feet
DESCRIPTION
Figure 1: Plan View of Bridge Deck
The Route 3A Bridge in Bristol, New Hampshire
Information
To Build On
Engineering • Consulting • Testing


APPROACH SLAB
CURB FACE
CURB FACE
SIDEWALK
RAIL
NH-3A-1
NH-3A-2
NH-3A-3
NH-3A-4
NH-3A-5
60.00'
LEGEND
New Core Location
Former Core Location
Transverse Cracks
Diagonal Cracks
Longitudinal Cracks
SCALE
10 feet
DESCRIPTION
Figure 2: Plan View of Core and Crack Locations
The Route 3A Bridge in Bristol, New Hampshire
PSI
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Photographic Documentation

	Photo 1: View looking south from the northern end of west facing bridge.
	Photo 2: View looking north from the southern end of west facing bridge.
	Photo 3: View looking south from the northern end of east facing bridge.

	Photo 4: Close up view of transverse crack.
	Photo 5: Close up view of longitudinal crack.
	Photo 6: General condition of joint pattern.

	Photo7: Close up view of underside of the bridge.
	Photo 8: Coring from the shoulder of the eastbound bridge.

APPENDIX G – Supplement 1

Route 3A Bridge, New Hampshire
Petrographic Examination

PETROGRAPHIC EXAMINATION OF CONCRETE CORES FROM A BRIDGE IN NEW HAMPSHIRE (NH3A)

A report submitted to the Structures Laboratory at TFHRC

Rongtang Liu
Petrographic Laboratory
FHWA TFHRC
(Reviewed by Richard Meininger, PE; Concrete Laboratory; printed in color 9-6-2006)

August 17, 2006

1. Introduction

Five concrete cores were received from the Structures Laboratory of the Turner-Fairbank Highway Research Center for petrographic examination. These cores were collected from a concrete bridge in New Hampshire. The identification on the cores, shown in Figure A-1, is as follows: NH-3A-1, NH-3A-2, NH-3A-3, NH-3A-4, and NH-3A-5.

Thin sections, as well as polished slabs, were made from the concrete cores for microscopic examination. Petrographic examination was performed on these samples using optical microscopes.

All of the cores showed evidence of being broken off, and not being drilled all the way through. Visual inspection of the concrete cores revealed that core NH-3A-4 has a crack extending down about 2-in. from the exposed surface. A longitudinal crack runs the full length of core NH-3A-1, as shown in Figure G1-1. The findings from microscopic examination indicate that the concrete has entrained air voids, and the air content is estimated as at a normal level; the hydration of the cement was reasonable; the presence of some unhydrated cement particles was observed in the cement paste; ground granulated blast-furnace slag was added as a supplementary cementitious material; cracks existed in the paste as well as in the aggregate peripheral zone; occasionally, ettringite as secondary deposit formed in some of the air voids.

2. Laboratory Procedures

Petrographic examination of the concrete samples was performed in accordance with ASTM C 856, "Standard Practice for Petrographic Examination of Hardened Concrete." Sections were polished and examined using a stereomicroscope at magnifications up to 200×. Small rectangular blocks were cut from concrete samples. One surface of each block was polished, dried, placed on petrographic slide with low−viscosity epoxy resin, and reduced to a thickness of approximately 25 micrometers (0.001 in.). These thin sections were examined using a polarized−light microscope at magnification up to 400×, to determine aggregate mineralogy, paste characteristics, and microstructure.

Two ¾ inch thick slabs were sawn from the concrete cores. The slabs were ground and polished to obtain a smooth and plane surface. They were examined using a stereomicroscope at magnifications up to 200×.

3. Findings

Six (6) thin section samples taken from the received cores were examined using the polarized light microscope. Two polished sections were examined using the stereomicroscope. The findings are summarized as follows:

Aggregates

The coarse aggregate in the concrete is gravel, which is composed of granite, quartzite, andesite, and basalt. Coarse aggregate particles are rounded to angular, and the maximum size is about 1 inch. Preferential orientation of coarse aggregate particles is not observed in this concrete, nor is segregation.

The fine aggregate fraction is composed of quartz, quartzite, granite, feldspar, mica, and andesite. The fine aggregate is from natural sand and the particles appear rounded to angular.

Cement Paste

The cement is well hydrated and the paste contains some unhydrated cement particles as seen under the microscope (Figure G1-2). Silica fume particles (as shown in Figure G1-3) are also present in the cement matrix.

Air Voids

Small, spherical air voids are observed in the concrete (Figure G1-4), hence the concrete was air entrained. Entrained air voids are well distributed in the concrete. The air content is estimated as at a normal level. A small amount of entrapped air voids is also present in the concrete.

Cement-Aggregate Bonding

In general, bonding between the cement paste and the aggregate is strong, as shown in Figure G1-5.

Secondary Deposit

Ettringite is sporadically observed in some air voids in the concrete (Figure G1-6).

Cracking

Occasionally, cracks are observed under the microscope. Micro-cracking is present in the cement paste, as shown in Figure G1-7. Cracking is also found in the peripheral zone between the paste and aggregate (Figure G1-8). Figure G1-9 shows cracks that are adjacent to the exposed surface. The sizes of these cracks are much larger than those found in the bulk of the concrete. And large cracks are mostly found in the region of about 1-in. from the exposed surface. It is speculated that cracking is probably due to drying shrinkage, although other mechanisms may also contribute to the distress.

4. Summary

The concrete was air entrained, and the entrained air voids were well distributed in the concrete. Unhydrated cement particles, as well as ground granulated blast-furnace slag particles, are present in the paste. In general, the bond between the aggregate and the paste appears strong. Cracks exist in the cement paste as well as in the peripheral zone between the paste and aggregate. It is speculated that shrinkage, among other mechanisms, may be the major cause of the cracking. Occasionally, ettringite is found in air voids. It is common to see ettringite as a secondary deposit in concrete.

Figure G1-1: Five cores as received.

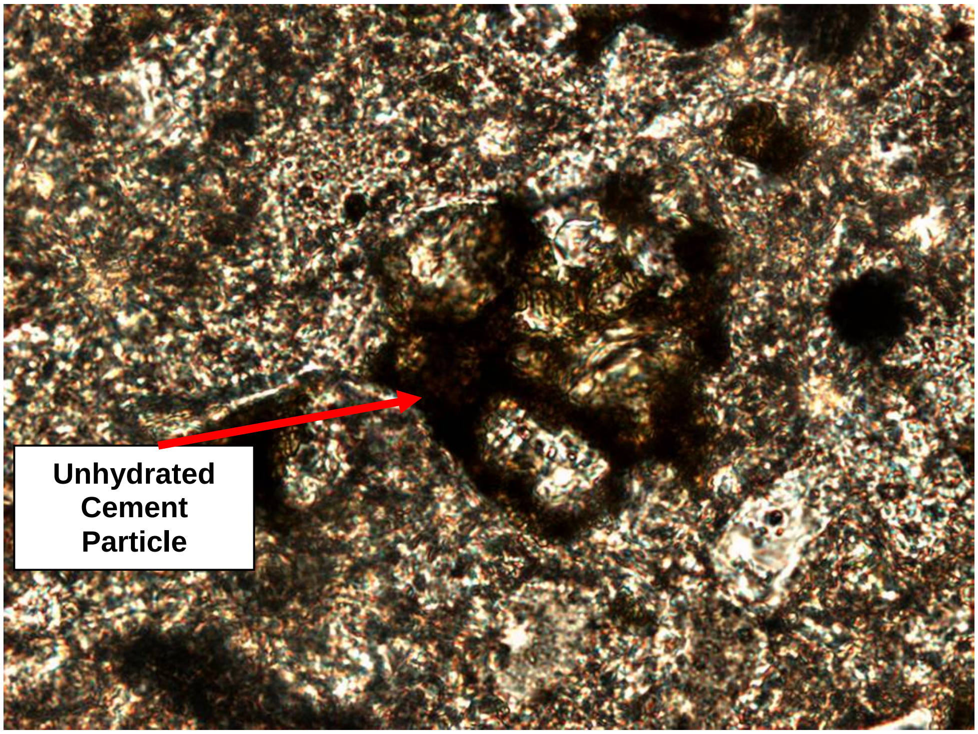


Figure G1-2: Unhydrated cement particle in paste. Width of field is 0.165 mm. Thin section image.

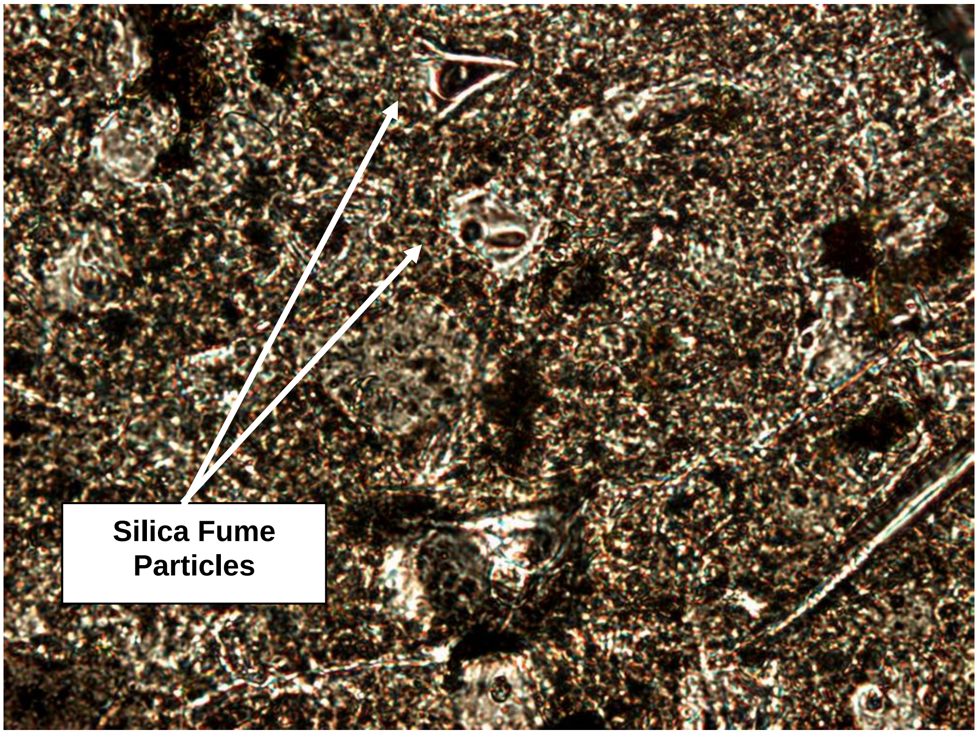


Figure G1-3: Ground granulated blast furnace slag particles in the cement matrix. Width of field is 0.165 mm. Thin section image.

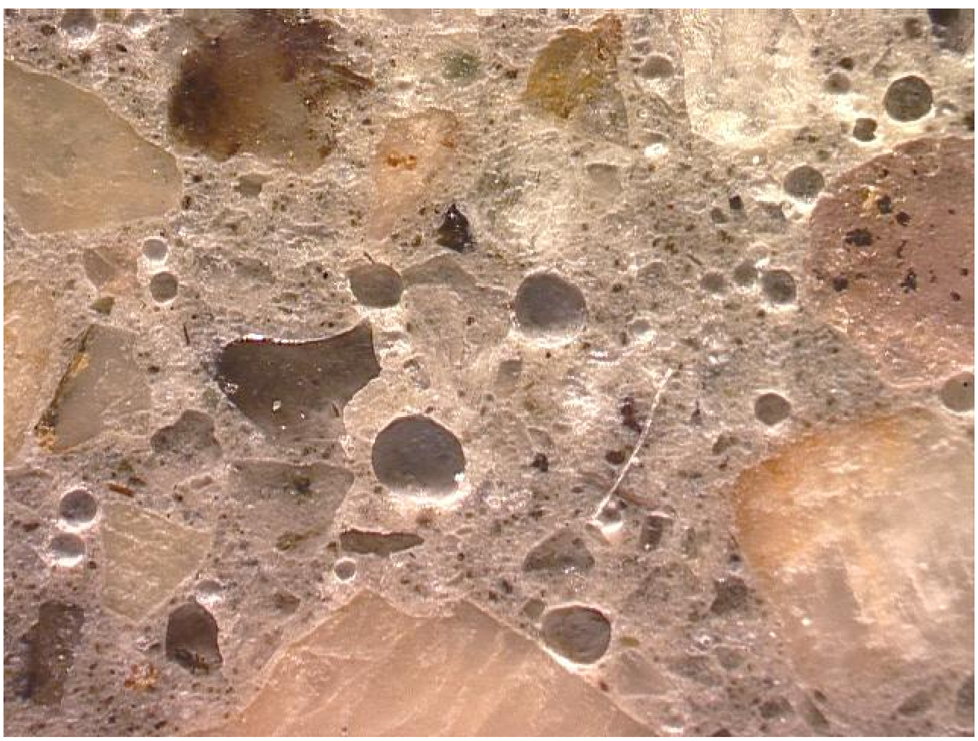


Figure G1-4: Entrained air voids in the concrete. Width of field is 4.0 mm. Polished surface image.

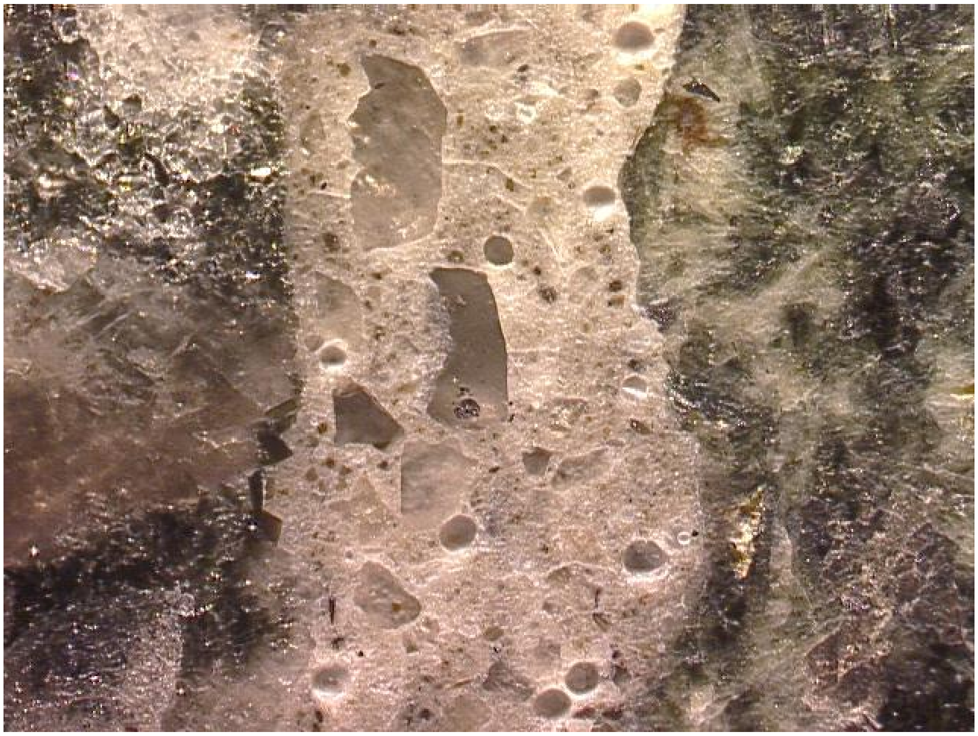


Figure G1-5: The bonding between aggregate and cement paste is strong. Width of field is 4.0 mm. Polished surface image.

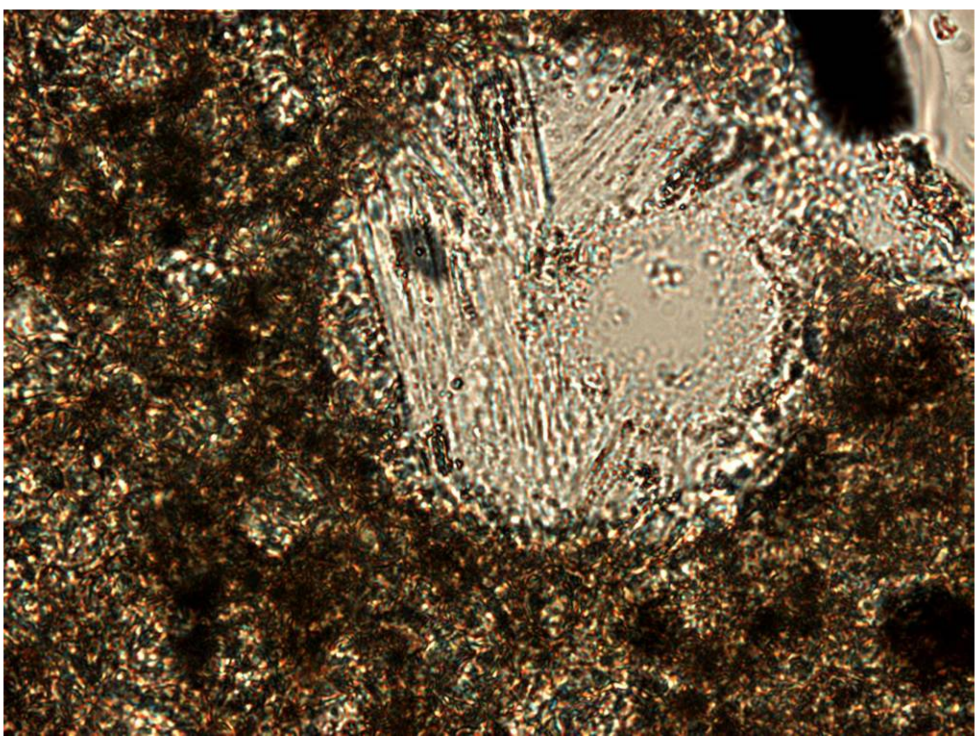


Figure G1-6: Ettringite in an air void. Width of field is 0.165 mm. Thin section image.

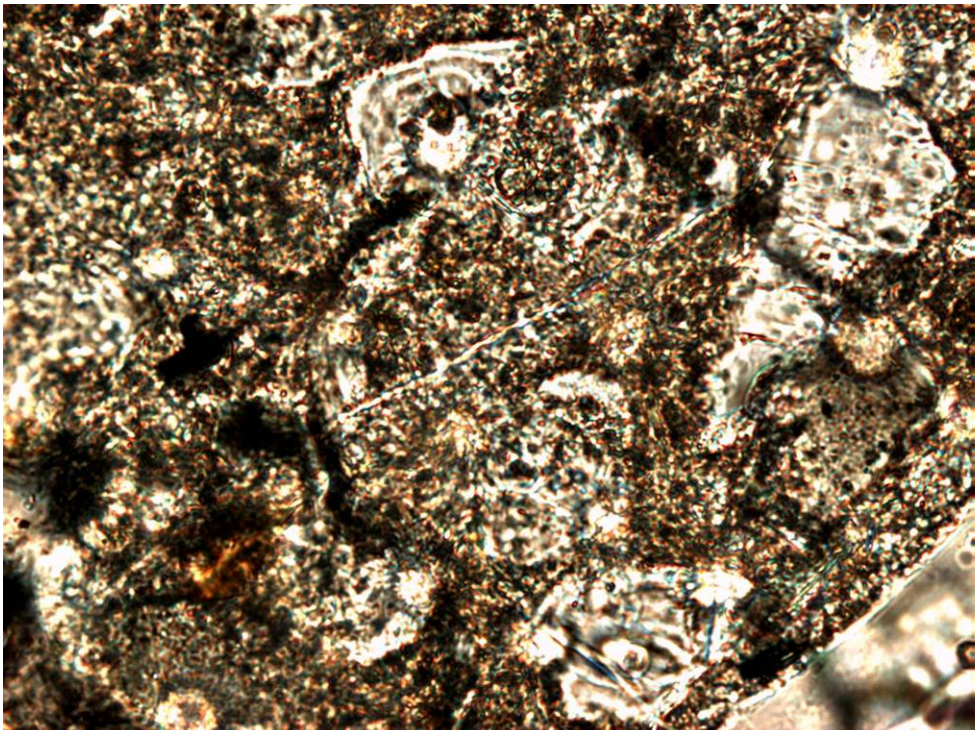


Figure G1-7: A crack in the paste. Width of field is 0.165 mm. Thin section image.

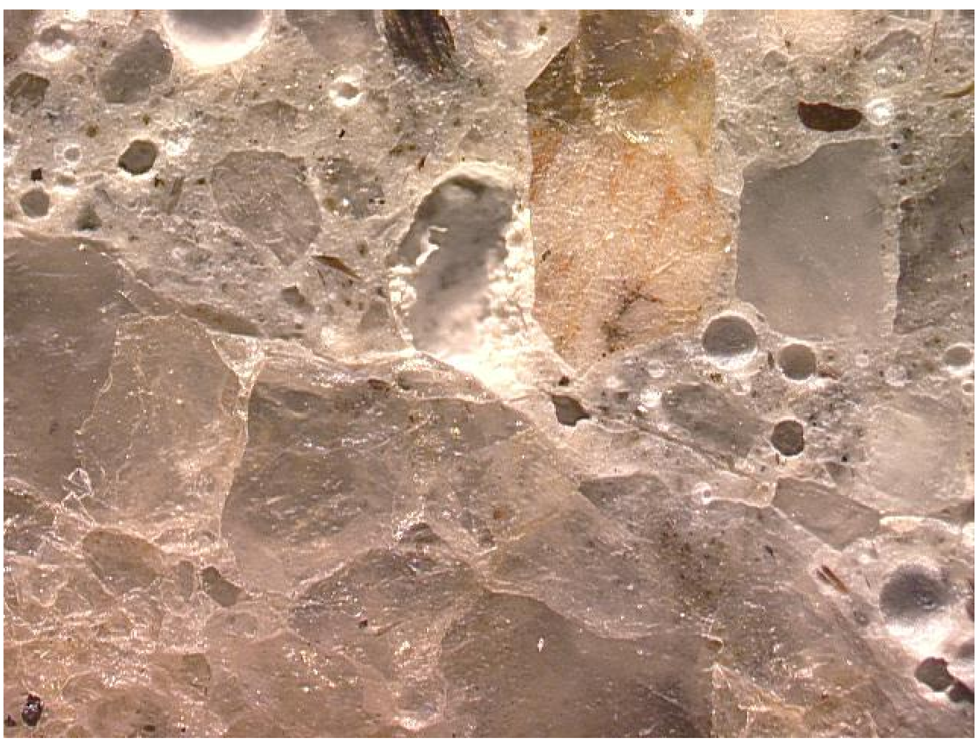


Figure G1-8: Crack in the aggregate-paste interfacial region. Width of field is 4.0 mm. Polished surface image.

Figure G1-9: Cracks that are found just below the exposed surface. Width of field is 4.0 mm. Polished surface section image. The exposed surface is on the right side.

APPENDIX G – Supplement 2

Route 3A Bridge, New Hampshire
Survey Checklist

Checklist

The following checklist is adapted from 201.1R-92, and provided to facilitate a thorough survey. The definition of terms and associated photographs in 201.1 R-92, are utilized to standardize the reporting of the condition of the concrete in the structures.

1. Description of structure or pavement
    - 1.1 Name, location, type, and size: The Route 3A Bridge over the Newfound River in Bristol, New Hampshire was opened to traffic on June 25, 1999. It is a simple-span structure about 60 ft long. The clear width of the deck is 30-ft, including two traffic lanes and two shoulders.
    - 1.2 Owner, project engineer, contractor, when built
      Owner-New Hampshire Department of Transportation. This bridge is part of a demonstration project for HPC in bridge structures which were co-sponsored by the Federal Highway Administration (FHWA) and the New Hampshire Department of Transportation (NHDOT). The bridge was constructed in 1999.
    - 1.3 Design
        - 1.3.1 Architect and/or engineer: the New Hampshire Department of Transportation (NHDOT)
        - 1.3.2 Intended use and history of use: To carry high volume of traffic over the Route 3A. Opened to traffic on June 25,1999.
        - 1.3.7 Special features: Bridge consists of one spans (60-ft long). The clear width of the deck is 30- ft, including two traffic lanes and two shoulders. New England Bulb-Tee (NEBT) prestressed concrete girders were used. HPC with specified strength of 6000 psi at 28 days was used in cast-in-place deck panels
    - 1.4 Construction
        - 1.4.1 Contractor-general, R.S. Audley of Bow, NH
        - 1.4.2 Subcontractors concrete placement: Northeast Concrete Products, Plainville, NH
        - 1.4.3 Concrete Supplier: Persons Concrete Inc. of Winnisquam, Camton Plant, NH.
        - 1.4.4 Agency responsible for testing: NHDOT and University of New Hampshire
        - 1.4.5 Other subcontractors: NA
    - 1.5 Photographs
        - 1.5.1 General view Photos 1 through 3
        - 1.5.2 Detailed close up of condition of area Photos 4 through 7
    - 1.12 Sketch map-orientation showing sunny and shady and well and poorly drained regions N/A

2. Present condition of structure Date of Evaluation: The week of May 24, 2004
    - 2.1 Overall alignment of structure No signs of misalignment
        - 2.1.1 Settlement

2.1.2 Deflection
2.1.3 Expansion
2.1.4 Contraction
2.2 Portions showing distress (beams, columns, pavement, walls, etc., subjected to strains and pressures) None Observed
2.3 Surface condition of concrete
2.3.1 General (good, satisfactory, poor, dusting, chalking, blisters) Good
2.3.2 Cracks Longitudinal, transverse
2.3.2.1 Location and frequency See Figure 2
2.3.2.12 Type and size (see Definitions) See Figure 2

Transverse
Width (from Crack comparator) Hairline
Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)

Craze N/A
Width (from Crack comparator) __________
Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)

Map N/A
Width (from Crack comparator) __________
Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)

D-Cracking N/A
Width (from Crack comparator) __________
Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)

Diagonal NA
Width (from Crack comparator) NA
Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)

2.3.2.13 Leaching, stalactites N/A
2.3.3 Scaling N/A
2.3.3.1 Area, depth __________
2.3.3.7 Type (see Definitions) __________
Light (Less than 1/8 in.)

Medium (1/8 in. – 3/8 in.)
Severe (3/8 in. – 3/4 in.)
Very Severe (Greater than 3/4 in.)

2.3.4 Spalls and popouts None observed
- 2.3.4.1 Number, size, and depth NA
- 2.3.4.7 Type (see Definitions) NA

Spalls
- Small (Less than 3/4 in. depth)
- Large (Greater than 3/4 in. depth)

Popouts
- Small (Less than 3/8 in. diameter)
- Medium (3/8 in. – 2 in. diameter)
- Large (Greater than 2 in. diameter)

2.3.5 Extent of corrosion or chemical attack, abrasion, impact, cavitation N/A

2.3.6 Stains, efflorescence None observed
2.3.7 Exposed reinforcement None
2.3.8 Curling and warping N/A
2.3.9 Previous patching or other repair N/A
2.3.10 Surface coatings N/A
- 2.3.10.1 Type and thickness N/A
- 2.3.10.2 Bond to concrete N/A
- 2.3.10.3 Condition N/A

2.3.11 Abrasion N/A
2.3.12 Penetrating sealers ___
- 2.3.12.1 Type N/A
- 2.3.12.2 Effectiveness N/A
- 2.3.12.8 Discoloration N/A

2.4 Interior condition of concrete (in situ and samples) N/A
- 2.4.1 Strength of cores
- 2.4.2 Density of cores
- 2.4.3 Moisture content
- 2.4.4 Evidence of alkali-aggregate or other reactions N/A
- 2.4.5 Bond to aggregate, reinforcing steel, joints N/A
- 2.4.6 Pulse velocity
- 2.4.7 Volume change
- 2.4.8 Air content and distribution
- 2.4.9 Chloride-ion content
- 2.4.10 Cover over reinforcing steel
- 2.4.11 Half-cell potential to reinforcing steel.
- 2.4.12 Evidence of reinforcement corrosion
- 2.4.13 Evidence of corrosion of dissimilar metals
- 2.4.19 Delaminations N/A
  - 2.4.19.1 Location N/A
  - 2.4.19.2 Number, and size N/A
- 2.4.15 Depth of carbonation

2.4.16 Freezing and thawing distress (frost damage)
2.4.17 Extent of deterioration
2.4.23 Aggregate proportioning, and distribution

3. Nature of loading and detrimental elements
3.1 Exposure
3.1.1 Environment (arid, subtropical, marine, freshwater, industrial, etc.) N/A
3.1.2 Weather-(July and January mean temperatures, 81ºF and 28ºF
mean annual rainfall and 36-in
months in which 60 percent of it occurs) July
3.1.3 Freezing and thawing Significant amount of exposure
3.1.4 Wetting and drying Significant amount of exposure
3.1.10 Drying under dry atmosphere N/A
3.1.6 Chemical attack-sulfates, acids, chloride N/A
3.1.7 Abrasion, erosion, cavitation, impact N/A
3.1.8 Electric currents N/A
3.1.9 Deicing chemicals which contain chloride ions N/A
3.1.10 Heat from adjacent sources N/A
3.2 Drainage N/A
3.2.1 Flashing ____
3.2.2 Weepholes ____
3.2.3 Contour ____
3.2.4 Elevation of drains ____
3.3 Loading Research Test Data Available in Compilation CD Version 3
3.3.1 Dead ____
3.3.2 Live ____
3.3.3 Impact ____
3.3.4 Vibration ____
3.3.5 Traffic index ____
3.3.6 Other ____
3.4 Soils (foundation conditions) ____
3.4.1 Compressibility
3.4.2 Expansive soil
3.4.3 Settlement
3.4.4 Resistivity
3.4.5 Evidence of pumping
3.4.6 Water table (level and fluctuations)

4. Original condition of structure Good
4.1 Condition of formed and finished surfaces Good
4.1.1 Smoothness ____
4.1.2 Air pockets ("bugholes") ____
4.1.3 Sand streaks ____
4.1.4 Honeycomb ____
4.1.5 Soft areas (retarded hydration) ____

4.1.6 Cold joints ____________________
4.1.17 Staining ____________________
4.1.18 Sand pockets ____________________
4.2 Defects N/A
4.2.1 Cracking ____________________
4.2.1.1 Plastic shrinkage ____________________
4.2.1.2 Thermal shrinkage ____________________
4.2.1.3 Drying shrinkage ____________________
4.2.7 Curling ____________________

5. Materials of Construction See Table 2

6. Construction Practices See Report pg. 3 and 6

APPENDIX H

Old Route 66 Bridge over Rio Puerco, New Mexico

HIGH PERFORMANCE CONCRETE BRIDGE DECK INVESTIGATION

Old Route 66 Bridge over Rio Puerco, New Mexico

I. BACKGROUND

Old Route 66 Bridge over Rio Puerco, west of Albuquerque, New Mexico, was the first HPC bridge deck project by New Mexico Highway and Transportation Department. The purpose of the project was to establish the viability of HPC in New Mexico. HPC was used throughout the superstructure. The Rio Puerco Bridge was completed and opened to traffic in December 2000.

The Rio Puerco Bridge has three spans of 96.1, 101.1, and 96.1 ft (29.3, 30.8, and 29.3 m), respectively. Each span consists of four 63-in. (1.6-m) deep bulb-tee beams spaced at 12.6 ft (3.8 m) centers. The prestressed concrete beams had specified concrete compressive strengths of 7000 psi (48 MPa) at release and 10,000 psi (69 MPa) at 56 days. The specified strength for the deck concrete was 6000 psi (41 MPa) at 28 days with a mix requirement of 7500 psi (52 MPa) at 56 days. Class F fly ash was used to mitigate the potential for alkali-silica reactivity. The Rio Puerco Bridge has a 8.7-in. (220-mm) thick cast-in-place concrete deck. The clear width of the deck is 47.6 ft (14.5 m).

The Rio Puerco Bridge is part of a demonstration project for HPC in bridge structures, and was co-sponsored by the Federal Highway Administration (FHWA) and the New Mexico Highway and Transportation Department. University of New Mexico was involved in testing the HPC mixture, and New Mexico State University undertook the research project to monitor the long-term behavior of HPC bridge. The prestressed concrete beams were instrumented and monitored using fiber-optic sensors.

The effects of New Mexico's initial experience with HPC at Rio Puerco Bridge have been significant and lasting. HPC has been used on many other projects thereafter. In addition, the success of the HPC precast, prestressed concrete beams has resulted in an increased confidence level with prestressed concrete construction in general. While the material costs for HPC were 20 percent higher than conventional concrete on the Rio Puerco Bridge construction, the enhanced workability achieved with HPC has been demonstrated to result in lower labor costs. The overall bridge cost increase is about 10 percent, and it is anticipated that, as more HPC projects are built, material costs will decrease to those of conventional concrete. HPC has proven to be a viable and effective alternative for bridge construction in New Mexico.

II. SCOPE OF SERVICES

Professional Service Industries (PSI) is under contract with the Federal Highway Administration (FHWA) to conduct bridge deck inspections for HPC bridges. Our scope of services on this bridge included a series of tasks and sub-tasks, which are described as follows:

- Collect all available information relevant to the construction of each bridge, including;
    - Deck Concrete Properties
    - Specified Deck Concrete Construction Procedures
    - Approved Concrete Mix Proportions
    - Measured Properties from QC
    - Other Measured Properties
    - Actual Method of Deck Placement
    - Average Daily Traffic (ADT)
    - Exposure Condition of the Bridge
    - Any Performed Maintenance
    - Any Inspection Reports
- Visually inspect the bridge, and obtain the following:
    - General condition of the deck top surface
    - Determination of the maximum crack width
    - General condition of the deck underside
    - General condition of the girders
    - Photograph areas of significant deterioration
    - Prepare drawings locating defects

III. COMPILATION OF BRIDGE CONSTRUCTION INFORMATION

Information sources used for this report include bridge drawings, inspection reports, bridge summary data sheets, and technical information contained in FHWA's "Compilation and Evaluation of Results from High Performance Concrete Bridge Projects" version 3.

Deck Concrete Properties

The specified strength for the deck concrete was 6000 psi (41 MPa) at 28 days and with a mix requirement of 7500 psi (52 MPa) at 56 days. In addition, the fresh concrete was required to have a higher slump than usual. A maximum slump of 9 in. (230 mm) was established without segregation. Class F fly ash was used in the concrete mix to mitigate the potential for alkali-silica reactivity, and to achieve the desired workability along with water reducers. Table 1 lists the specified concrete properties for the cast-in-place deck.

TABLE 1: Specified Concrete Properties

Property	Deck
Cement type:	I/II
Air Content:	4.5-9.0%
Slump:	≤9 in.
Compressive Strength - Design:	41 MPa at 28 days

Specified Deck Concrete Construction Procedures

The placement of the HPC deck required a number of special procedures. First, a fogging system was developed to maintain a high localized relative humidity for the finished concrete in the otherwise arid New Mexico climate. It was specified that no high performance concrete shall be placed if the evaporation potential is in excess of 0.73 kg of water/square meter/hour. The evaporation potential shall be determined prior to fogging and outside the wind protection. Immediately after the concrete has been placed, the concrete shall be protected to reduce or eliminate pre-mature evaporation from the surface of concrete. A movable windbreak surrounded the sides and rear of the fogging system will be utilized during concrete pouring and finishing operations. After finishing the deck surface, a curing compound was required, and the deck was covered with wet burlap and polyethylene sheeting for a minimum of 14 days.

Another special requirement for the Rio Puerco Bridge construction project was the placement of a test slab. A 44.5 x 30.2 ft (13.6 x 9.2 m) slab was placed using the proposed concrete mix, fogging system, and finishing machine, and the same personnel used on the cast-in-place deck pour. If test results from the test slab are not approved by the Central Materials Laboratory, the contractor will be required to construct additional test slabs at the contractor expense until test results are approved.

Approved Concrete Mix Proportions

The approved proportions for cast-in-place deck are shown in Table 2.

TABLE 2: Approved Mix Proportions for the Old Route 66 Bridge

Mix Parameters	Cast-in-Place Deck
Cement Brand:	Phoenix Cement
Cement Type:	I/II
Cement Quantity:	687 lb/yd^3
Fly Ash Type:	F
Fly Ash Quantity:	172 lb/yd^3
Fine Aggregate Quantity:	1190 lb/yd^3
Fine Aggregate FM:	2.77
Fine Aggregate SG:	2.53
Coarse Aggregate, Max. Size:	½ in.
Coarse Aggregate SG:	2.67
Coarse Aggregate Quantity:	1400 lb/yd^3
Water:	275 lb/yd^3
High Range Water Reducer Brand:	Adva Flow
High Range Water Reducer Type:	F
High Range Water Reducer Quantity:	56.3 fl oz/yd^3
Air Entrainment Brand:	Daravair 1000
Air Entrainment Type:	Saponified rosin
Air Entrainment Quantity:	8.6 fl oz /yd^3
Water/Cementitious Materials Ratio:	0.32

Measured Properties of the Approved Concrete Mix

The approved concrete mix had a slump of 7.25 in. and an air content of 7%, as indicated in Table 3.

TABLE 3. Measured Properties of the Approved Concrete Mix for the Cast-in-Place Deck

Property	Value
Slump	7.25 in
Air Content	7.0%
Unit Weight:	137.9 lb/ft^3
Compressive Strength:	5955 psi at 14 days 7873 psi at 28 days 9341 psi at 56 days

Measured Properties from QC Tests of Production Concrete

Cast-in-Place Deck

Measured properties of the deck concrete mix from QC tests are summarized in Table 4.

TABLE 4: Measured Properties of the Actual Concrete for the Cast-in-Place Deck

Property	Value
Slump	65-215 mm
Air Content	4.5-8.2%
Unit Weight:	2139-2403 kg/m^3

Measured Properties from Research Tests of Production Concrete for the Deck

Coefficient of Thermal Expansion

The Coefficient of Thermal Expansion test results on the production concrete are average values determined from strain and temperature measurements on the beams. Measured coefficient of thermal expansion from research tests are summarized in Table 6.

TABLE 5: Measured Properties of QC Tests of the Actual Cured Production Concrete Mixes for the Cast-in-Place Deck

Set No.	Compressive Strength, psi		
	7 days	28 days	56 days
Bridge Deck			
1	5458	6777 7029 6979	—
2	4889	5780 5714 5841	—
3	4958	5941 6009	7417
4	4508	5261 5222	6693
5	4957	5750 5718	7161
6	5160	6059 6286	8087
7	5196	6482 6295	7949
8	5406	6526 6395	7696
Bridge Deck and Diaphragm			
1	4961	5947 6105	—
2	5460	6430 6626	—
3	5109	6262 6283	—
4	5326	6182 6248	—
Average	5116	6160	7501

TABLE 6: Measured Coefficient of Thermal Expansion From Research Test of the Production Concrete

Age, days	Coefficient of Thermal Expansion, millionths/^{o}C
7	12.5
31	12.7

Actual Method of Deck Placement

In the construction of the Rio Puerco Bridge, immediately following the application of the final finish, the concrete were completely and comprehensively covered with an approved curing compound. Following application of the curing compound, the concrete were covered immediately with saturated burlap and a polyethylene sheeting material. This sheeting material was applied in such a manner that all joints were overlapped by the adjacent sheet by at least 24 inches. All joints were immediately covered with duct tape. The sheeting material used was completely free from any holes, tears, or other openings. Any openings discovered were immediately sealed with a permanent sealing method.

The entire deck was re-saturated every day during the curing period. During the curing period, there was no traffic, other than foot traffic, allowed upon this concrete deck. A water fog was continuously applied over the surface of the freshly placed concrete in such a manner that the entire surface was kept at a relative humidity of 90% or greater. The wet burlap was not applied until the deck can receive the wet burlap and any placement loads without deformation.

Average Daily Traffic (ADT)

Average daily traffic for both eastbound and westbound lanes was calculated based on a count of all vehicles crossing the bridge during a 10 minutes period beginning at 1235 hrs on June 9, 2004. The ADT was 3,168, including 2,592 cars and 576 trucks. The estimation of traffic flow was made while the PSI inspection crew was on-site.

Exposure Condition of the Bridge

The National Weather Service reports that the normal maximum temperature varies between 92°F in July and 47°F in January. The normal minimum temperature varies between 65°F in July and 23°F in January. The normal precipitation varies between 0.4 inches per month in January to 1.5 inch per month in August. Very few days per year the temperature drops below 32°F. Based on this information, the bridge has minimal annual exposure to wet/dry and freeze/thaw cycles.

Performed Maintenance

No documents were found which would indicate any maintenance had been performed since the bridge was constructed.

Inspection Reports

New Mexico State University undertook the research project to monitor the long-term behavior of HPC bridge. A research report "The Rio Puerco Bridge: Monitoring Prestress Losses in a High Performance Concrete Bridge with a Built-In Fiber-optic Sensor System," was published in May 2002. No further information has been available.

IV. BRIDGE DECK INSPECTION

PSI personnel performed a visual inspection of the bridge decks during the week of June 7, 2004. The results of the inspection are summarized as follows.

General Condition of the Deck Top Surface

Figure 1 shows the general layout of the decks for the Rio Puerco Bridge. Results of visual inspection of the decks are shown in Figure 2. Surface defects observed and documented during visual inspection primarily are longitudinal cracks, diagonal cracks, and transverse cracks (see photos 5 through 7). Apparent signs of other serious damages such as freeze-thaw, D-cracking, alkali-silica reaction, and alkali-aggregate reaction were not observed.

A total of 169 cracks (50 traverse cracks, 30 longitudinal cracks, and 89 diagonal cracks) were recorded during visual survey of the bridge decks (see Figure 2). The sum of crack lengths was 651.3 ft over a bridge deck area of 13,964.1 ft^2. Crack density (total crack length / deck area) for the eastbound and westbound bridges combined was calculated to be 0.047 ft/ft^2.

It is noted that the number of diagonal crack accounts for majority of cracks recorded, and the total length for diagonal cracks is 260 ft. The 50 transverse cracks have the greatest length of 301.8 ft. Span A, defined as bridge deck between bearing abutment #1 and pier diaphragm #1, has the least amount of cracks combined (46 crack counts), Span B, between pier diaphragm #1 and pier diaphragm #2 has 57 cracks, and Span C, between pier diaphragm #2 and bearing abutment #2 has 66 cracks. All cracks measured are hairline crack with a width of less than 0.031 in. Typical longitudinal crack, diagonal crack, and transverse crack on the bridge decks are shown in photos 5, 6 and 7, respectively.

Cracks were typically limited at span ends. Small surface spalls, which either occurred due to breaking of tined edges or the crack edges, were observed. Figure 2 also illustrates the locations of drilled cores.

The number, length and density of cracks for entire bridge decks in both directions are shown in Tables 7 through 9, and described below according to the crack type.

Transverse Cracks: Figure 2 illustrates the transverse cracks that were identified on the surface of the bridge decks. Table 7 provides the detailed information regarding transverse cracks identified on the bridge decks. The crack densities (crack length per deck area) range from 0.0120 to 0.0298 ft/ft^2 for the 3 spans investigated.

TABLE 7: Measured Transverse Cracks on the Bridge Decks

Traverse Cracks	Count	Length Range (feet)	Mean Length of Cracks (feet)	Median Length of Cracks (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span A	12	1 to 7	4.6	5	103.5	4577.2	0.0226
Span B	21	0.3 to 22	6.8	6	143.3	4809.6	0.0298
Span C	17	2 to 14	6.1	5	55.0	4577.2	0.0120

NOTES: Transverse cracks include cracks oriented parallel to skewed joints

Diagonal Cracks: The diagonal crack densities (crack length per deck area) range from 0.0153 to 0.0247 ft/ft^2 for the 3 spans investigated. Diagonal cracks in the bridge decks typically present near the joints.

TABLE 8: Measured Diagonal Cracks on the Bridge Decks

Traverse Cracks	Count	Length Range (feet)	Mean Length of Cracks (feet)	Median Length of Cracks (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span A	24	1 to 8	3.1	2	73.5	4577.2	0.0161
Span B	29	0.5 to 8	2.5	2	73.5	4809.6	0.0153
Span C	36	1 to 8	3.2	2.5	113	4577.2	0.0247

Longitudinal Cracks: The length of longitudinal cracks is insignificant. Several of the longitudinal cracks were along the beams. The longitudinal crack densities (crack length per deck area) range from 0.0040 to 0.0100 ft/ft^2 for the 3 spans investigated.

TABLE 9: Measured Longitudinal Cracks on the Bridge Decks

Traverse Cracks	Count	Length Range (feet)	Mean Length of Cracks (feet)	Median Length of Cracks (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span A	10	1 to 4	2.5	2.25	24.5	4577.2	0.0054
Span B	7	1 to 4	2.7	3	19	4809.6	0.0040
Span C	13	2 to 5	3.5	3	46	4577.2	0.0100

Maximum Crack Width

The maximum width of transverse cracks was measured to be 0.020 in. According to ACI 201, these crack widths are classified as hairline cracks.

General Condition of the Deck Underside

The underside of the deck exhibits no signs of distress, as illustrated by photos 3 and 4.

General Condition of the Girders

The girders were inspected without the aide of any access equipment. No signs of distress were noted on any of the girders.

It should be noted that no concrete core samples were taken during our visual inspection of the bridge decks.

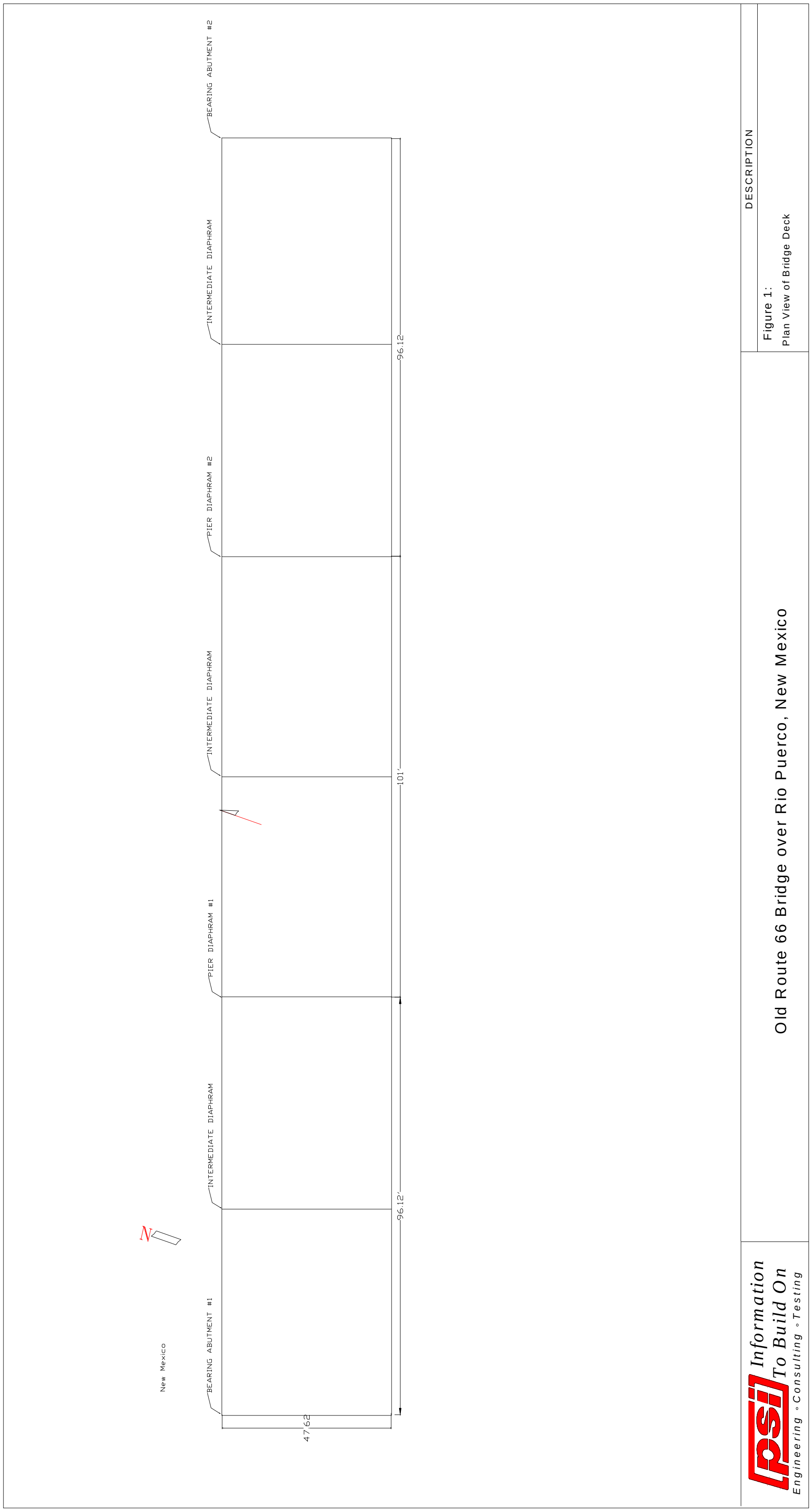
Preliminary Conclusions

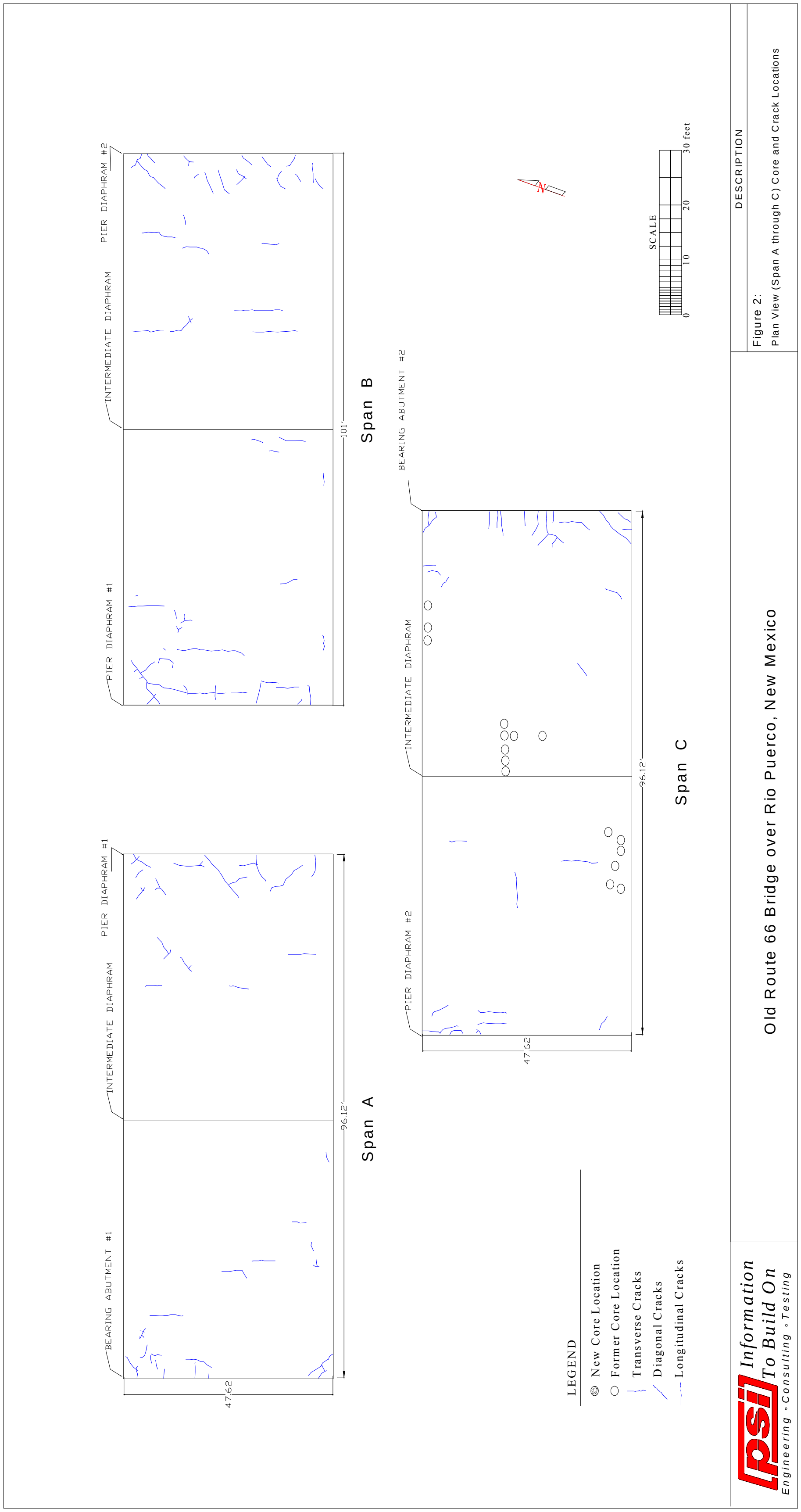
The construction of old Route 66 Bridge over Rio Puerco, west of Albuquerque, New Mexico, was the first HPC bridge project by New Mexico Highway and Transportation Department. It replaces an old bridge built in 1933 (see photo 1). The Rio Puerco Bridge was completed and opened to traffic in December 2000. Researchers from University of New Mexico and New Mexico State University undertook the research project to monitor the long-term behavior of HPC bridge. HPC has been used on many other projects in New Mexico since then.

The visual inspection of the bridge decks as part of our study was performed about three and half years after the bridge opened to traffic. A total of 169 cracks (50 traverse cracks, 30 longitudinal cracks, and 89 diagonal cracks) were recorded during visual survey of the bridge decks (see Figure 2). The sum of crack lengths was 651.3 ft over a bridge deck area of 13,964.1 ft^2. Crack density (total crack length / deck area) for the eastbound and westbound bridges combined was calculated to be 0.047 ft/ft^2. All cracks on the bridge were hairline cracks with a width of less than 0.031 in. No major distresses were observed in our bridge survey. Majority of the cracks observed were short and randomly distributed diagonal cracks (see photos 6 and 10). The three spans have similar bridge deck width and length. Cracks were typically limited at span ends. Other defects such as small surface spalls occurred due to breaking of tined edges or the crack edges were observed.

Considering the heavy ADT on the bridge, the Rio Puerco Bridge was in good condition. HPC designs provide significantly higher strength that can lead to more efficient designs and improved durability.

Respectfully Submitted,
Professional Service Industries, Inc.
Structural Investigation & Petrography Department


BEARING ABUTMENT #2
INTERMEDIATE DIAPHRAM
PIER DIAPHRAM #2
INTERMEDIATE DIAPHRAM
PIER DIAPHRAM #1
INTERMEDIATE DIAPHRAM
BEARING ABUTMENT #1
96.12'
101'
96.12'
47.62'
N
New Mexico
DESCRIPTION
Figure 1:
Plan View of Bridge Deck
Old Route 66 Bridge over Rio Puerco, New Mexico
PSI
Information
To Build On
Engineering • Consulting • Testing



BEARING ABUTMENT #1
INTERMEDIATE DIAPHRAM
PIER DIAPHRAM #1
INTERMEDIATE DIAPHRAM
PIER DIAPHRAM #2
Span A
96.12'
47.62
Span B
101'
BEARING ABUTMENT #2
INTERMEDIATE DIAPHRAM
PIER DIAPHRAM #2
Span C
96.12'
47.62
LEGEND
New Core Location
Former Core Location
Transverse Cracks
Diagonal Cracks
Longitudinal Cracks
SCALE
0
10
20
30 feet
N
DESCRIPTION
Figure 2:
Plan View (Span A through C) Core and Crack Locations
Old Route 66 Bridge over Rio Puerco, New Mexico
PSI
Information To Build On
Engineering • Consulting • Testing

Photographic Documentation

	Photo 1: View looking from the western end of the historical bridge at the west facing bridge. Description illustrates the history and use of Rio Puerco Bridge.
	Photo 2: View looking from the eastern end of east facing bridge. Historical bridge in the background.
	Photo 3: General condition of underside eastbound bridge.

	Photo 4: General condition of the piles at eastbound bridge. Cracks observed around the pile in the stucco.
	Photo 5: Close up view of longitudinal crack in the traffic lane.
	Photo 6: Close up view of typical diagonal cracks at bridge deck.

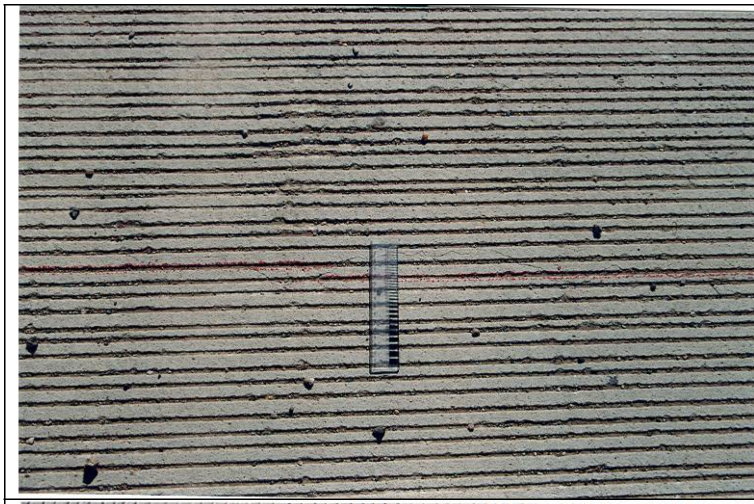
	Photo 7: Close up view of typical transverse crack.
	Photo 8: Close up view of broken tinned edges and polished surface.
	Photo 9: Cores taken by others and sign of broken tinned edges.

	Photo 10: Close up view of the cracks at deck joints.

APPENDIX H – Supplement 1

Old Route 66 Bridge over Rio Puerco, New Mexico
Survey Checklist

Checklist

The following checklist is adapted from 201.1R-92, and provided to facilitate a thorough survey. The definition of terms and associated photographs in 201.1 R-92, are utilized to standardize the reporting of the condition of the concrete in the structures.

1. Description of structure or pavement
   - 1.1 Name, location, type, and size: Old Route 66 Bridge over Rio Puerco, west of Albuquerque, New Mexico opened to traffic in December 2000. It is a three-span structure 293.3 ft long. The clear width of the deck is 47.6 ft, including two through-traffic lanes and two shoulders.
   - 1.2 Owner, project engineer, contractor, when built
     Owner- New Mexico Highway and Transportation Department. This bridge is part of a demonstration project for HPC in bridge structures which were co-sponsored by the Federal Highway Administration (FHWA) and the New Mexico Highway and Transportation Department.
   - 1.3 Design
     - 1.3.1 Architect and/or engineer: the New Mexico Highway and Transportation Department
     - 1.3.2 Intended use and history of use: To carry high volume of traffic on Route 66 over Rio Puerco . Opened to traffic in December 2000.
     - 1.3.8 Special features: HPC with specified strength of 6000 psi at 28 days was used. Class F fly ash was used to mitigate the potential for alkali-silica reactivity.
   - 1.4 Construction
     - 1.4.1 Contractor-general, NA
     - 1.4.2 Subcontractors concrete placement: NA
     - 1.4.3 Concrete Supplier: NA.
     - 1.4.4 Agency responsible for testing: the New Mexico Highway and Transportation Department and New Mexico State University.
     - 1.4.5 Other subcontractors: NA
   - 1.5 Photographs
     - 1.5.1 General view Photos 1 through 2
     - 1.5.2 Detailed close up of condition of area Photos 3 through 10
   - 1.13 Sketch map-orientation showing sunny and shady and well and poorly drained regions N/A
2. Present condition of structure Date of Evaluation The week of June 7, 2004
   - 2.1 Overall alignment of structure No signs of misalignment
     - 2.1.1 Settlement
     - 2.1.2 Deflection
     - 2.1.3 Expansion
     - 2.1.4 Contraction
     - 2.2 Portions showing distress (beams, columns, pavement, walls, etc., subjected to strains and pressures) None Observed
     - 2.3 Surface condition of concrete
     - 2.3.1 General (good, satisfactory, poor, dusting, chalking, blisters) Good

2.3.2 Cracks Longitudinal, transverse, and diagonal
2.3.2.1 Location and frequency See Figure 2
2.3.2.14 Type and size (see Definitions) See Figure 2
Transverse Observed
Width (from Crack comparator) Hairline
Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)
Craze N/A
Width (from Crack comparator) ____________
Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)
Map N/A
Width (from Crack comparator) _______________
Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)
D-Cracking N/A
Width (from Crack comparator) ____________
Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)
Diagonal NA
Width (from Crack comparator) NA
Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)
2.3.2.15 Leaching, stalactites N/A
2.3.3 Scaling N/A
2.3.3.1 Area, depth __________________
2.3.3.8 Type (see Definitions) __________________
Light (Less than 1/8 in.)
Medium (1/8 in. – 3/8 in.)
Severe (3/8 in. – 3/4 in.)
Very Severe (Greater than 3/4 in.)
2.3.4 Spalls and popouts None observed
2.3.4.1 Number, size, and depth NA
2.3.4.8 Type (see Definitions) NA
Spalls
Small (Less than 3/4 in. depth)

Large (Greater than 3/4 in. depth)
Popouts
Small (Less than 3/8 in. diameter)
Medium (3/8 in. – 2 in. diameter)
Large (Greater than 2 in. diameter)

2.3.5 Extent of corrosion or chemical attack, abrasion, impact, cavitation N/A
2.3.6 Stains, efflorescence none observed
2.3.7 Exposed reinforcement None
2.3.8 Curling and warping N/A
2.3.9 Previous patching or other repair N/A
2.3.10 Surface coatings N/A
2.3.10.1 Type and thickness N/A
2.3.10.2 Bond to concrete N/A
2.3.10.3 Condition N/A
2.3.11 Abrasion N/A
2.3.12 Penetrating sealers ___
2.3.12.1 Type N/A
2.3.12.2 Effectiveness N/A
2.3.12.9 Discoloration N/A

2.4 Interior condition of concrete (in situ and samples) N/A
2.4.1 Strength of cores
2.4.2 Density of cores
2.4.3 Moisture content
2.4.4 Evidence of alkali-aggregate or other reactions N/A
2.4.5 Bond to aggregate, reinforcing steel, joints N/A
2.4.6 Pulse velocity
2.4.7 Volume change
2.4.8 Air content and distribution
2.4.9 Chloride-ion content
2.4.10 Cover over reinforcing steel
2.4.11 Half-cell potential to reinforcing steel.
2.4.12 Evidence of reinforcement corrosion
2.4.13 Evidence of corrosion of dissimilar metals
2.4.20 Delaminations N/A
2.4.20.1 Location N/A
2.4.20.2 Number, and size N/A
2.4.15 Depth of carbonation
2.4.16 Freezing and thawing distress (frost damage)
2.4.17 Extent of deterioration
2.4.24 Aggregate proportioning, and distribution

3. Nature of loading and detrimental elements
3.1 Exposure
3.1.1 Environment (arid, subtropical, marine, freshwater, industrial, etc.) arid
3.1.2 Weather-(July and January mean temperatures, 92ºF and 47ºF

		Mean annual rainfall and Months in which 60 percent of it occurs)	8.4-in August
	3.1.3	Freezing and thawing	Minimal
	3.1.4	Wetting and drying	Minimal
	3.1.5	Drying under dry atmosphere	N/A
	3.1.6	Chemical attack-sulfates, acids, chloride	N/A
	3.1.7	Abrasion, erosion, cavitation, impact	N/A
	3.1.8	Electric currents	N/A
	3.1.9	Deicing chemicals which contain chloride ions	N/A
	3.1.10	Heat from adjacent sources	N/A
3.2	Drainage		N/A
	3.2.1	Flashing	
	3.2.2	Weepholes	
	3.2.3	Contour	
	3.2.4	Elevation of drains	
3.3	Loading	Research Test Data Available in Compilation CD Version 3	
	3.3.1	Dead	
	3.3.2	Live	
	3.3.3	Impact	
	3.3.4	Vibration	
	3.3.5	Traffic index	
	3.3.6	Other	
3.4	Soils (foundation conditions)		
	3.4.1	Compressibility	
	3.4.2	Expansive soil	
	3.4.3	Settlement	
	3.4.4	Resistivity	
	3.4.5	Evidence of pumping	
	3.4.6	Water table (level and fluctuations)	

4.	Original condition of structure			Good
4.1	Condition of formed and finished surfaces			Good
	4.1.1	Smoothness		
	4.1.2	Air pockets ("bugholes")		
	4.1.3	Sand streaks		
	4.1.4	Honeycomb		
	4.1.5	Soft areas (retarded hydration)		
	4.1.6	Cold joints		
	4.1.19	Staining		
	4.1.20	Sand pockets		
4.2	Defects			N/A
	4.2.1	Cracking		
		4.2.1.1	Plastic shrinkage	
		4.2.1.2	Thermal shrinkage	
		4.2.1.3	Drying shrinkage	
	4.2.8	Curling		

5. Materials of Construction See Table 2

6. Construction Practices See Report pg. 3 and 6

APPENDIX I

U.S. 401 Bridge, Raleigh, North Carolina

HIGH PERFORMANCE CONCRETE BRIDGE DECK INVESTIGATION

US 401 Bridge Over the Neuse River
Raleigh, North Carolina

I. BACKGROUND

The US 401 bridge over the Neuse River in Wake County, just north of Raleigh, North Carolina, is the first High Performance Concrete (HPC) Bridge built in North Carolina (see photos 1 through 3). The US 401 bridge consists of two parallel structures. HPC was used in the girders and decks of the northbound and southbound bridges. After the completion of the northbound bridge, it opened to traffic in July 2000. The southbound US 401 bridge opened to traffic in September 2002.

The US 401 bridge has four spans on both the southbound and northbound sides — two spans of 91.9 ft (28 m) using AASHTO Type IV girders and two spans of 57.4 ft (17.5 m) using the AASHTO Type III girders. The overall length of the bridge is 91 m (299 ft). Each bridge is 14.4-m (47.1-ft) wide and carries a 12.0-m (39.4-ft) roadway section and a 1.9-m (6.2-ft) sidewalk. The 215-mm (8.5-in.) thick deck was placed on a stay-in-place metal form. The AASHTO Type IV prestressed concrete I-girders are 1.37-m (54-in.) deep and the AASHTO Type III prestressed I-girders are 1.15-m (45-in.) deep. There were five girders per span at 3.12 m (10.25 ft) on center. Girders were pretensioned with 15.2-mm (0.6-in.) diameter draped and straight strands. The use of 10,000 psi (69 MPa) HPC in the girders and 6000 psi (41 MPa) HPC in the deck allowed the designer to reduce the number of girder lines from six to five.

The US 401 bridge is part of a demonstration project for HPC in bridge structures which are co-sponsored by the Federal Highway Administration (FHWA) and the North Carolina Department of Transportation (NCDOT). As part of this program, North Carolina State University (NCSU) undertook a research project that consisted of the tasks of providing instrumentation and monitoring four prestressed HPC girders used in the bridge. The structures are intended to be compared for relative durability and performance based on the extensive use of HPC. Completion of the US 401 bridge proves that it is feasible to construct an HPC bridge in North Carolina with local materials and local contractors.

II. SCOPE OF SERVICES

Professional Service Industries (PSI) is under contract with the Federal Highway Administration (FHWA) to conduct bridge deck inspections for HPC bridges. Our scope of services on this bridge included a series of tasks and sub-tasks, which are described as follows:

- Collect all available information relevant to the construction of each bridge, including;
    - Deck Concrete Properties

- Specified Deck Concrete Construction Procedures
- Approved Concrete Mix Proportions
- Measured Properties from QC
- Other Measured Properties
- Actual Method of Deck Placement
- Average Daily Traffic (ADT)
- Exposure Condition of the Bridge
- Any Performed Maintenance
- Any Inspection Reports
- Visually inspect the bridge, and obtain the following:
    - General condition of the deck top surface
    - Determination of the maximum crack width
    - General condition of the deck underside
    - General condition of the girders
    - Photograph areas of significant deterioration
    - Prepare drawings locating defects
    - Extract 13 concrete core samples including 6 for RCPT tests at NCDOT

III. COMPILATION OF BRIDGE CONSTRUCTION INFORMATION

Information sources used for this report include bridge drawings, research report from NCDOT, North Carolina State University (FHWA/NC/2002-003), and technical information contained in FHWA's "Compilation and Evaluation of Results from High Performance Concrete Bridge Projects" version 3.

Deck Concrete Properties

HPC was used in the cast-in-place concrete deck. The water-to-cement ratio was specified to be 0.33. The strength requirement for the cast-in-place concrete was 41 MPa (6,000 psi) at 28 days. The mixture proportion for the deck concrete included a 20% fly ash replacement of Portland cement.

TABLE 1: Specified Concrete Properties

Property	Cast-in-place Deck
Max. Water/Cementitious Materials Ratio:	0.43
Min. Percentage of Fly C Ash:	0
Max. Percentage of Fly C Ash:	0
Min. Percentage of Fly F Ash:	20
Max. Percentage of Fly F Ash:	20
Min. Percentage of Silica Fume:	0
Max. Percentage of Silica Fume:	0
Slump:	127 mm
Air Content:	4.5~7.5%
Compressive Strength - Design:	41 MPa @ 28 days

Specified Deck Concrete Construction Procedures

General requirement for the curing of HPC used in the US 401 Bridge includes the use of a curing medium consisting of burlap under polyethylene sheets. The burlap or other approved curing medium must be wet when placed on the deck and kept moist for a minimum of seven curing days. Water must be applied to the curing medium through soaker hoses or other methods approved by the Engineer. Water must be applied to the deck in amounts that keep the medium moist but there is no flow or ponding on the deck. The Membrane Curing Compound Method was not allowed. A test panel was required for each deck pour.

Approved Concrete Mix Proportions

Holnam Type I/II cement was used in the cast-in-place deck of the US 401 Bridge. The approved proportions for cast-in-place decks are shown in Table 2.

TABLE 2: Approved Mix Proportions for the US 401 Bridge

Mix Parameters	Cast-in-Place Deck
Cement Brand:	Holnam
Cement Type:	I/II
Cement Quantity:	348 kg/m^3
Fly Ash Brand	Roanoke
Fly Ash Type:	F
Fly Ash Quality:	104 kg/m^3
Fine Aggregate FM:	2.36
Fine Aggregate SG	2.65
Fine Aggregate Quantity:	595 kg/m^3
Coarse Aggregate, Max. Size:	25 mm
Coarse Aggregate Quantity	1083 kg/m^3
Coarse Aggregate SG:	2.63
Water:	148 kg/m^3
High Range Water Reducer Brand:	Adva 100
High Range Water Reducer Type:	F
High Range Water Reducer Quantity:	As required
Retarder Brand:	Daratard 17
Retarder Type:	B and D
Air Entrainment Brand:	Daravair 1000
Air Entrainment Type:	Saponified Rosin
Air Entrainment Quantity	As required
Water/Cementitious Materials Ratio:	0.33

Measured Properties from QC Tests of Production Concrete

Cast-in-Place Deck

Measured properties of concrete mix for the cast-in-place deck are summarized in Table 3. For the northbound bridge, the average strength from the cylinders was 7150 psi (49 MPa), well above the specified 6000 psi (41 MPa) at 28 days. For the southbound bridge, three of the five sets of cylinders with an average strength of 5700 psi (39.3 MPa) at 28 days were accepted with the assumption that the strengths would increase to 6000 psi (41 MPa) at 56 days. The other two sets of cylinders had strength values well below the required strength, with one having a 4100 psi (28.3 MPa) strength at 28 days. The reason for this lower strength for the southbound cylinders is not known.

TABLE 3: Measured Properties of Approved Concrete Mixes for Cast-in-Place Deck

Property	Value/Comment
Actual Curing Procedure for Deck:	Wet for 7 days
Slump	4-5 in
Air Content	5.7-6.8 %
Compressive Strength	Northbound 7150 psi at 28 days Southbound 5700 psi at 28 days

Measured Properties from Research Tests of Production Concrete in Cast-in-Place Deck

Research tests on the production concrete for the deck showed that the compressive strengths at 28 days ranged from 5310 to 6700 psi. By 56 days, some of the cylinders still had not reached the specified 6000 psi (41 MPa), as shown in Table 4.

TABLE 4: Measured Compressive Strength of Production Concrete Used in the Southbound Cast-in-Place Deck

Compressive Strength:	5310 to 6700 psi at 28 days 5900 to 7020 psi at 56 days

Actual Method of Deck Placement

The northbound bridge construction was completed in Spring 2000 and the bridge opened to traffic in July 2000. The southbound bridge opened to traffic in September 2002.

The concrete was placed into the forms using an overhead bucket. Both standard vibrators and an external vibrator on the side-form were utilized to ensure proper placement. Fogging of the concrete deck started when the concrete was in the plastic state. This procedure avoided the surface moisture evaporation and plastic shrinkage cracks. This construction practice is particularly important for HPC. The deck was cured using wet burlap for 7 days. Wet burlaps were kept moist.

Average Daily Traffic (ADT)

Average daily traffic for both eastbound and westbound lanes was calculated based on a count of all vehicles crossing the bridge during a 15 minutes period beginning at 1310 hrs on April 7, 2004. These vehicle counts gave an ADT of 2,688. The estimation of traffic flow was made while the PSI inspection crew was on-site.

Exposure Condition of the Bridge

The US 401 Bridges near Raleigh, North Carolina experience wide ranges of conditions throughout the year. The National Weather Service reports that the average maximum daily temperature varies between 89°F in July and 50°F in January. The average daily minimum temperature varies between 58°F in July and 30°F in January. The Raleigh area experiences about 75 days per year in which air temperatures drop below 32°F, implying a considerable number of freeze-thaw cycles. The possibility of below freezing temperatures and the fact that the area receives on average about 7 inches of snow per year, suggest that the roads are treated for ice and snow. The average precipitation varies between 4.5 inches per month in July to 3.05 inches per month in December, with an annual average total of 42.4 inches. Considerable precipitation throughout the year implies that the bridge experiences many wet/dry cycles.

Performed Maintenance

No documents were found which would indicate any maintenance had been performed since the bridge was constructed in 2000.

Inspection Reports

As part of the project, North Carolina State University (NCSU) undertook a research project that consisted of the tasks of providing instrumentation and monitoring four prestressed HPC girders used in the bridge. The researchers have developed an instrumentation program to monitor the structural performance of the bridge and its components as described in "The Behavior of Prestressed High Performance Concrete Bridge Girders for US Highway 401 over the Neuse River in Raleigh, NC", Report FHWA/NC/2002-003.

IV. BRIDGE DECK INSPECTION

PSI personnel performed a visual inspection of the bridge decks during the week of April 7, 2004. The results of the inspection are summarized as follows.

General Condition of the Deck Top Surface

Figure 1 shows the general layout of the decks for the US 401 Bridge. Results of visual inspection of the decks are shown in Figure 2a and Figure 2b. Surface defects observed and documented during visual inspection primarily included transverse cracks, longitudinal cracks, and diagonal cracks (see photos 5 through 7). Other defects observed and documented included small spalls at joints and cracks, broken tinned edges (photo 8), polished surface (photo 9), and failed repairing patches (photo 10). However, apparent signs of other serious damages such as freeze-thaw, D-cracking, alkali-silica reaction, and alkali-aggregate reaction were not observed. Both longitudinal and diagonal cracks observed on the top surface of cast-in-place deck were marginal. Cracking can be seen on the concrete barrier wall along the bridge.

A total of 166 cracks (129 traverse cracks, 30 longitudinal cracks, and 7 diagonal cracks) were recorded during visual survey of the bridge decks (see Figure 2). The sum of crack lengths was 1,308.5 ft over a bridge deck area of 23,501 ft^2. Crack density (total crack length / deck area) for the northbound and southbound bridges combined was calculated to be 0.056 ft/ft^2.

It is noted that the number of transverse cracks accounts for majority of the cracks measured. Out of the total crack length of 1,310.9 ft, 1,236.3 ft was measured for transverse crack (94.3%). Longitudinal cracks and diagonal cracks are short and most often seen at the joints between the spans. Compared to all other spans, Span A and Span B have more cracks. All cracks measured are hairline crack with a width of less than 0.031 in. Typical cracks on the bridge decks are shown in photos 5 through 8, respectively.

Cracks were typically limited at span ends. Small surface spalls, which either occurred due to breaking of tined edges or the crack edges, were observed. Figure 2a and Figure 2b also illustrates the locations of drilled cores from our investigation and previous work done by others.

The number, length and density of cracks for entire bridge decks in both directions are shown in Tables 8 through 11, and described below according to the crack type.

Transverse Cracks: Figure 2 illustrates the transverse cracks that were identified on the surface of the bridge decks. Tables 5 and 6 provide the detailed information regarding transverse cracks identified on the bridge decks. The crack densities (crack length per deck area) range from 0.0274 to 0.0568 ft/ft^2 for the 4 spans investigated.

TABLE 5: Measured Transverse Cracks on the Northbound Bridge Decks

Northbound Traverse Cracks	Count	Length Range (feet)	Mean Length of Cracks (feet)	Median Length of Cracks (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span A	16	3 to 26	10.4	9.5	166.5	3615.6	0.0461
Span B	31	3 to10	32.2	6	205.5	3615.6	0.0568
Span C	7	3 to 22	14.3	7	62.0	2259.6	0.0274
Span D	12	2 to 21	17.3	7.5	88	2259.6	0.0389

TABLE 6: Measured Transverse Cracks on the Southbound Bridge Decks

Southbound Traverse Cracks	Count	Length Range (feet)	Mean Length of Cracks (feet)	Median Length of Cracks (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span A	30	2 to 37	11.1	7	332.75	3615.6	0.0920
Span B	29	3 to 37	12.4	9	359.5	3615.6	0.0994
Span C	2	3 to 11	7	7	14	2259.6	0.0062
Span D	2	4 to 5	4.5	4.5	9	2259.6	0.0040

Diagonal Cracks: The diagonal crack densities (crack length per deck area) ranges from 0.0022 to 0.0025 ft/ft^2 for Span A northbound bridge and southbound bridges, respectively. Diagonal cracks in Span A bridge decks typically present near the joints. Diagonal cracks were not observed in other spans.

TABLE 7: Measured Diagonal Cracks on the Northbound Bridge Decks

Northbound Traverse Cracks	Count	Length Range (feet)	Mean Length of Cracks (feet)	Median Length of Cracks (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span A	2	3 to 6	4.5	4.5	9	3615.6	0.0025
Span B	NA	NA	NA	NA	NA	3615.6	NA
Span C	NA	NA	NA	NA	NA	2259.6	NA
Span D	NA	NA	NA	NA	NA	2259.6	NA

TABLE 8: Measured Diagonal Cracks on the Southbound Bridge Decks

Southbound Traverse Cracks	Count	Length Range (feet)	Mean Length of Cracks (feet)	Median Length of Cracks (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span A	5	1 to 2.2	1.6	1.67	8	3615.6	0.0022
Span B	NA	NA	NA	NA	NA	3615.6	NA
Span C	NA	NA	NA	NA	NA	2259.6	NA
Span D	NA	NA	NA	NA	NA	2259.6	NA

Longitudinal Cracks: The number and length of longitudinal cracks are mostly insignificant. Several of the longitudinal cracks were along the beams.

TABLE 9: Measured Longitudinal Cracks on the Northbound Bridge Decks

Northbound Traverse Cracks	Count	Length Range (feet)	Mean Length of Cracks (feet)	Median Length of Cracks (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span A	NA	NA	NA	NA	NA	3615.6	NA
Span B	7	1 to 8	3	2.5	21	3615.6	0.0058
Span C	2	3 to 4	3.5	3.5	7	2259.6	0.0031
Span D	3	1 to 1	1	1	3	2259.6	0.0013

TABLE 10: Measured Longitudinal Cracks on the Southbound Bridge Decks

Southbound Traverse Cracks	Count	Length Range (feet)	Mean Length of Cracks (feet)	Median Length of Cracks (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span A	5	0.5 to 4	1.6	1	8	3615.6	0.0022
Span B	7	1 to 2	1.14	1	8	3615.6	0.0022
Span C	NA	NA	NA	NA	NA	2259.6	NA
Span D	6	1 to 2	1.17	1	7	2259.6	0.0031

Maximum Crack Width

The maximum width of transverse cracks was measured to be 0.016 in. According to ACI 201, these crack widths are classified as hairline cracks. The fine width cracks were generally located at span ends and some exhibited spalling due to the breaking of the edges.

General Condition of the Deck Underside

The underside of the deck exhibits no signs of distress. At very limited locations, efflorescence was observed. Photo 4 shows a general view of the underside of the deck.

General Condition of the Girders

The girders were inspected from a motor boat, without the aide of any access equipment. No signs of distress were noted on any of the girders.

Concrete Core Samples

Thirteen cores, 3-¾ inches in diameter, were retrieved from the decks. The core sample locations are shown on Figure 2a and 2b. The locations were evenly distributed along each shoulder of the bridge. The cores were labeled NC-1 through NC-13. Out of the 13 cores, 6 were reserved for NCDOT to perform rapid chloride permeability test, and the other 7 were transferred to FHWA for a petrographic analysis.

TABLE 11: Core Dimensions

Sample	NC-2	NC-4	NC-5	NC-7	NC-8	NC-10	NC-11
Diameter (in.)	3¾	3¾	3¾	3¾	3¾	3¾	3¾
Length (in.)	3½	3½	4.0	2 ½	3½	2½	2½

Preliminary Conclusions

The construction of the US401 Bridge is part of a demonstration project for HPC in bridge structures. The northbound bridge was completed in 2000, and the southbound bridge was completed in 2002. Researchers from North Carolina State University

(NCSU) undertook a research project that provides instrumentation and monitoring four prestressed HPC girders used in the bridge.

The visual inspection of the bridge decks as part of our study was performed about four years after the northbound bridge opened to traffic, and one-and-half years after the southbound US 401 bridge opened to traffic in September 2002. A total of 166 transverse, longitudinal, and diagonal cracks were recorded on the bridge with a combined total crack length of 1,308.5 ft over a bridge deck area of 23,501ft^2. All cracks on the bridge were hairline cracks with a width of less than 0.031 in. No major distresses were observed in our bridge survey.

As compared to other spans in the bridge, transverse cracks are greater in Span A and Span B near the span ends. A higher crack density is calculated for both Span A and Span B. It should be noted that the structural system of the US 401 Bridge is flexible compared to conventional bridges considering the wide beam spacing, large span, and relatively thin deck used. This relatively flexible structural system combined with the heavy ADT on the bridge might have contributed to the development and widening of some cracks.

In general, the work on the US 401 Bridge shows that HPC designs provide significantly higher strength that can lead to more efficient designs and improved durability.

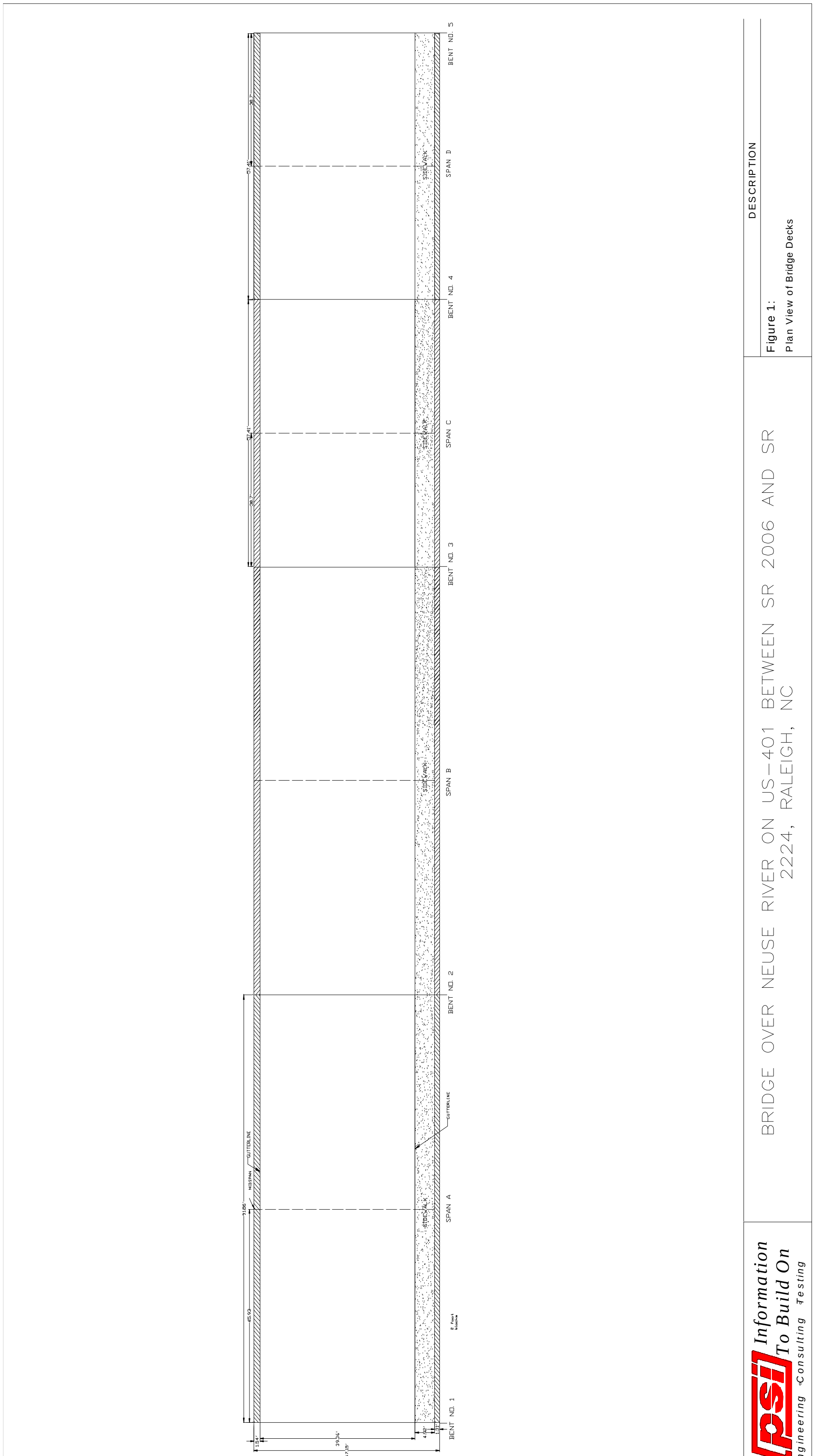
Petrographic analysis of seven core samples retrieved from the decks of the northbound and southbound bridges was performed at TFHRC.

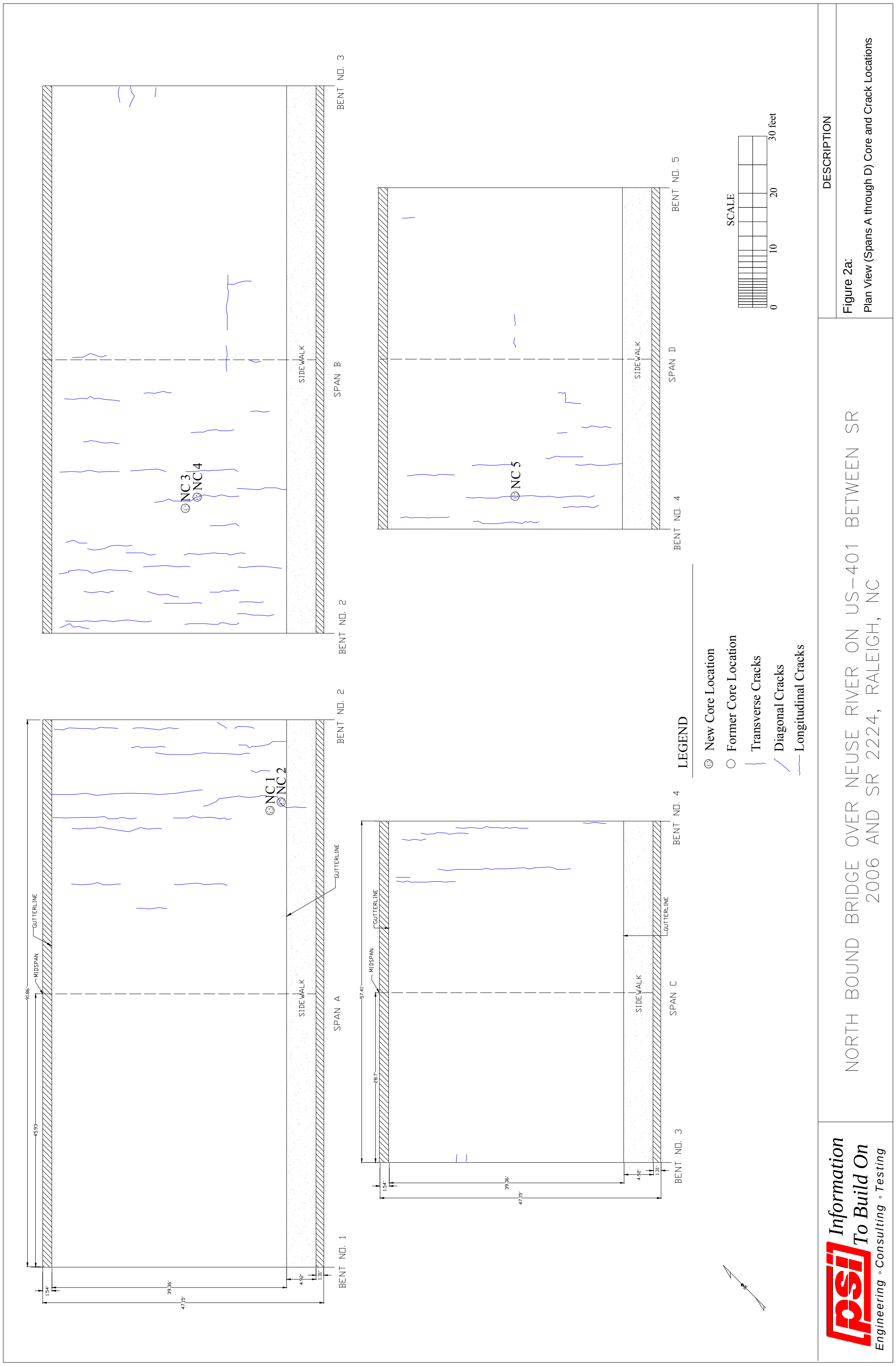
Visual inspection of the concrete cores revealed that four cores (NC-2, NC-4, NC-5, and NC-7) had longitudinal cracks. These cracks ran through coarse aggregate particles as well as in the cement paste, there were no micro-cracks found in the concrete samples. Ettringite was found in air voids, but the occasion was very rare. Cores NC-10 and NC-7 show that the rebar level was about 2 in. below the surface.

The coarse aggregate in the concrete was crushed granite. Most coarse aggregate particles were angular, and the maximum size was about 1 inch. Preferential orientation of coarse aggregate particles was not observed in this concrete. The fine aggregate fraction was natural sand and mainly composed of quartz. The fine aggregate particles appeared rounded to angular.

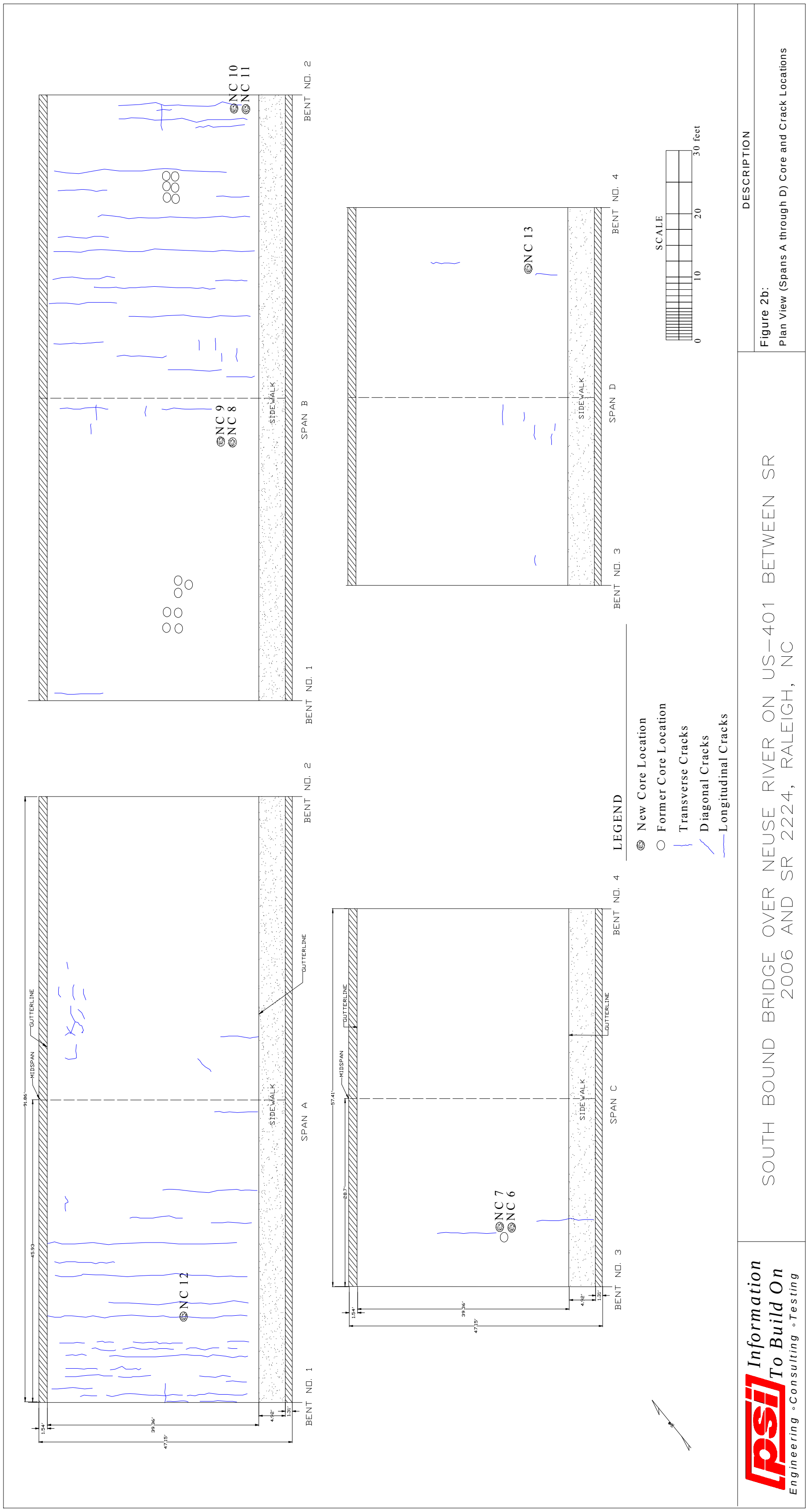
The cement was reasonably hydrated and the paste contained some unhydrated cement particles, fly ash particles were also present in the paste matrix. Small, spherical air voids were observed in the concrete, hence the concrete was air entrained. Entrained air voids were well distributed in the concrete. The air content was estimated to be at a normal level. A small amount of entrapped air voids was also present in the concrete. The paste/aggregate bond appeared to be good.

Respectfully Submitted,
Professional Service Industries, Inc.
Structural Investigation & Petrography Department


BRIDGE OVER NEUSE RIVER ON US-401 BETWEEN SR 2006 AND SR 2224, RALEIGH, NC
DESCRIPTION
Figure 1:
Plan View of Bridge Decks
Information To Build On
Engineering • Consulting • Testing
BENT NO. 1
SPAN A
BENT NO. 2
SPAN B
BENT NO. 3
SPAN C
BENT NO. 4
SPAN D
BENT NO. 5
SIDEWALK
GUTTERLINE
MIDSPAN



BENT NO. 1
BENT NO. 2
BENT NO. 3
BENT NO. 4
BENT NO. 5
SPAN A
SPAN B
SPAN C
SPAN D
SIDEWALK
GUTTERLINE
MIDSPAN
NC 1
NC 2
NC 3
NC 4
NC 5
LEGEND
New Core Location
Former Core Location
Transverse Cracks
Diagonal Cracks
Longitudinal Cracks
SCALE
0
10
20
30 feet
DESCRIPTION
Figure 2a:
Plan View (Spans A through D) Core and Crack Locations
NORTH BOUND BRIDGE OVER NEUSE RIVER ON US-401 BETWEEN SR 2006 AND SR 2224, RALEIGH, NC
psi
Information To Build On
Engineering • Consulting • Testing


SOUTH BOUND BRIDGE OVER NEUSE RIVER ON US-401 BETWEEN SR 2006 AND SR 2224, RALEIGH, NC
DESCRIPTION
Figure 2b:
Plan View (Spans A through D) Core and Crack Locations
LEGEND
New Core Location
Former Core Location
Transverse Cracks
Diagonal Cracks
Longitudinal Cracks
SCALE
0
10
20
30 feet
BENT NO. 1
BENT NO. 2
BENT NO. 3
BENT NO. 4
SPAN A
SPAN B
SPAN C
SPAN D
SIDEWALK
GUTTERLINE
MIDSPAN
NC 6
NC 7
NC 8
NC 9
NC 10
NC 11
NC 12
NC 13
psi
Information To Build On
Engineering • Consulting • Testing

Photographic Documentation

Photo 1: View looking south from the northern end of southbound bridge.

Photo 2: View looking north from the southern end of northbound bridge.

Photo 3: View looking from the northern bank. Photo shows both the northbound and southbound bridges.

	Photo 4: View of the underside of bridge.
	Photo 5: View of the transverse cracks on bridge deck.
	Photo 6: Close view of broken tinned edges along the transverse crack.

Photo 7: Close up view of longitudinal cracks on bridge deck.

Photo 8: Close up view of typical polished surface near the joints.

Photo 9: Close up view of typical pop out.

	Photo 10: Failed patching work on bridge deck.

APPENDIX I – Supplement 1

U.S. 401 Bridge, Raleigh, North Carolina
Petrographic Examination

PETROGRAPHIC EXAMINATION OF CONCRETE CORES FROM A BRIDGE IN NORTH CAROLINA (NC)

A report submitted to the Structures Laboratory at TFHRC

Rongtang Liu
Petrographic Laboratory
FHWA TFHRC
(Reviewed by Richard Meininger, PE; Concrete Laboratory; printed in color 9-12-2006)

August 9, 2006

Introduction

Seven concrete cores were received from the Structures Laboratory of the Turner-Fairbank Highway Research Center for petrographic examination. Reportedly, these cores were collected from a concrete bridge in North Carolina.

The dimension of the concrete cores was 4-in. diameter, 2- to 4-in. long. The identification on the cores was as following: NC-2 Span A, NC-4 Span B, NC-5 Span D, NC-7 Span C, NC-8, NC-10 Span B, and NC-11 Span B (Figure I1-1).

Thin sections, as well as polished slabs, were made from the concrete cores for microscopic examination. Petrographic examination was performed on these samples using optical microscopes.

Visual inspection of the concrete cores revealed that four cores (NC-2, NC-4, NC-5, and NC-7) have longitudinal cracks. Cores NC-10 and NC-7 show that the rebar level was about 2 in. below the surface. The findings from microscopic examination indicate that the concrete has entrained air voids, and the air content is estimated to be at a normal level; the hydration of the cement was reasonable, and the presence of unhydrated cement particles was observed in the cement paste; fly ash particles were also found in the concrete; no cracks of microscopical scale were observed; ettringite as secondary deposit was found on very rare occasion.

Laboratory Procedures

Petrographic examination of the concrete samples was performed in accordance with ASTM C 856, “Standard Practice for Petrographic Examination of Hardened Concrete.” Sections were polished and examined using a stereomicroscope at magnifications up to 200×. Small rectangular blocks were cut from concrete samples. One surface of each block was polished, dried, placed on a petrographic slide with low–viscosity epoxy resin, and reduced to a thickness of approximately 25 micrometers (0.001 in.). These thin sections were examined using a polarized–light microscope at magnification up to 400×, to determine aggregate mineralogy, paste characteristics, and microstructure.

Two ¾-inch thick slabs were sawn from the concrete cores. The slabs were ground and polished to obtain a smooth and plane surface. They were examined using a stereomicroscope at magnifications up to 200×.

Findings

Six thin section samples taken from the received cores were examined using the polarized light microscope. Two polished sections were examined using the stereomicroscope. The findings are summarized as follows:

Aggregate

The coarse aggregate in the concrete is crushed granite. Most coarse aggregate particles are angular, and the maximum size is about 1 inch. Preferential orientation of coarse aggregate particles is not observed in this concrete.

The fine aggregate fraction is natural sand and mainly composed of quartz. The fine aggregate particles appear rounded to angular.

Cement Paste

The cement is reasonably hydrated and the paste contains some unhydrated cement particles as seen under the microscope (Figure I1-2). Fly ash particles are also present in the paste matrix (Figure I1-3).

Air Voids

Small, spherical air voids are observed in the concrete (Figure I1-4), hence the concrete was air entrained. Entrained air voids are well distributed in the concrete. The air content is estimated to be at a normal level. A small amount of entrapped air voids is also present in the concrete (Figure I1-5).

Cement-Aggregate Bonding

The paste/aggregate bond appears to be good, as shown in Figure I1-6.

Cracking

Major cracks are visible in cores NC-2, NC4, NC-5 and NC-7. These cracks run through coarse aggregate particles as well as in the cement paste, as shown in Figures I1-7 and I1-8.

There is no micro-crack found in the concrete samples.

Secondary Deposit

Ettringite was found in air voids, but the occasion is very rare.

Summary

The concrete is air entrained and the air content is estimated to be at a normal level. The entrained air voids are well distributed in the concrete. Cement was reasonably hydrated and unhydrated cement particles are present in the concrete. Fly ash is also found in the concrete.

The bond between the aggregate and paste is good. There is no microscopical cracking in the concrete. Major cracks are visible in four of the seven cores. It is speculated that shrinkage may be the cause of the cracking. However, other factors may also have contributed to the cracking, such as mechanical failure.

Figure I1-1: Seven concrete cores as received.

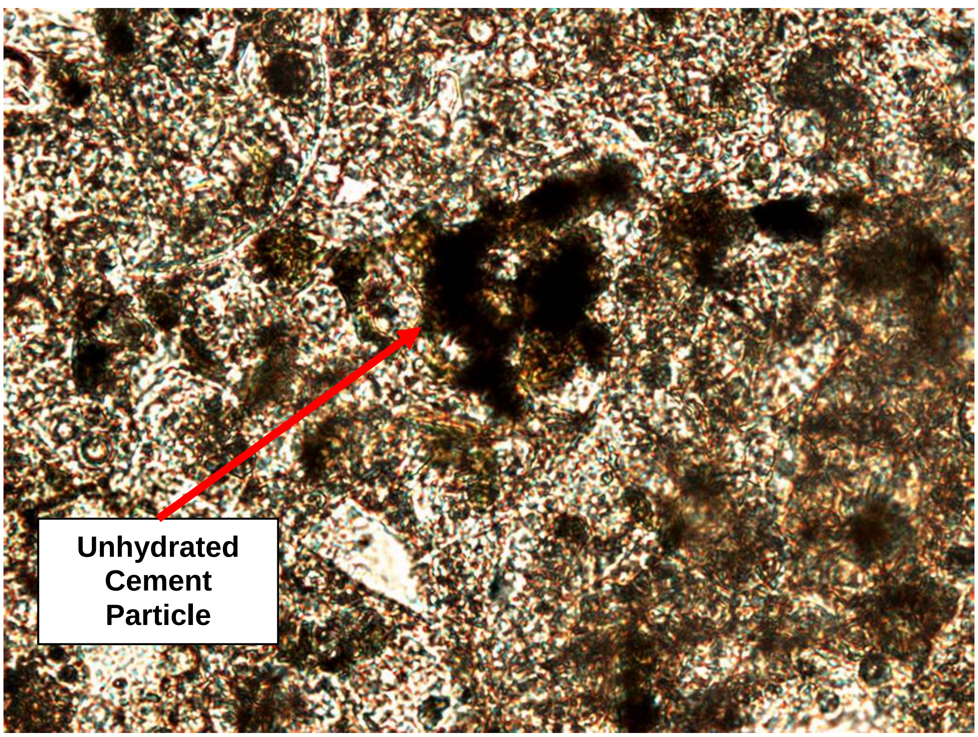


Figure I1-2: Unhydrated cement particle in the paste. Width of field is 0.165 mm. Thin section image.

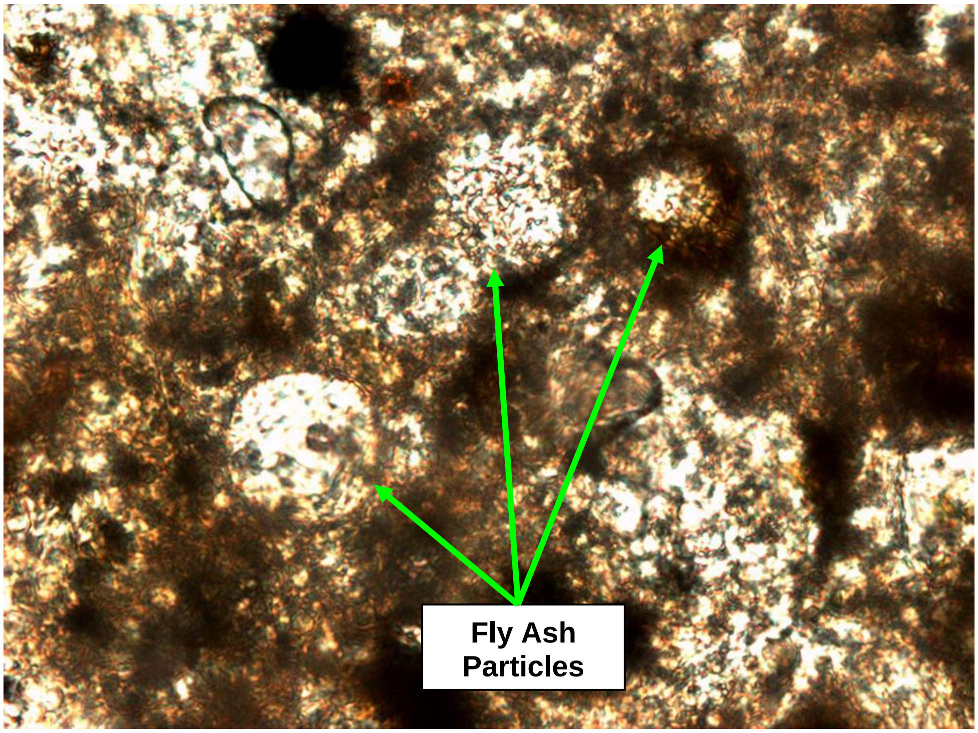


Figure I1-3: Fly ash particles in the cement matrix. Width of field is 0.165 mm. Thin section image.

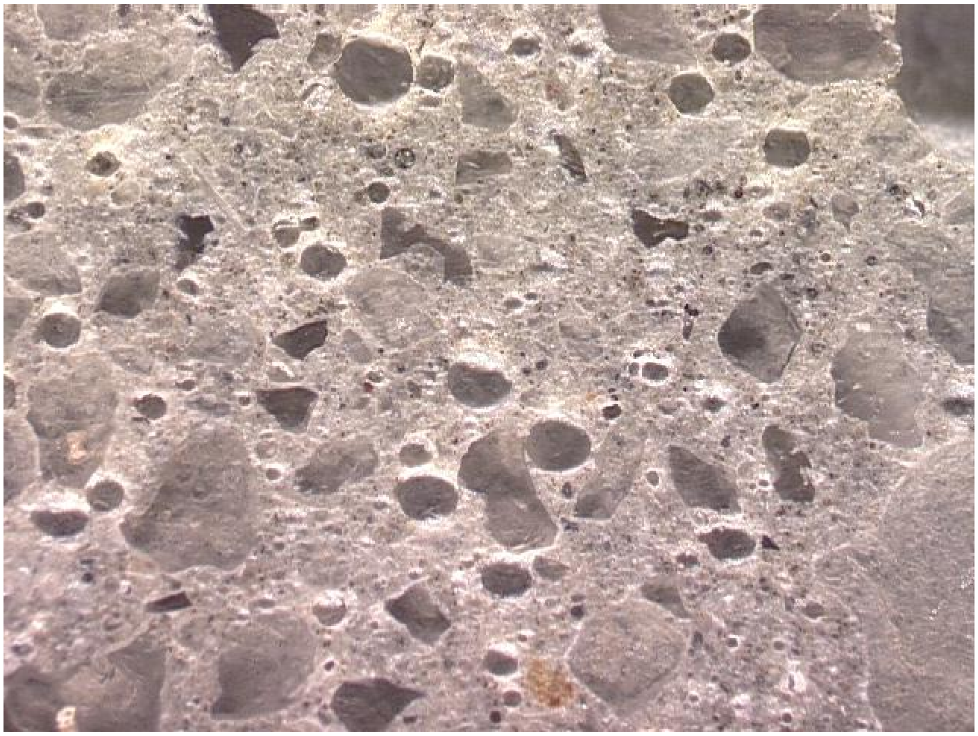


Figure I1-4: Entrained air voids in the concrete. Width of field is 4.0 mm. Polished surface image.

Figure I1-5: An entrapped air void between two aggregate particles. Width of field is 4.0 mm. Polished surface image.

Figure I1-7: The bonding between aggregate and cement paste is good. Width of field is 4.0 mm. Polished surface image.

Figure I1-8. Another image of the major crack in core NC-5. Width of field is 6.5 mm.

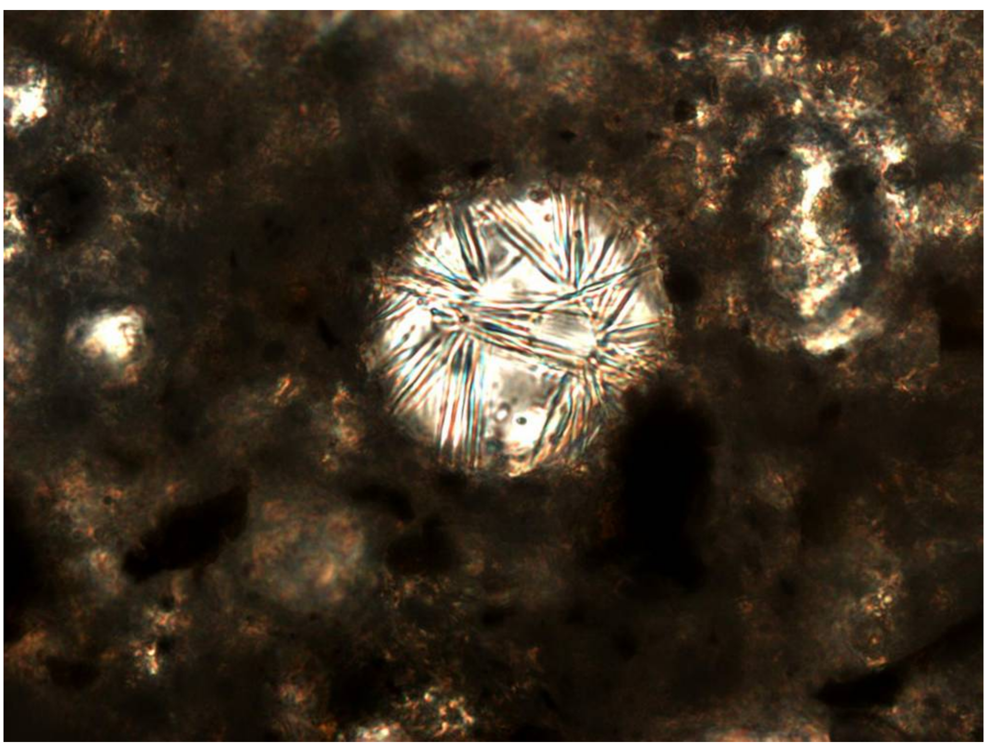


Figure I1-9: Ettringite in an air void. Width of field is 0.165 mm. Thin section image.

APPENDIX I – Supplement 2

U.S. 401 Bridge, Raleigh, North Carolina
Checklist Survey

Checklist

The following checklist is adapted from 201.1R-92, and provided to facilitate a thorough survey. The definition of terms and associated photographs in 201.1 R-92, are utilized to standardize the reporting of the condition of the concrete in the structures.

1. Description of structure or pavement
   - 1.2 Name, location, type, and size
     The US 401 bridge over the Neuse River in Wake County, just north of Raleigh, North Carolina, consists of two parallel structures. The northbound bridge opened to traffic in July 2000. The southbound bridge opened to traffic in September 2002. The US 401 bridge has four spans on both the southbound and northbound sides — two spans of 91.9 ft (28 m) and two spans of 57.4 ft (17.5 m) The overall length of the bridge is 91 m (299 ft). Each bridge is 14.4 m (47.1 ft) wide and carries a 12.0-m (39.4-ft) roadway section and a 1.9-m (6.2-ft) sidewalk.
   - 1.2 Owner, project engineer, contractor, when built
     Owner-North Carolina Department of Transportation. This bridge is part of a demonstration project for HPC in bridge structures which were co-sponsored by the Federal Highway Administration (FHWA) and the North Carolina Department of Transportation (NCDOT). The contractor was W.C. English, Inc. of Lynchburg, VA. The first phase of construction for the northbound bridge was completed in Fall 1999.
   - 1.3 Design
     - 1.3.1 Architect and/or engineer: The North Carolina Department of Transportation (NCDOT)
     - 1.3.2 Intended use and history of use: To carry traffic over the US 401. Northbound bridge opened to traffic in July 2000, and southbound bridge opened to traffic in September 2002.
     - 1.3.3 Special features: Bridge consists of four spans (299-ft in total). The span consists of Type III AASHTO or Type IV AASHTO girders made of precast, prestressed HPC. The use of 10,000 psi (69 MPa) HPC in the girder and 6000 psi (41 MPa) HPC in the deck allowed the designer to reduce the number of girder lines from six to five.
   - 1.4 Construction
     - 1.4.1 Contractor-general W.C. English, Inc. of Lynchburg, VA. Carolina Prestress L.L.C. of Charlotte, NC fabricated the HPC girders.
     - 1.4.2 Subcontractors concrete placement: N/A
     - 1.4.3 Concrete supplier: Southern Concrete Materials of Charlotte, NC
     - 1.4.4 Agency responsible for testing: North Carolina State University
     - 1.4.5 Other subcontractors: N/A
   - 1.5 Photographs
     - 1.5.1 General view — Photos 1 through 3
     - 1.5.2 Detailed close up of condition of area — Photos 4 through 10

1.14 Sketch map-orientation showing sunny and shady and well and poorly drained regions N/A

2. Present condition of structure Date of Evaluation The week of April 7, 2004

2.1 Overall alignment of structure No signs of misalignment

2.1.1 Settlement

2.1.2 Deflection

2.1.3 Expansion

2.1.4 Contraction

2.2 Portions showing distress (beams, columns, pavement, walls, etc., subjected to strains and pressures) Cracks and Efflorescence along the concrete barrier wall

2.3 Surface condition of concrete

2.3.1 General (good, satisfactory, poor, dusting, chalking, blisters) Good

2.3.2 Cracks Transverse

2.3.2.1 Location and frequency See Figure 2a and Figure 2b

2.3.2.16 Type and size (see Definitions) See Figure 2a and Figure 2b

Transverse At the beam diaphragm and panel boundaries

Width (from Crack comparator) Less than 0.03 in..

- Hairline (Less than 1/32 in.)
- Fine (1/32 in. - 1/16 in.)
- Medium (1/16 – 1/8 in.)
- Wide (Greater than 1/8 in.)

Craze N/A

Width (from Crack comparator) ____________

- Hairline (Less than 1/32 in.)
- Fine (1/32 in. - 1/16 in.)
- Medium (1/16 – 1/8 in.)
- Wide (Greater than 1/8 in.)

Map N/A

Width (from Crack comparator) ______________

- Hairline (Less than 1/32 in.)
- Fine (1/32 in. - 1/16 in.)
- Medium (1/16 – 1/8 in.)
- Wide (Greater than 1/8 in.)

D-Cracking N/A

Width (from Crack comparator) ____________

- Hairline (Less than 1/32 in.)
- Fine (1/32 in. - 1/16 in.)
- Medium (1/16 – 1/8 in.)
- Wide (Greater than 1/8 in.)

Diagonal NA

Width (from Crack comparator) NA

Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)

2.3.2.17 Leaching, stalactites N/A

2.3.3 Scaling N/A

2.3.3.1 Area, depth ______

2.3.3.9 Type (see Definitions) ______

Light (Less than 1/8 in.)
Medium (1/8 in. – 3/8 in.)
Severe (3/8 in. – 3/4 in.)
Very Severe (Greater than 3/4 in.)

2.3.4 Spalls and popouts None Observed

2.3.4.1 Number, size, and depth NA

2.3.4.9 Type (see Definitions) NA

Spalls
Small (Less than 3/4 in. depth)
Large (Greater than 3/4 in. depth)

Popouts
Small (Less than 3/8 in. diameter)
Medium (3/8 in. – 2 in. diameter)
Large (Greater than 2 in. diameter)

2.3.5 Extent of corrosion or chemical attack, abrasion, impact, cavitation N/A

2.3.6 Stains, efflorescence Efflorescence at a few locations on the underside of the bridge and along the concrete barrier wall

2.3.7 Exposed reinforcement None

2.3.8 Curling and warping N/A

2.3.9 Previous patching or other repair N/A

2.3.10 Surface coatings N/A

2.3.10.1 Type and thickness N/A

2.3.10.2 Bond to concrete N/A

2.3.10.3 Condition N/A

2.3.11 Abrasion N/A

2.3.12 Penetrating sealers ___

2.3.12.1 Type N/A

2.3.12.2 Effectiveness N/A

2.3.12.10 Discoloration N/A

2.4 Interior condition of concrete (in situ and samples) N/A

2.4.1 Strength of cores

2.4.2 Density of cores

2.4.3 Moisture content

2.4.4 Evidence of alkali-aggregate or other reactions N/A

2.4.5 Bond to aggregate, reinforcing steel, joints N/A

2.4.6 Pulse velocity

2.4.7 Volume change

2.4.8 Air content and distribution
2.4.9 Chloride-ion content
2.4.10 Cover over reinforcing steel
2.4.11 Half-cell potential to reinforcing steel.
2.4.12 Evidence of reinforcement corrosion
2.4.13 Evidence of corrosion of dissimilar metals
2.4.21 Delaminations N/A
2.4.21.1 Location N/A
2.4.21.2 Number, and size N/A
2.4.15 Depth of carbonation
2.4.16 Freezing and thawing distress (frost damage)
2.4.17 Extent of deterioration
2.4.25 Aggregate proportioning, and distribution

3. Nature of loading and detrimental elements
3.1 Exposure
3.1.1 Environment (arid, subtropical, marine, freshwater, industrial, etc.) marine
3.1.2 Weather-(July and January mean temperatures, 89ºF and 50ºF
Mean annual rainfall and 42.4-in
Months in which 60 percent of it occurs) NA
3.1.3 Freezing and thawing 75 days per year air temperatures drop below 32 °F
3.1.4 Wetting and drying Considerable amount of annual exposure
3.1.6 Drying under dry atmosphere N/A
3.1.6 Chemical attack-sulfates, acids, chloride N/A
3.1.7 Abrasion, erosion, cavitation, impact N/A
3.1.8 Electric currents N/A
3.1.9 Deicing chemicals which contain chloride ions N/A
3.1.10 Heat from adjacent sources N/A
3.2 Drainage N/A
3.2.1 Flashing
3.2.2 Weepholes
3.2.3 Contour
3.2.4 Elevation of drains
3.3 Loading Research Test Data Available in Compilation CD Version 3
3.3.1 Dead
3.3.2 Live
3.3.3 Impact
3.3.4 Vibration
3.3.5 Traffic index
3.3.6 Other
3.4 Soils (foundation conditions)
3.4.1 Compressibility
3.4.2 Expansive soil
3.4.3 Settlement

3.4.4 Resistivity
3.4.5 Evidence of pumping
3.4.6 Water table (level and fluctuations)

4.	Original condition of structure	Good
4.1	Condition of formed and finished surfaces	Good
4.1.1	Smoothness	______
4.1.2	Air pockets ("bugholes")	______
4.1.3	Sand streaks	______
4.1.4	Honeycomb	______
4.1.5	Soft areas (retarded hydration)	______
4.1.6	Cold joints	______
4.1.21	Staining	______
4.1.22	Sand pockets	______
4.2	Defects	N/A
4.2.1	Cracking	______
4.2.1.1	Plastic shrinkage	______
4.2.1.2	Thermal shrinkage	______
4.2.1.3	Drying shrinkage	______
4.2.9	Curling	______
5.	Materials of Construction	See Table 2
6.	Construction Practices	See Report pgs. 3 and 5

APPENDIX J

State Route 22 Bridge near Cambridge, Ohio

HIGH PERFORMANCE CONCRETE
BRIDGE DECK INVESTIGATION

State Route 22 Bridge at Milepost 6.57
Near Cambridge, Guernsey County, Ohio

I. BACKGROUND

The State Route 22 Bridge located at Milepost 6.57 (Bridge GUE-22-6.57) in Guernsey County, near Cambridge, Ohio, is the first showcase High Performance Concrete (HPC) Box Girder Bridge built in Ohio. HPC was used in both the beams and the stub abutments. The bridge opened to traffic in November 1998.

Bridge GUE-22-6.57 is a 118.66 ft (35.2 m) single-span structure over Crooked Creek and is composed of 12 side-by-side prestressed concrete box-beams. The total deck thickness is 8.5 in. (216 mm), including a 5.5-in. (140-mm) thick concrete flange and 3-in. (76-mm) thick asphalt wearing surface. The bridge deck has a clear width of 48-ft (14.6-m), including two lanes and two shoulders in southbound and northbound directions.

Originally, the bridge was designed to consist of three spans using 21-in. deep concrete box beams. To lower construction costs by eliminating piers and to improve flow characteristics of the Crooked Creek, Bridge GUE-22-6.57 was redesigned as a single span box girder bridge. 10,000 psi (at 56 days) compressive strength concrete was used. The beams are of type B 42-48 ODOT. Each beam measures 48-in. (1219-mm) wide and 42-in. (1067-mm) deep. The bridge is supported by stub-type abutments on a single row of H-section steel pile supports. All concrete used in Bridge GUE-22-6.57 was required to have a rapid chloride permeability value of below 1000 at 56 days. Concrete mixtures containing silica fume were specified to obtain the required strength and durability requirements. The cast-in-place abutment concrete met the 55-MPa (8000-psi) design strength in 28 days. Using HPC concrete, the box beam’s span range was increased, enabling a lowest cost single span bridge design. In addition, the structure’s service life will be enhanced because of the durability benefits associated with HPC’s low permeability.

Bridge GUE-22-6.57 is part of a demonstration project for HPC in bridge structures, which are co-sponsored by the Federal Highway Administration (FHWA) and the Ohio Department of Transportation (ODOT). The Ohio Department of Transportation (ODOT) conducted research on this bridge with the University of Cincinnati to monitor the long-term behavior of HPC pretensioned concrete girders.

II. SCOPE OF SERVICES

Professional Service Industries (PSI) is under contract with the Federal Highway Administration (FHWA) to conduct bridge deck inspections for HPC bridges. Our scope of services on this bridge included a series of tasks and sub-tasks, which are described as follows:

- Collect all available information relevant to the construction of each bridge, including;
    - Deck Concrete Properties
    - Specified Deck Concrete Construction Procedures
    - Approved Concrete Mix Proportions
    - Measured Properties from QC
    - Other Measured Properties
    - Actual Method of Deck Placement
    - Average Daily Traffic (ADT)
    - Exposure Condition of the Bridge
    - Any Performed Maintenance
    - Any Inspection Reports
- Visually inspect the bridge, and obtain the following:
    - General condition of the deck top surface
    - Determination of the maximum crack width
    - General condition of the deck underside
    - General condition of the girders
    - Photograph areas of significant deterioration
    - Prepare drawings locating defects
    - Extract 7 concrete core samples.

III. COMPILATION OF BRIDGE CONSTRUCTION INFORMATION

Information sources used for this report include bridge drawings, FHWA newsletters and reports, and technical information contained in FHWA's "Compilation and Evaluation of Results from High Performance Concrete Bridge Projects" version 3.

Specified Girder Concrete Properties

It should be noted that the top flange of the box girders serves as a portion of the bridge deck, which was again covered with 3 in. of asphalt wearing surface. The focus of this report therefore will be on the HPC girders.

The water to cementitious materials mass ratio was specified to be 0.28 for the prestressed girders. The design specifies that the girders attain a compressive strength of 10,000 psi at 56 days with an air content of 5-7%. The release strength was specified to be 6000 psi (41 MPa). Table 1 lists the specified concrete properties used in the prestressed box girders.

TABLE 1: Specified Girder Concrete Properties

Property	Cast-in-place Deck
Max. Water/Cementitious Materials Mass Ratio:	0.28
Cementitious Materials Content:	946 lb/yd^3
Min. Quantity of Silica Fume:	100 lb/yd^3
Max. Quantity of Silica Fume:	100 lb/yd^3
Max. Aggregate Size:	3/8 in.
Slump:	6-8 in.
Air Content:	5-7%
Compressive Strength - release of strands:	6,000 psi
Compressive Strength-Design:	10,000 psi @ 56 days
Freeze-thaw resistance:	Dynamic Modulus > 80%
Chloride Permeability:	< 1000 coulombs at 56 days

Specified Concrete Girder Construction Procedures

Before actual construction, two beams should be constructed to determine if the approved HPC mixture would meet the specifications, even in severe conditions. Moreover, the structural behavior of the beams as determined through non-destructive tests should be assessed. The two test beams were also developed so that the fabricator could gain experience with the approved mixture.

The prestressed concrete girders were designed based on 41-MPa (6000-psi) release strength and 69-MPa (10,000-psi) ultimate strength at 56 days. The test cylinders intended to determine the release strengths were to be steam cured along with the girders. Other cylinders were to be cured according to AASHTO T 23 Standard Cure. All cylinders were 6 in. x 12 in., capped with unbonded neoprene caps in testing. One set of cylinders was required for each beam and were tested at release, 7, 28, and 56 days according to the AASHTO T 22 Cylinder Test Method.

Approved Concrete Mix Proportions

Concrete Girders

The State Route 22 bridge concrete mixture for the girders was developed by a research team at University of Cincinnati and contained Type III cement and a water-to-cementitious materials mass ratio of 0.28. The approved proportions for girders are shown in Table 2:

TABLE 2: Approved Mix Proportions for the State Route 22 Bridge

Mix Parameters	Concrete Girders
Cement Brand:	NA
Cement Type:	III
Cement Quantity:	846 lb/yd^3
Silica Fume Brand	Master Builder's Rheomac SF100
Silica Fume Quality:	100 lb/yd^3
Fine Aggregate Type:	Natural River Sand
Fine Aggregate FM:	3.06
Fine Aggregate SG	2.63
Fine Aggregate Quantity:	927 lb/yd^3
Coarse Aggregate, Max. Size:	3/8 in.
Coarse Aggregate Type:	Crushed River Gravel
Coarse Aggregate Quantity	1774 lb/yd^3
Coarse Aggregate SG:	2.71
Water:	262 lb/yd^3
High Range Water Reducer Brand:	Master Builders Rheobuild 1000
High Range Water Reducer Type:	A and F
High Range Water Reducer Quantity:	203 fl. oz./yd^3
Retarder Brand:	Master Builders Pozzolith 100-XR
Retarder Type:	B and D
Retarder Quantity:	28 fl. oz./yd^3
Air Entrainment Brand:	Master Builders MB AE 90
Air Entrainment Type:	Anionic Surfactant
Air Entrainment Quantity	21 oz./yd^3
Water/Cementitious Materials Ratio:	0.28

Measured Properties from QC Tests of Production Concrete for Girders

Measured properties of concrete mixture for the girders are summarized in Table 3, and the compressive strength results are shown in Table 4. The air content was 5.8 % and the slump was 3 in. Most of the test cylinders cured with ASTM C 31 standard curing and steam methods reached strength of 10,000 psi by 28 days, while the specification was 10,000 psi by 56 days (Table 4). One set of cylinders reached strength of 9,720 psi by 28 days, just below the specified value.

TABLE 3: Measured Slump and Air Content of the Approved Concrete Mixes for Girders

Property	Value
Slump	3 in.
Air Content	5.8 %

TABLE 4: Measured Compressive Strength of the Approved Concrete Mixes for Girders

Age, days	Compressive Strength (AASHTO T 22), psi				
	Trial Mix (2)	Beams (1)			
		ASTM Cure (3)		Beam Cure (4)	
		A	B	A	B
1	8935	7285	5880	8490	—
3	9450	8965	7415	9290	8560
7	9805	9870	8125	9720	8850
12	—	10,550	9385	9770	9205
21	—	11,325	9595	10,200	8960
28	10,780	11,540	10,400	10,250	9720
56	—	12,460	11,015	—	—

NOTES:
(1) Strengths obtained during trial mixes.
(2) Properties measured from concrete used in prototype beams A and B.
(3) AASHTO T 23 (ASTM C 31) Standard Cure.
(4) Steam cured with beams, and then moist cured until tested.

Preliminary tests with the approved concrete mixture included measurements of the modulus of elasticity, modulus of rupture, splitting tensile strength, shrinkage, chloride permeability, alkali-silica reactivity, freeze-thaw resistance, and abrasion resistance (Table 5). The freeze-thaw resistance with a modulus of 86.1% exceeded that requested in the specifications (> 80%).

TABLE 5: Measured Properties of the Approved Concrete Mixes for Girders

Age (1)	Beams (2)
Modulus of Elasticity (ASTM C 469), ksi	
56 days	4647
Modulus of Rupture (AASHTO T 97), psi	
7 days, 28 days, 56 days	1080, 1140, 1250
Splitting Tensile Strength (AASHTO T 198), psi	
7 days, 28 days, 56 days	520, 640, 620
Shrinkage (AASHTO T 160), millionths	
150 days	900
Chloride Permeability (AASHTO T 277), coulombs	
28 days, 56 days	342, 358
Freeze-Thaw Resistance (AASHTO T 161, Procedure A), %	
56 days	86.1

NOTES:
(1) All specimens were cured per AASHTO T 23 Standard Cure prior to test age.
(2) Properties were measured from concrete used in prototype beams.

In QC tests of the production concrete for the girders, the air content was found to be 6-7% with an average slump of 6 in. (Table 6), both slightly higher than the approved values (see Table 3).

TABLE 6: Measured Properties of QC of Production Concrete for Girders

Property	Value
Slump	Average: 6 in. Range: 4.75-7.75 in.
Actual Curing Procedure for Girders	Steam for 18 h
Air Content	6-7 %

Measured Properties from Research Tests of Production Concrete for Girders

Compressive Strength

Research tests of the production concrete showed 56 day compressive strengths of greater than 10,000 psi (the specification) for all but beam #7, which had a strength of about 9,500 psi at 56 days (Table 7). Table 8 shows the results of the chloride permeability tests.

TABLE 7: Measured compressive strength from research tests of production concrete for girders.

Beam No.	Age, days			
	1	7	28	56
1	8220	9360	11,810	12,490
2	8130	9490	11,830	12,920
3	7760	9390	11,670	12,270
4	7480	8620	10,590	11,460
5	7330	9250	11,180	11,570
6	9210	9780	11,960	12,420
7	6670	7750	9840	9570
8	7870	8940	11,520	12,030
9	7800	9170	11,670	12,050
10	8010	9050	11,630	11,830
11	7700	8580	10,340	11,100
12	7500	9340	11,460	12,040
Average	7810	9060	11,290	11,810

NOTES: All tests were made on 6x12-in cylinders. Cylinders for release strengths were cured alongside beams. All other cylinders used AASHTO T 23 Standard Cure.

Rapid Chloride Permeability

All concrete used in Bridge GUE-22-6.57 was required to have a rapid chloride permeability value below 1000 at 56 days. At an age of 10.5 months, the measured chloride permeability from research tests of three girder samples is 167, 180, and 292 coulombs, respectively. According to AASHTO T 27, the chloride permeability of production concrete for girders is very low.

Creep and Shrinkage

The creep and shrinkage data for production concrete used in the precast girders is shown in Figure 1 and Figure 2. The creep specimen was a 6x12-in. cylinder, loaded at 4000 psi at a concrete age of 7 days. The shrinkage specimen was a 3x3x11.25-in. prism. The specimens were cured inside molds for 1 day followed by moist room curing until loaded.

Figure 1. Creep measurements (ASTM C 512) for research tests of production concrete for girders.

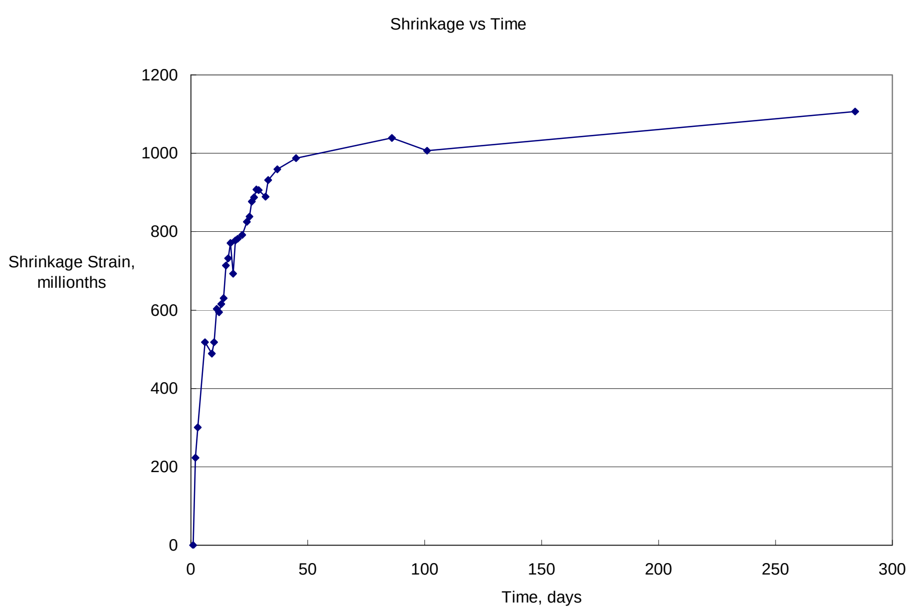


Figure 2. Shrinkage measurements (AASHTO T 160) for research tests of production concrete for girders.

Other Research Tests

The temperature of the beam end blocks, the transfer length, and the camber were measured using two test beams. Fatigue loading, taking into account load distribution and impact factor, designed to simulate the passage of a single HS 20 truck was used to assess the flexural capacity. One test beam was intentionally cracked and then subjected to 653,000 cycles of loading. No fatigue effects were found from deflection and strain data.

The use of 0.6-in. diameter strands allows for the extension of the spans of precast/high strength concrete beams. Transfer lengths were found to be between 36 and 48 in. While the camber at release was calculated to be about 1 in., tests showed it to be 0.25 in. Prestress losses measured at the time the girders were loaded to crack them were 17% and 18%, slightly lower than the 20% calculated with the AASHTO Standard Specifications. The cracking moment calculated with the AASHTO Standard Specifications was about 21% lower than the cracking moment of the two test beams. Finally, the measured flexural strengths of the beams were about 4% greater than the values based on the AASHTO Standard Specifications.

Actual Method of Deck Placement

To allow traffic to use the bridge during construction, the bridge was constructed in two phases. In Phase I, seven HPC beams were installed after half of the old deck was removed. In Phase II, the remaining five beams were placed. Live truckload tests were conducted. These tests consisted of using up to four dump trucks, each weighing approximately 30 kips, placed on the bridge in different arrangements. The measured maximum static deflections were 35 to 50 percent of that allowed by the AASHTO Standard Specifications. With such values, the bridge acts as if the beams are acting together as a single unit. The bridge opened to traffic in November 1998.

Average Daily Traffic (ADT) and Average Daily Truck Traffic (ADTT)

Average daily traffic for the eastbound lanes was about 8,208 cars and 144 trucks, based on a count of all vehicles crossing the bridge during a 10 minute period beginning at 1330 hrs on May 21, 2004. For the westbound lane, these values were about 8,496 cars and 288 trucks per day. The estimation of traffic flow was made while the PSI inspection crew was on-site.

Exposure Condition of the Bridge

Based on climatology, the State Route 22 Bridge in Guernsey County, near Cambridge, OH experiences a wide range of conditions throughout the year. The mean daily maximum temperatures for Columbus, OH (about 80 miles west of Cambridge) range from a low of 34.1^{o}F in January to a high of 83.7^{o}F in July. Mean daily minimum temperatures in Columbus vary between 18.5^{o}F in January and 62.7^{o}F in July. In Akron, OH, 80 miles to the north of Cambridge, the mean daily maximum temperatures range from 32.6^{o}F in January to 82.3^{o}F in July, whereas the mean daily minimum temperatures range from 16.9^{o}F in January to 61.5^{o}F in July. The Cambridge area experiences about 120 days per year in which air temperatures drop below 32^{o}F, implying a considerable number of freeze-thaw cycles. The possibility of below freezing temperatures and the fact that Columbus and Akron receive on average about 28 and 47 inches of snow per year, respectively, suggests that the roads are treated for ice and snow. Southeastern Ohio receives an average monthly precipitation total of between 2.2 inches in January and 4.2 inches in July, with an annual average total of about 38 inches. Considerable precipitation throughout the year implies that the bridge experiences many wet/dry cycles.

Performed Maintenance

No documents that indicate any maintenance had been performed since the bridge was constructed in 1998 have been found. Visual inspection of the top surface of the asphalt overlay indicated that several large longitudinal cracks had been filled with a sealant.

Inspection Reports

As part of the project, bridge instrumentation and bridge monitoring are being performed by University of Cincinnati in cooperation with the ODOT. Live truckload tests and vibrations of the bridge under dynamic loading were conducted. The researchers have also developed an instrumentation program to monitor the structural performance of the bridge girders. The research report “Use of High Performance Concrete for an Abutment Box Beam Bridge, Guernsey County, Ohio, Bridge # GUE-22-0657” provided details of the instrumentation program.

IV. BRIDGE DECK INSPECTION

PSI personnel performed a visual inspection of the bridge decks during the week of May 17, 2004. The results of the inspection are summarized as follows.

General Condition of the Deck Top Surface

Figure 3 shows the general layout of the decks for Bridge # GUE-22-0657. Results of visual inspection of the decks are shown in Figure 4. Surface defects observed and documented during visual inspection primarily included longitudinal cracks, limited transverse cracks, and diagonal cracks (see photos 4 through 7). However, covered with more than 3 in. asphalt overlay, apparent signs of other serious damages such as freeze-thaw, D-cracking, alkali-silica reaction, and alkali-aggregate reaction were not observed. Longitudinal cracks along the traffic lanes have been sealed. Moisture penetration can be seen on the underside of the bridge (see photo 8).

A total of 21 cracks (18 longitudinal cracks, 2 traverse cracks, and 1 diagonal crack) were recorded during visual survey of the bridge decks (see Figure 2). The sum of crack lengths was 310.5 ft over a bridge deck area of 5,695.7 ft^2. Crack density (total crack length / deck area) for the southbound and northbound bridges combined was calculated to be 0.054 ft/ft^2.

Longitudinal cracks account for majority of the cracks recorded. The total longitudinal crack length is 304 ft. Most cracks were sealed, and the accurate crack width measurement is not available. It is noted that the placement of girders adjacent to each other results in longitudinal gaps similar to cracks. Longitudinal cracks were typically in the center lanes and at joints between adjacent beam girders. Figure 4 illustrates the locations of cracks and drilled cores from our inspection.

The number, length and density of cracks for entire bridge decks in both directions are shown in Tables 8. The crack density (crack length per deck area) for longitudinal cracks is 0.053 ft/ft^2. Crack densities for transverse crack and diagonal cracks are negligible.

TABLE 8: Measured Cracks on the Bridge Decks

Crack Type	Count	Length Range (feet)	Mean Length of Cracks (feet)	Median Length of Cracks (feet)	Total Length of Cracks (feet)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Longitudinal	18	1.5 to 48	16.9	9	304	0.0530
Transverse	2	2 to 3	1.5	1.5	5	0.0009
Diagonal	1	1.5	1.5	1.5	1.5	0.0003

Maximum Crack Width

It is noted that most cracks on the bridge deck surface were sealed, and the crack width measurement is not available. In a few locations where cracks were not sealed, the width of crack was measured to be 0.026 in. (0.65 mm) on the asphalt surface. According to ACI 201, these crack widths are classified as fine cracks. The cores drilled through the flange of box girder beam do not show any apparent cracks.

General Condition of the Deck Underside

The underside of the deck exhibits no signs of distress, except for moisture intrusion. Photo 8 shows a general view of the underside of the deck.

General Condition of the Girders

The girders were inspected from a motor boat, without the aide of any access equipment. No signs of distress were noted on any of the girders.

Concrete Core Samples

Seven cores, 4 of 2 ¾ inches in diameter and 3 of 3 ¾ inches in diameter, were retrieved from the decks. The core sample locations are shown on Figure 4. The locations were evenly distributed along the center lanes and shoulder of the bridge. The cores were labeled OH-1 through OH-7 and were transferred to FHWA for further analysis.

TABLE 9: Core Dimensions

Sample	OH-1	OH-2	OH-3	OH-4	OH-5	OH-6	OH-7
Diameter (in.)	2¾	2¾	2¾	2¾	3¾	3¾	3¾
Length (in.)	3	3	2½	1	3	2½	1¾

Preliminary Conclusions

The construction of Bridge # GUE-22-0657 is a research demonstration project developed to show that the use of high performance concrete in box beam bridges can increase the usable span length of structure, thus achieving a more economical bridge design and extending ODOT and FHWA funds. Beyond the structural gains, low permeability and high strength of the production concrete will improve the durability of this type of structure, decreasing the bridge's life cycle costs. The bridge was completed in 1998. Researchers from University of Cincinnati performed material testing, bridge instrumentation, and bridge monitoring throughout this project. Bridge # GUE-22-6.57 has been declared as a "special features" bridge by both the Ohio Department of Transportation (ODOT) and the Federal Highway Administration (FHWA), which means that the bridge has several unusual or experimental features.

The visual inspection of the bridge decks as part of our study was performed about five and half years after the bridge opened to traffic. A total of 21 longitudinal, transverse, and diagonal cracks were recorded on the bridge with a combined total crack length of 310.5 ft over a bridge deck area of 5,695.7 ft^2. No major distresses were observed in our bridge survey.

Bridge # GUE-22-0657 has a deck thickness of 8.5 in. (216 mm), including a 5.5-in. (140-mm) thick concrete flange and 3-in. (76-mm) thick asphalt wearing surface. It appears that the asphalt wearing surface has protected the concrete underneath from cracking and deterioration.

In general, the work on Bridge # GUE-22-0657 shows that HPC designs provide significantly higher strength that can lead to more efficient designs and improved durability.

Petrographic examination was performed on seven concrete cores that were retrieved from the bridge decks. The top surface of three cores appeared to have asphalt. All of the cores showed evidence of being broken off, and not being drilled all the way through. Visual inspection of the concrete cores revealed that core OH-6 had honeycombing. No gross defects were observed in the other cores.

The coarse aggregate in the concrete was gravel containing limestone and dolomite. Coarse aggregate particles were mostly rounded, and the maximum size, measured from the prepared samples, was about 1/2 inch. Preferential orientation of coarse aggregate particles was not observed. The natural sand fine aggregate was composed of quartz, limestone, dolomite, and feldspar. The fine aggregate particles appeared rounded to angular.

The cement was reasonably hydrated with respect to the age of the concrete. The cement paste contained some unhydrated cement particles. In general, the cement/aggregate interface was moderately strong.

The concrete was air entrained, and small, spherical air voids were well distributed in the concrete. A small amount of entrapped air voids was also present in the concrete. Honeycombing was found in core OH-6, indicating that some sections of the concrete was poorly consolidated.

Microscopical examination revealed numerous random cracks throughout the cement paste. Some cracks partially surrounded fine and coarse aggregate particles. In some cases, cracks going through cement paste connected to the cracks at the cement paste/aggregate interface. It was speculated that shrinkage might be the cause of the cracking.

Respectfully Submitted,
Professional Service Industries, Inc.
Structural Investigation & Petrography Department

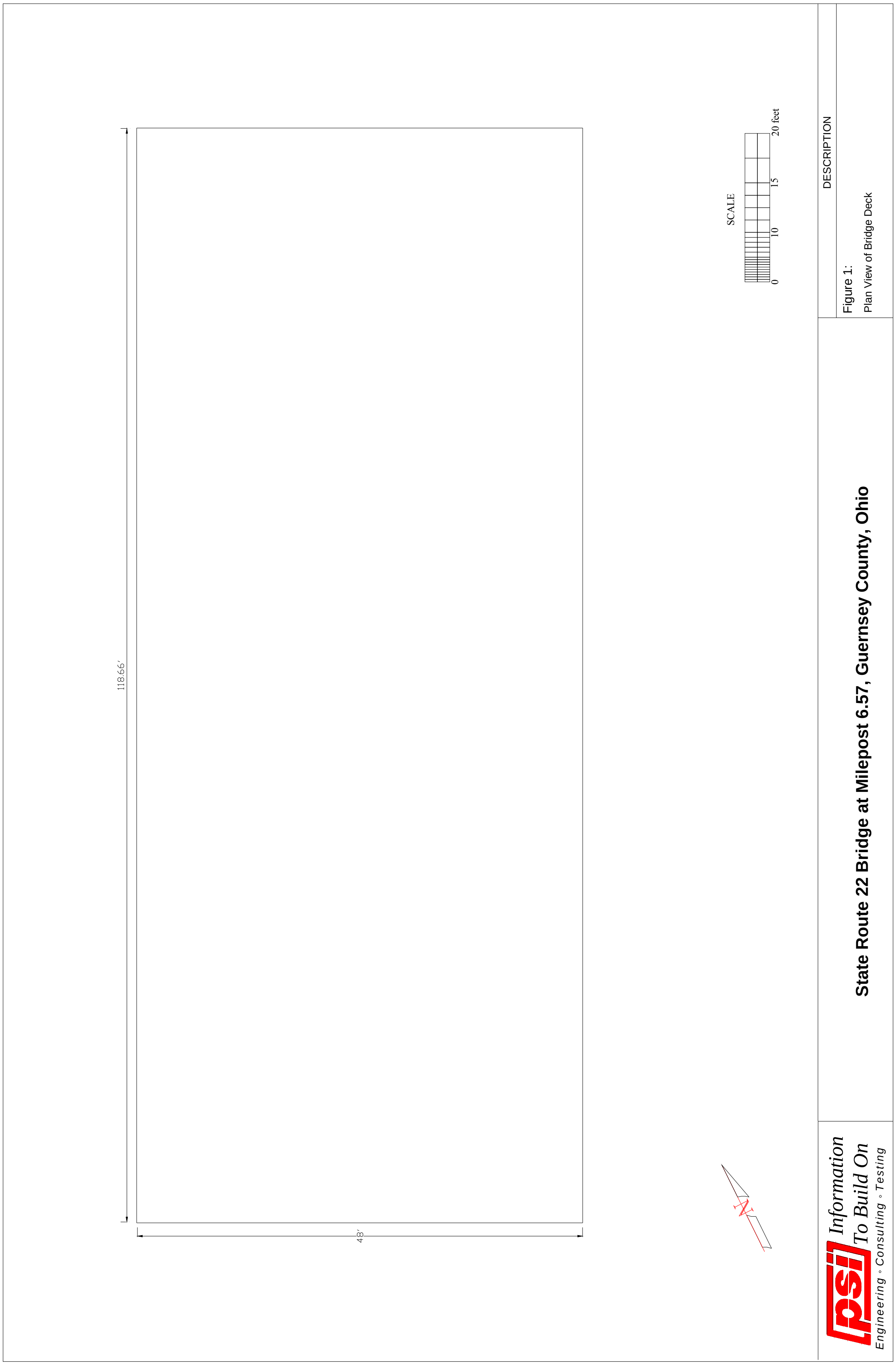

118.66'
43'
N
SCALE
0
10
15
20 feet
DESCRIPTION
Figure 1:
Plan View of Bridge Deck
State Route 22 Bridge at Milepost 6.57, Guernsey County, Ohio
psi
Information
To Build On
Engineering • Consulting • Testing

Figure 2:
Plan View of Core and Crack Locations

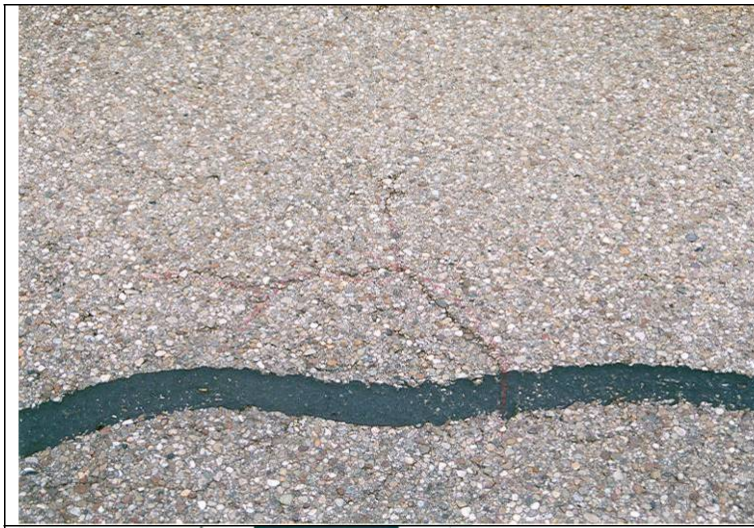
Photographic Documentation

	Photo 1: View looking of the east facing bridge from the bank.
	Photo 2: View looking north from the southern end of bridge decks.
	Photo 3: General overall view of the bridge facing east from the southern end bridge decks.

Photo 4: General overall view of the sealed cracks on asphalt wearing surface.

Photo 5: Transverse expansion joint at the end of the bridge.

Photo 6: Close up view of the cracks at the joints.

	Photo 7: Sealed longitudinal cracks on asphalt overlay.
	Photo 8: View of the underside of bridge.
	Photo 9: Close up view of coring process through asphalt concrete pavement to the top of girder flange.

APPENDIX J – Supplement 1

State Route 22 Bridge near Cambridge, Ohio
Petrographic Report

PETROGRAPHIC EXAMINATION OF CONCRETE CORES FROM AN OHIO BRIDGE (OH)

A report submitted to the Structures Laboratory at TFHRC

Rongtang Liu
Petrographic Laboratory
FHWA TFHRC
(Reviewed by Richard Meininger, PE; Concrete Laboratory)

July 27, 2006

1. Introduction

Seven concrete cores were received from the Structures Laboratory of the Turner-Fairbank Highway Research Center for petrographic examination. The cores were collected from a concrete bridge in Ohio (Figure J1-1). The identification on the cores was as follows: OH-1, OH-2, OH-3, OH-4, OH-5, OH-6 and OH-7.

The top surface of three cores appears to have asphalt and other greasy stuff. All of the cores showed evidence of being broken off, and not being drilled all the way through. The dimensions of the cores are as follows:

Core ID	Diameter (in.)	Length (in.)
OH-1	2.75	3
OH-2	2.75	3
OH-3	2.75	2 1/2
OH-4	2.75	7/8
OH-5	3.75	2
OH-6	3.75	2
OH-7	3.75	1 1/2

Petrographic examination was performed on samples using optical microscopes. Thin sections, as well as polished slabs, were made from the concrete cores for microscopic examination.

Visual inspection of the concrete cores revealed that core OH-6 has honeycombing, as shown in Figure J1-2. No gross visual defects were observed in the other cores. The findings from microscopic examination indicate that the concrete has normal levels of entrained air voids; the hydration of the cement was reasonable; the presence of some unhydrated cement particles was also observed in the cement paste; a significant amount of cracking is present in the concrete, and it is believed that the cracking is due to shrinkage.

2. Laboratory Procedures

Petrographic examination of the concrete samples was performed in accordance with ASTM C 856, “Standard Practice for Petrographic Examination of Hardened Concrete.” Sections were polished and examined using a stereomicroscope at magnifications up to 200×. Small rectangular blocks were cut from concrete samples. One surface of each block was polished, dried, placed on petrographic slide with low−viscosity epoxy resin, and reduced to a thickness of approximately 25 micrometers (0.001 in.). These thin sections were examined using a polarized−light microscope at magnification up to 400×, to determine aggregate mineralogy, paste characteristics, and microstructure.

Two ¾-inch thick slabs were sawn from the concrete cores. The slabs were ground and polished to obtain a smooth and plane surface. They were examined using a stereomicroscope at magnifications up to 200×.

3. Findings

Six thin section samples taken from the received cores were examined using the polarized light microscope. Two polished sections were examined using the stereomicroscope. The findings are summarized as follows:

Aggregates

The coarse aggregate in the concrete is gravel containing limestone and dolomite. Coarse aggregate particles are mostly rounded, and the maximum size is about 1/2 inch. Preferential orientation of coarse aggregate particles is not observed in this concrete.

The fine aggregate fraction is composed of quartz, limestone, dolomite, and feldspar. The fine aggregate is from natural sand and the particles appear rounded to angular.

Cement Paste

The cement is reasonably hydrated and the paste contains some unhydrated cement particles as seen under the microscope (Figure J1-3).

Air Voids

Small, spherical air voids are observed in the concrete (Figure J1-4), hence the concrete was air entrained. Entrained air voids are well distributed in the concrete. Small amount of entrapped air voids are also present in the concrete.

Cement-Aggregate Bonding

In general, the cement/aggregate interface is moderate to poor, as shown in Figure J1-5. And, cracking is observed at some aggregate/paste interface locations (Figure J1-7 and Figure J1-8).

Honeycombing

Honeycombing, as shown in Figure J1-2, is found in core OH-6, indicating that some sections of the concrete was poorly consolidated.

Cracking

Examination of thin sections and polished slabs revealed numerous random cracks throughout the cement paste (Figure J1-6). Some cracks partially surrounded fine and coarse aggregate particles (Figure J1-7 and Figure J1-8). Cracks going through cement paste connect to the cracks at the interface of cement paste and aggregate in some cases (Figure J1-9).

4. Summary

The concrete is air entrained, and the entrained air voids are well distributed in the concrete. Cement was reasonably hydrated and unhydrated cement particles are present in the concrete. The bond between the aggregate and the paste appears moderately strong.

Due to the lack of information (mix, curing, environmental condition, etc.), the cause of the significant cracking in the concrete is uncertain. It is speculated that shrinkage may be the cause of the cracking. High shrinkage may be associated with a high cement or cementitious content in the mix, high temperatures at placement, and/or poor curing. Investigations beyond petrographic examination may assist in providing further information about the cause of the cracking.

(See Appendix J – Supplement 3 which includes some further follow-up information about the concrete mixture.)

Figure J1-1: Seven cores as received. Notice that the small piece at the bottom (OH-4) is a 7/8-in. thick disc.

Figure J1-2: Honeycombing in core OH-6.

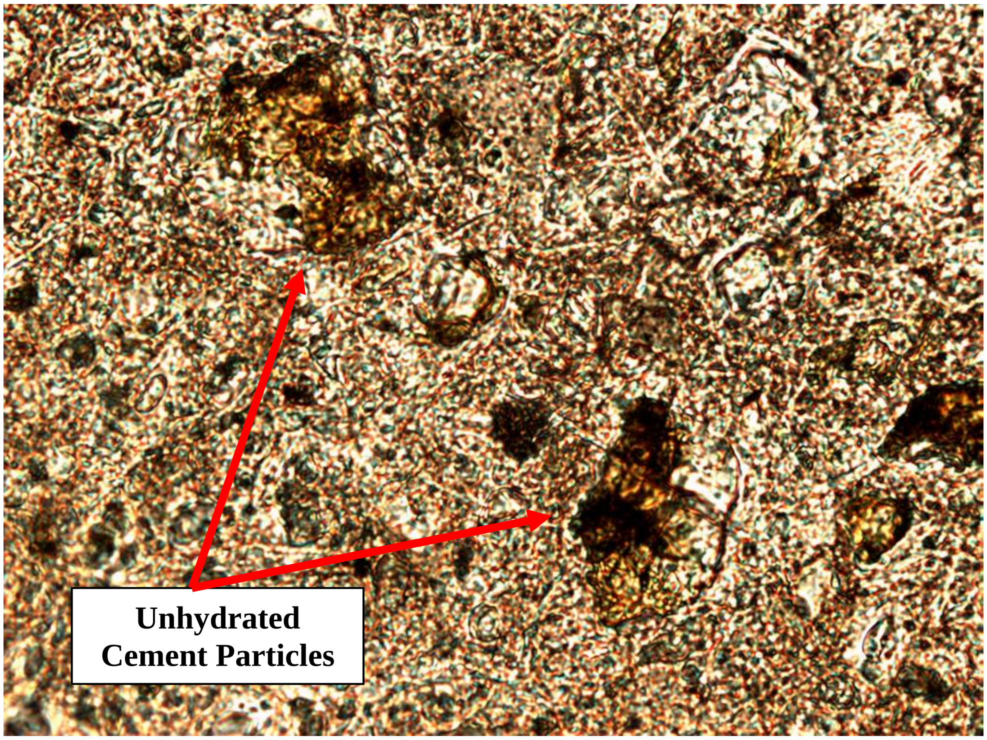


Figure J1-3: Unhydrated cement particles in paste. Width of field is 0.165 mm. Thin section image.

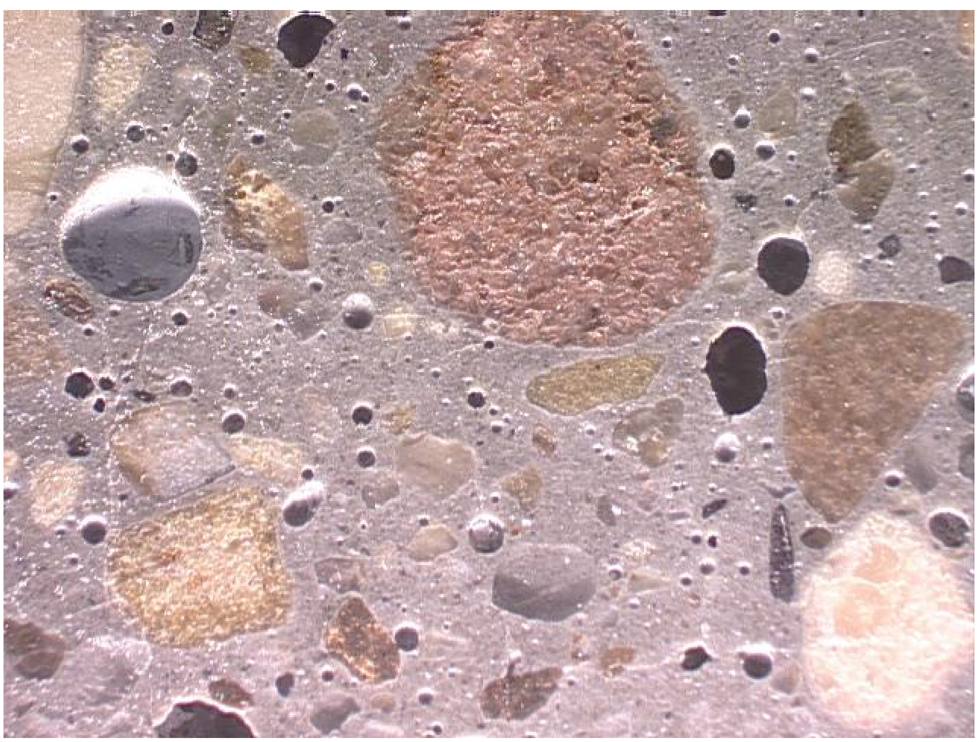


Figure J1-4: Entrained air voids in the concrete. Width of field is 4.0 mm. Polished surface image. Many cracks are also noticeable in this image.

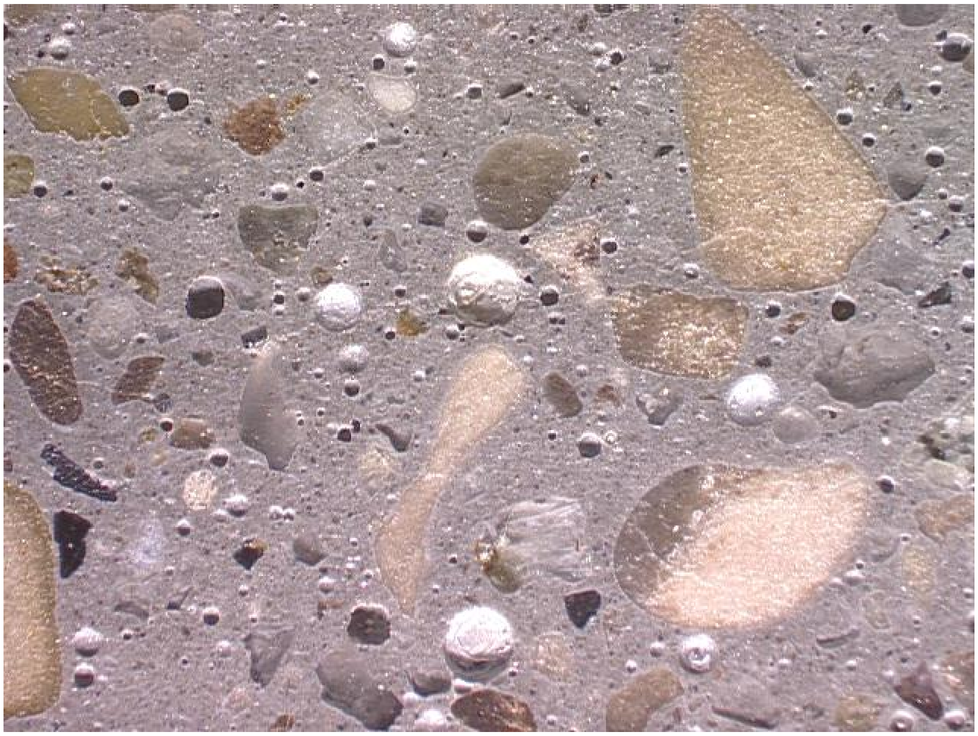


Figure J1-5: The bonding between aggregate and cement paste is moderate to poor. Width of field is 6.5 mm. Polished surface image.

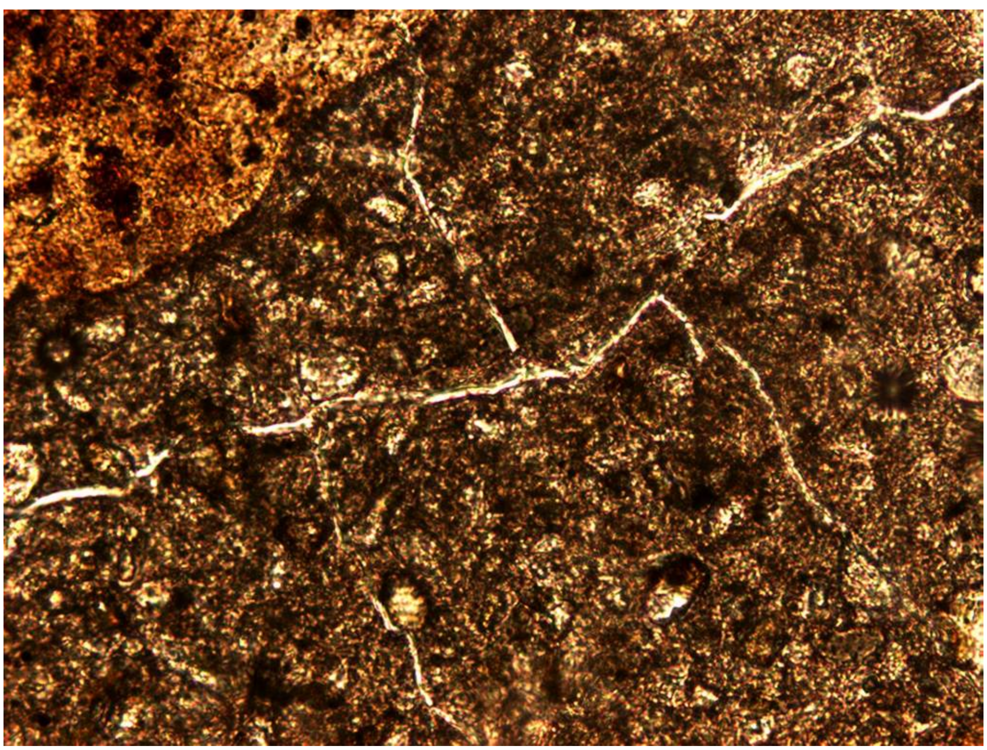


Figure J1-6: Cracks in the paste form network. Width of field is 0.33 mm. Thin section image.

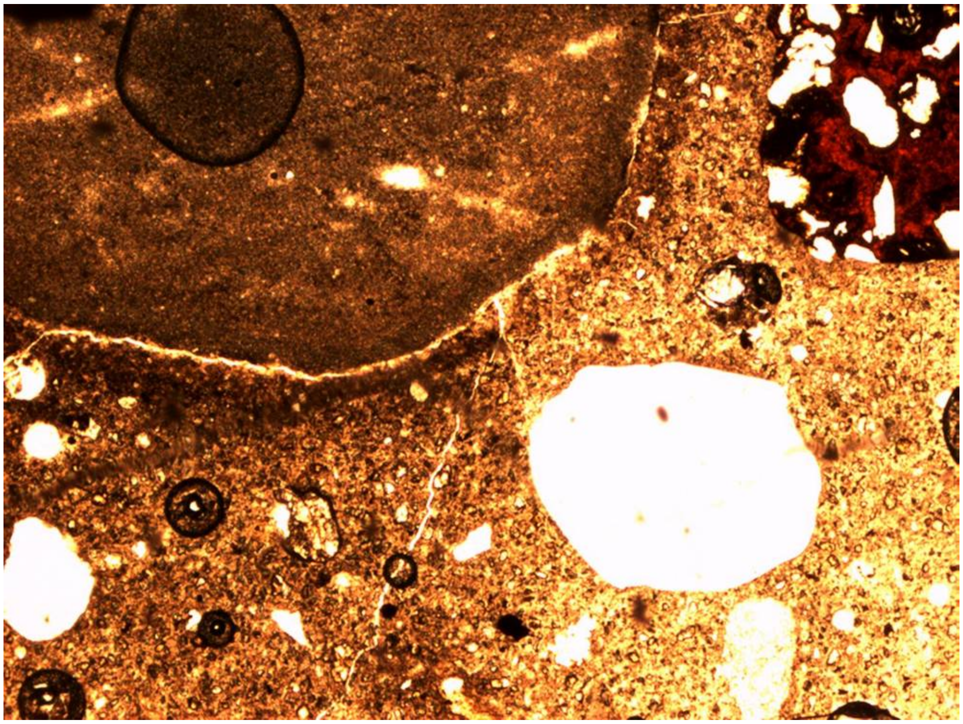


Figure J1-7: Cracks around aggregate as well as in paste. Width of field is 0.16 mm. Thin section image.

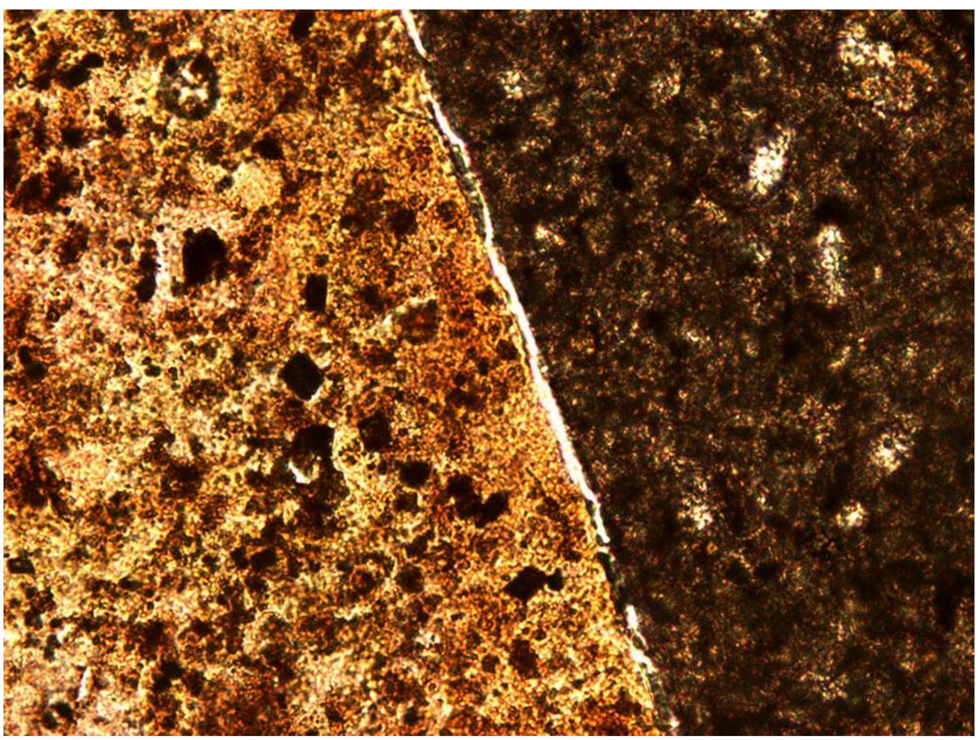


Figure J1-8: Crack along the aggregate/paste interface. Width of field is 0.33 mm. Thin section image.

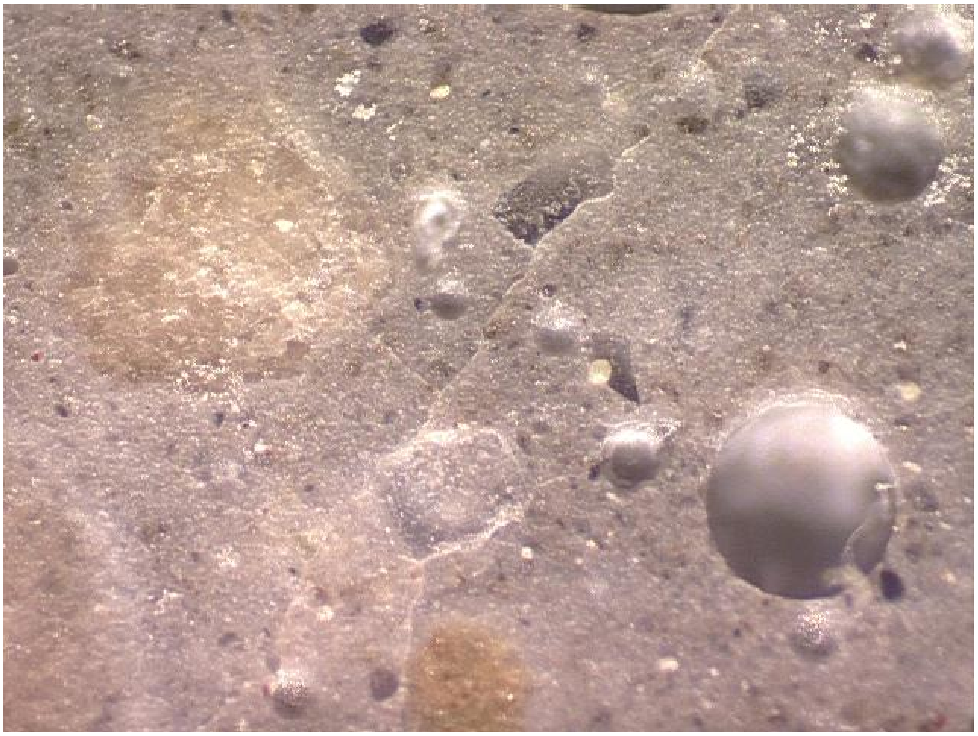


Figure J1-9: Cracks as seen in the polished slab. Width of field is 1.1 mm.

APPENDIX J – Supplement 2

State Route 22 Bridge near Cambridge, Ohio
Survey Checklist

Checklist

The following checklist is adapted from 201.1 R-92, and provided to facilitate a thorough survey. The definition of terms and associated photographs in 201.1 R-92, are utilized to standardize the reporting of the condition of the concrete in the structures.

1. Description of structure or pavement
   - 1.1 Name, location, type, and size The State Route 22 Bridge at Milepost 6.57 (Bridge GUE-22-6.57) in Guernsey County, near Cambridge, Ohio is a 115.5 ft (35.2 m) single-span structure over Crooked Creek and is composed of 12 side-by-side prestressed concrete box-beams. The bridge deck has a clear width of 48-ft (14.6 m), including two lanes and two shoulders in southbound and northbound directions.
   - 1.2 Owner, project engineer, contractor, when built Owner-Ohio State Department of Transportation. This bridge is part of a demonstration project for HPC in bridge structures which was co-sponsored by the Federal Highway Administration (FHWA) and the Ohio State Department of Transportation (ODOT). The bridge opened to traffic in November 1998.
   - 1.3 Design
     - 1.3.1 Architect and/or engineer: Ohio State Department of Transportation (ODOT).
     - 1.3.2 Intended use and history of use: Replace an old bridge to carry traffic over the SR22 over Crooked Creek.
     - 1.3.3 Special features: a single span box girder bridge. 10,000 psi (at 56 days) compressive strength concrete was used. 3 in. (76mm) thick asphalt overlay on top of concrete flange.
   - 1.4 Construction

1.4.1 Contractor-general: Ohio/West Virginia Excavating

     - 1.4.2 Subcontractors concrete placement: Prestress Service of Melbourne (KY)
     - 1.4.3 Concrete supplier: Caldwell Concrete
     - 1.4.4 Agency responsible for testing: ODOT and University of Cincinnati
     - 1.4.5 Other subcontractors: N/A
   - 1.5 Photographs
     - 1.5.1 General view Photos 1 through 2
     - 1.5.2 Detailed close up of condition of area Photos 4 through 8
   - 1.15 Sketch map-orientation showing sunny and shady and well and poorly drained regions: N/A

2. Present condition of structure Date of Evaluation The week of May 17, 2004
   - 2.1 Overall alignment of structure No signs of misalignment
     - 2.1.1 Settlement
     - 2.1.2 Deflection

2.1.3 Expansion
2.1.4 Contraction
2.2 Portions showing distress (beams, columns, pavement, walls, etc., subjected to strains and pressures) N/A
2.3 Surface condition of concrete
2.3.1 General (good, satisfactory, poor, dusting, chalking, blisters) Good
2.3.2 Cracks Transverse, Diagonal, and longitudinal
2.3.2.1 Location and frequency See Figure 2
2.3.2.18 Type and size (see Definitions) See Figure 2

Longitudinal Center lanes and over girder lines
Width (from Crack comparator) Cracks sealed and not measurable.

Hairline	(Less than 1/32 in.)
Fine	(1/32 in. - 1/16 in.)
Medium	(1/16 – 1/8 in.)
Wide	(Greater than 1/8 in.)

Transverse Throughout the length
Width (from Crack comparator) not measurable

Hairline	(Less than 1/32 in.)
Fine	(1/32 in. - 1/16 in.)
Medium	(1/16 – 1/8 in.)
Wide	(Greater than 1/8 in.)

Craze N/A
Width (from Crack comparator) ____________

Hairline	(Less than 1/32 in.)
Fine	(1/32 in. - 1/16 in.)
Medium	(1/16 – 1/8 in.)
Wide	(Greater than 1/8 in.)

Map N/A
Width (from Crack comparator) ________________

Hairline	(Less than 1/32 in.)
Fine	(1/32 in. - 1/16 in.)
Medium	(1/16 – 1/8 in.)
Wide	(Greater than 1/8 in.)

D-Cracking N/A
Width (from Crack comparator) ____________

Hairline	(Less than 1/32 in.)
Fine	(1/32 in. - 1/16 in.)
Medium	(1/16 – 1/8 in.)
Wide	(Greater than 1/8 in.)

Diagonal At Skew Ends and acute corners
Width (from Crack comparator) not measurable

Hairline	(Less than 1/32 in.)
Fine	(1/32 in. - 1/16 in.)

		Medium	(1/16 – 1/8 in.)
		Wide	(Greater than 1/8 in.)
2.3.2.19	Leaching, stalactites		N/A
2.3.3	Scaling		N/A
2.3.3.1	Area, depth		
2.3.3.10	Type (see Definitions)		
		Light	(Less than 1/8 in.)
		Medium	(1/8 in. – 3/8 in.)
		Severe	(3/8 in. – 3/4 in.)
		Very Severe	(Greater than 3/4 in.)
2.3.4	Spalls and popouts		None observed
2.3.4.1	Number, size, and depth		
2.3.4.10	Type (see Definitions)		
	Spalls		
		Small	(Less than 3/4 in. depth)
		Large	(Greater than 3/4 in. depth)
	Popouts		
		Small	(Less than 3/8 in. diameter)
		Medium	(3/8 in. – 2 in. diameter)
		Large	(Greater than 2 in. diameter)
2.3.5	Extent of corrosion or chemical attack, abrasion, impact, cavitation		N/A
2.3.6	Stains, efflorescence		at the bottom side of the bridge deck moist penetration observed
2.3.7	Exposed reinforcement		N/A
2.3.8	Curling and warping		N/A
2.3.9	Previous patching or other repair		N/A
2.3.10	Surface coatings		N/A
2.3.10.1	Type and thickness		N/A
2.3.10.2	Bond to concrete		N/A
2.3.10.3	Condition		N/A
2.3.11	Abrasion		N/A
2.3.12	Penetrating sealers		N/A
2.3.12.1	Type		N/A
2.3.12.2	Effectiveness		N/A
2.3.12.11	Discoloration		N/A
2.4	Interior condition of concrete (in situ and samples)		N/A
2.4.1	Strength of cores		
2.4.2	Density of cores		
2.4.3	Moisture content		
2.4.4	Evidence of alkali-aggregate or other reactions		N/A
2.4.5	Bond to aggregate, reinforcing steel, joints		N/A
2.4.6	Pulse velocity		
2.4.7	Volume change		
2.4.8	Air content and distribution		
2.4.9	Chloride-ion content		

2.4.10 Cover over reinforcing steel
2.4.11 Half-cell potential to reinforcing steel.
2.4.12 Evidence of reinforcement corrosion
2.4.13 Evidence of corrosion of dissimilar metals
2.4.22 Delaminations ______ N/A
2.4.22.1 Location ______ N/A
2.4.22.2 Number, and size ______ N/A
2.4.15 Depth of carbonation
2.4.16 freezing and thawing distress (frost damage)
2.4.17 Extent of deterioration
2.4.26 Aggregate proportioning, and distribution

3. Nature of loading and detrimental elements
3.1 Exposure
3.1.1 Environment (arid, subtropical, marine, freshwater, industrial, etc.)
______
3.1.2 Weather-(July and January mean temperatures, 82.3˚F/32.6˚F
mean annual rainfall and 38 in.
months in which 60 percent of it occurs) July
3.1.3 Freezing and thawing significant annual exposure to F-T
3.1.4 Wetting and drying significant annual exposure
3.1.7 Drying under dry atmosphere ______ N/A
3.1.6 Chemical attack-sulfates, acids, chloride ______ N/A
3.1.7 Abrasion, erosion, cavitation, impact ______ N/A
3.1.8 Electric currents ______ N/A
3.1.9 Deicing chemicals which contain chloride ions______ N/A
3.1.10 Heat from adjacent sources ______ N/A
3.2 Drainage ______ N/A
3.2.1 Flashing ______
3.2.2 Weepholes ______
3.2.3 Contour ______
3.2.4 Elevation of drains ______
3.3 Loading Research Test Data Available in Compilation CD Version 3
3.3.1 Dead ______
3.3.2 Live ______
3.3.3 Impact ______
3.3.4 Vibration ______
3.3.5 Traffic index ______
3.3.6 Other ______
3.4 Soils (foundation conditions) ______
3.4.1 Compressibility
3.4.2 Expansive soil
3.4.3 Settlement
3.4.4 Resistivity
3.4.5 Evidence of pumping
3.4.6 Water table (level and fluctuations)

4.	Original condition of structure	Good
	4.1 Condition of formed and finished surfaces	Good
	4.1.1 Smoothness	______
	4.1.2 Air pockets ("bugholes")	______
	4.1.3 Sand streaks	______
	4.1.4 Honeycomb	______
	4.1.5 Soft areas (retarded hydration)	______
	4.1.6 Cold joints	______
	4.1.23 Staining	______
	4.1.24 Sand pockets	______
	4.2 Defects	______
	4.2.1 Cracking	______
	4.2.1.1 Plastic shrinkage	NA
	4.2.1.2 Thermal shrinkage	______
	4.2.1.3 Drying shrinkage	______
	4.2.10 Curling	______
5.	Materials of Construction	See Tables 2
6.	Construction Practices	See Report pg. 3 and 10

APPENDIX J – Supplement 3

State Route 22 Bridge near Cambridge, Ohio
Correspondence

-----Original Message-----
From: Liu, Rongtang
Sent: Tue 8/15/2006 11:53 AM
To: Meininger, Richard
Cc:
Subject: RE: HPC Deck Scan Project - Question regarding cores from Ohio

The high cement content and the addition of silica fume strongly support my initial conclusion that shrinkage is the main cause of the cracking. SEM examination did not reveal any evidence of DEF. The silica fume was well mixed and hydrated, since I could not find any silica fume agglomerates.

Rongtang

-----Original Message-----
From: Meininger, Richard
Sent: Monday, August 14, 2006 6:56 PM
To: Liu, Rongtang
Cc: Ramadan, Jussara; Graybeal, Benjamin; Livingston, Dick
Subject: FW: HPC Deck Scan Project - Question regarding cores from Ohio

Rongtang -- Here is the mixture for the OH bridge. I am not sure you finalized this report yet.

This was precast concrete with a very high cementitious content -- almost 850 pcy of type III cement plus 100 pcy of silica fume -- The temperature must have been very high. Should we look for interstitial DEF with the SEM? The concrete has a lot of cracking and gaps around aggregate particles.

Here are my earlier comments:

"It is possible the concrete deck element(s) may have been precast. If so, the concrete may have been cured at a high temperature and then allowed to cool or dry rapidly (DEF?, Thermal Effects?, Shrinkage?) The coarse aggregate is a small maximum size so the paste content and cement content may be high. Precast concrete probably would not have fly ash or ground slag, since rapid strength gain would have been an objective. Do you see any fly ash or ground slag?

The bridge had a thick asphalt overlay which would keep the concrete wetter once water and deicing chemicals got through to the concrete. Is it possible some chemical other than chlorides were used for deicing?

Micro cracks in the paste may be related to freezing and thawing if the concrete got very saturated -- even though it is air-entrained -- it may have exceeded critical saturation.

We should report what we see (with some pictures) -- even if we cannot say why the cracking occurred."

Rick Meininger

-----Original Message-----

From: Graybeal, Benjamin
Sent: Monday, August 14, 2006 11:30 AM
To: Meininger, Richard
Subject: FW: HPC Deck Scan Project - Question regarding cores from Ohio

Rick,
Mohammad has sent the mix design from the Ohio cores (see attached). He says that the cores Rongtang is investigating are from the top flange of the adjacent box beams that comprised the superstructure/deck combination in that particular bridge. The concrete was overlaid by a thick asphalt wearing course. Hope this helps.
Ben

-----Original Message-----
From: Khan, Mohammad [mailto:mohammad.khan@psiusa.com]
Sent: Friday, August 11, 2006 10:18 AM
To: Graybeal, Benjamin
Subject: RE: HPC Deck Scan Project - Question regarding cores from Ohio

Ben - Attached please find the concrete mix data for US Route 22 Bridge near Cambridge, Ohio.

Sincerely,

Mohammad S. Khan, Ph.D., P.E.
Senior Vice President
Professional Service Industries, Inc. (PSI) 13800 Coppermine Road, Suite 200 Herndon, VA 20171
(703) 234-5301
(703) 234-5798 Fax
(703) 626-9837 Cell
mohammad.khan@psiusa.com
www.psiusa.com

From: Graybeal, Benjamin [mailto:Benjamin.Graybeal@fhwa.dot.gov]
Sent: Mon 8/7/2006 2:31 PM
To: Khan, Mohammad
Cc: Meininger, Richard
Subject: HPC Deck Scan Project - Question regarding cores from Ohio

Mohammad,

The preliminary petrographic report on the cores from the bridge in Ohio indicate that there are numerous fine cracks in the cement paste, some surrounding the fine and coarse aggregate particles. Cracks such as these have not been observed in prior cores from other HPC bridges within this study. Rongtang Liu (petrographer) and Rick Meininger (Concrete Lab manager) have asked whether there could be an explanation within a construction report or field report for the casting of this deck. Maybe high heat during initial curing? Maybe poor curing procedures? Maybe high shrinkage caused by a high cement content? Maybe something else? If you have any information that could help them out within their petrographic investigation, let me know. Thanks.

Ben

Benjamin Graybeal, Ph.D., P.E.
Research Structural Engineer
Federal Highway Administration
Turner-Fairbank Highway Research Center
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McLean, VA 22101
Tel: 202-493-3122
Fax: 202-493-3442
Email: benjamin.graybeal@fhwa.dot.gov

APPENDIX K

I-29 Northbound Bridge, South Dakota

HIGH PERFORMANCE CONCRETE BRIDGE DECK INVESTIGATION

I-29 Northbound Bridge near Sioux Falls, South Dakota

I. BACKGROUND

The South Dakota Department of Transportation's (SDDOT) first time use of high performance concrete (HPC) in an entire superstructure was the construction of I-29 Northbound Bridge in Minnehaha County, near Sioux Falls. The I-29 Northbound Bridge was built in the summer of 1999. HPC was used in the girders, deck, and bent diaphragms.

The I-29 Northbound Bridge is a railroad overpass structure. The bridge consisted of typical three-span precast, prestressed concrete girders with standard integral abutments and integral bent diaphragms. AASHTO Type II girders were used in the 54-ft (16.5-m) long end spans and the 61-ft (18.6-m) long main span. The total length of the I-29 Northbound Bridge is 172-ft (52.4-m). There are two traffic lanes and two shoulders for a clear deck width of 40 ft (12.2 m). The deck of I-29 Northbound Bridge is composed of 9-in. thick cast-in-place concrete.

The reason that the I-29 Northbound Bridge was chosen was mainly because of the high traffic counts and heavy use of deicing salts. This provided a test of the strength and durability of HPC in bridge decks. The use of HPC allowed designers to reduce the number of girders in each span from five to four. Design compressive strength of the girder concrete was 9900 psi (68.3 MPa) at 28 days and 8250 psi (56.9 MPa) at release of the strands. The deck utilized a 4500 psi (31 MPa) compressive strength concrete. To improve durability, the cementitious materials in the deck concrete consisted of fly ash (17%) and silica fume (8%). The girders had a low water-cementitious materials-ratio of 0.25. Curing was required for a minimum of seven days.

The I-29 Northbound Bridge is part of a demonstration project for HPC in bridge structures, which are co-sponsored by the Federal Highway Administration (FHWA) and the South Dakota Department of Transportation (SDDOT). South Dakota School of Mines and Technology did trial batches and testing to optimize mix designs for the girders and the deck. South Dakota State University instrumented, monitored, and tested the girder and deck concrete during and after construction.

Following the success of the I-29 Northbound Bridge, the South Dakota Department of Transportation (SDDOT) decided to construct another HPC bridge, the I-29 Southbound Bridge in the summer of 2000. The I-29 Southbound Bridge would serve as a comparison and for additional research purposes.

II. SCOPE OF SERVICES

Professional Service Industries (PSI) is under contract with the Federal Highway Administration (FHWA) to conduct bridge deck inspections for HPC bridges. Our scope of services on this bridge included a series of tasks and sub-tasks, which are described as follows:

- Collect all available information relevant to the construction of each bridge, including;
    - Deck Concrete Properties
    - Specified Deck Concrete Construction Procedures
    - Approved Concrete Mix Proportions
    - Measured Properties from QC
    - Other Measured Properties
    - Actual Method of Deck Placement
    - Average Daily Traffic (ADT)
    - Exposure Condition of the Bridge
    - Any Performed Maintenance
    - Any Inspection Reports
- Visually inspect the bridge, and obtain the following:
    - General condition of the deck top surface
    - Determination of the maximum crack width
    - General condition of the deck underside
    - General condition of the girders
    - Photograph areas of significant deterioration
    - Prepare drawings locating defects
    - Extract 7 concrete core samples

III. COMPILATION OF BRIDGE CONSTRUCTION INFORMATION

Information sources used for this report include bridge drawings, inspection reports, bridge summary data sheets, and technical information contained in FHWA's "Compilation and Evaluation of Results from High Performance Concrete Bridge Projects" version 3.

Deck Concrete Properties

The concrete mixture design for cast-in-place bridge decks of the I-29 Northbound Bridge is based on 4500 psi compressive strength at 28 days. Fly ash and silica fume were used. Maximum water-to-cementitious materials ratio of 0.39 and Type II cement was specified. For adequate protection against the freeze-thaw cycles, the air content was specified to be 6.5±1%. Table 1 lists the specified concrete properties for the cast-in-place deck.

TABLE 1: Specified Concrete Properties

Property	Deck
Cement type:	Type II
Minimum Cementitious Materials Content:	684 lb/yd^3
Max. Water/Cementitious Materials Ratio:	0.39
Min. Quantity of Fly Ash:	118 lb/yd^3
Max. Quantity of Fly Ash:	118 lb/yd^3
Min. Quantity of Silica Fume:	55 lb/yd^3
Max. Quantity of Silica Fume:	55 lb/yd^3
Air Content:	6.5±1%
Slump:	5-7 in.
Compressive Strength - Design:	4500 psi @ 28 days

Specified Deck Concrete Construction Procedures

In the construction of the I-29 Northbound Bridge, it was specified that a test pour shall be conducted a minimum of 30 days prior to the deck pour. The test pour shall consist of an unreinforced slab 40-ft wide, 36-ft long, and 9-in thick. The test pour shall be set up; placed, finished, and cured in the same manner the bridge deck shall be done. The last 10-ft of the test pour shall not be fogged but instead shall have an evaporation retardant applied immediately behind the carpet drag on the finish machine. Following the initial application of the evaporation retardant, the test pour slab shall be given a grooved finish using a metal tine, and the evaporation retardant shall be reapplied. Application of the evaporation retardant shall be in accordance with the manufacturer's instructions. Curing was required for a minimum of seven days using wet burlap, soaker hoses, and polyethylene sheeting.

Approved Concrete Mix Proportions

The approved proportions for cast-in-place deck are shown in Table 2. Note that the selected mix design was chosen based on performance during trial batching.

TABLE 2: Approved Mix Proportions for the I-29 Northbound Bridge

Mix Parameters	Cast-in-Place Deck
Cement Type:	Type II
Cement Quantity:	511 lb/yd^3
Fly Ash Quantity:	118 lb/yd^3
Silica Fume Quantity:	55 lb/yd^3
Fine Aggregate Quantity:	1100 lb/yd^3
Coarse Aggregate Type:	Quartzite
Coarse Aggregate Quantity:	1725 lb/yd^3
Water:	264 lb/yd^3
Water Reducer Brand:	Polyheed 997 (1)
Water Reducer Type:	A and F
Water Reducer Quantity:	40.9 fl oz/yd^3
Air Entrainment Brand:	MB-VR
Air Entrainment Type:	Neutralized vinsol resin
Air Entrainment Quantity:	—
Water/Cementitious Materials Ratio:	0.39

NOTE: (1) Contractor was allowed to change the water reducer.

Measured Properties from QC Tests of Production Concrete

Cast-in-Place Deck

Measured properties of the deck concrete mix from QC tests are summarized in Table 3. Air content, slump, and compressive strengths meet the specifications (Table 2).

TABLE 3: Measured Properties of QC Tests of the Production Concrete Mixes for the Cast-in-Place Deck

Property	Value
Slump:	5-7 in.
Air Content:	5.5-8%
Average Unit Weight:	139.7 lb/ft^3
Compressive Strength:	5135 psi at 14 days 6140 psi at 28 days

Measured Properties from Research Tests of Production Concrete for the Deck

Research tests of the production concrete showed that the compressive strength of the I-29 Northbound Bridge had 28-day strengths greater than 7000 psi, well above the specified 4500 psi. The results from compressive strength test are shown in Table 4. The measured modulus of elasticity from research tests of production concrete for the cast-in-place deck is listed in Table 5. Modulus of rupture was measured and the results are shown in Table 6. The results from the chloride permeability test (AASHTO T 277) are present in Table 7.

TABLE 4: Measured Compressive Strength from Research Tests of Production Concrete for the Cast-in-place Deck

Sample	Concrete Age, days						
	3	7	14	28	90	191	365
1	4030	5300	6730	7530	8260	8140	8510
2	3740	5020	6100	6830	7560	7820	8080
3	3720	5040	5960	6960	7520	7900	8290
4	3640	4960	5970	6750	7430	7890	8280
5	3990	5290	6490	7290	8000	8410	8570
Average	3830	5120	6250	7070	7750	8030	8350

TABLE 5: Measured Modulus of Elasticity from Research Tests of Production Concrete for the Cast-in-place Deck

Concrete Age, days	3	7	14	28	91	191	365
Compressive Strength, psi	3750	5170	6090	7120	7840	7820	8150
Modulus of Elasticity, ksi	3840	4510	4900	5220	5140	5340	5370

NOTE: All results are the average values from three tests on 6x12-in. cylinders

TABLE 6: Measured Modulus of Rupture from Research Tests of Production Concrete for the Cast-in-place Deck

MOR test	Range	Average	Range MOR/$\sqrt{f'_c}$	Average MOR/$\sqrt{f'_c}$	Concrete Age
Value	715-828 psi	775 psi	8.50-9.85	9.22	28

TABLE 7: Measured Chloride Permeability from Research Tests of Production Concrete for the Cast-in-place Deck (AASHTO T 277)

Sample No.	Chloride Permeability, coulombs			
31	281	323	—	—
33	323	342	—	—
34	359	414	535	534
35	625	654	581	622
36	300	484	432	355
Extras	439	582	482	560

Actual Method of Deck Placement

Construction of the northbound I-29 bridge began in summer of 1999. Placing HPC in the bridge deck was not allowed until successful completion of a test pour. Successful completion was defined as achieving concrete compressive strengths in excess of the minimum specified and demonstrating a successful fogging and curing operation with minimal cracking of the test pour.

As soon as the bridge deck concrete was finished by the finish machine, it was given a carpet drag finish with the carpet drag attached to the finish machine. Fog curing with approved fogging equipment began immediately behind the finish machine until wet burlap was applied. Fogging was considered inadequate when the relative humidity was less than 85% within 6-in above the deck surface. In this case fogging was immediately applied and the area of coverage was increased. The engineer monitored relative humidity in the field. Wet burlap was placed as soon as the concrete surface could support it without deformation. The burlap was kept continuously and thoroughly wet with soaker hoses for not less than seven days after placing the concrete. Polyethylene sheeting was placed over the wet burlap and soaker hoses as soon as the concrete could be walked on without damaging it. Bridge deck grooving was sawed into the surface at least 14 days after the deck pour. Grooving was cut transverse to the centerline of the roadway, succeeding passes should not overlap, and grooving was terminated one foot from the barrier curb. Curing compound was not allowed unless fogging was ineffective due to high winds or equipment malfunction. Because wind can cause fogging to be ineffective, the contractor attempted to pour the bridge deck when light winds were forecast. The contractor was not allowed to pour when winds were in excess of 20 mph at the start of the pour.

Average Daily Traffic (ADT)

Average daily traffic for both northbound and southbound lanes was calculated based on a count of all vehicles crossing the bridge during a 10 minutes period beginning at 1045 hrs on June 15, 2004. The northbound ADT was 11,448, including 9,072 cars and 2,376 trucks per day. The estimation of traffic flow was made while the PSI inspection crew was on-site.

Exposure Condition of the Bridge

The I-29 Northbound Bridge in Minnehaha County, near Sioux Falls, South Dakota is a railroad overpass structure. It experiences a high volume of traffic and wide range of climate conditions throughout the year. The mean daily maximum temperatures for Sioux Falls range from 66^{o}F in January to a high of 110^{o}F in July. Mean daily minimum temperatures in Sioux Falls vary between a low -36 ^{o}F in January and 38^{o}F in July. The Sioux Falls area experiences about 25.1 in. precipitation per year, implying that the bridge experiences many wet/dry cycles. The temperature history throughout the year indicates a considerable number of freeze-thaw cycles.

Performed Maintenance

No documents were found which would indicate any maintenance had been performed since the bridge was constructed.

Inspection Reports

As part of the project, South Dakota State University instrumented, monitored, and tested the girder and deck concrete during and after construction. No research report has been available.

IV. BRIDGE DECK INSPECTION

PSI personnel performed a visual inspection of the bridge decks during the week of June 14, 2004. The results of the inspection are summarized as follows.

General Condition of the Deck Top Surface

Figure 1 shows the general layout of the decks for the I-29 northbound bridge. Results of visual inspection of the decks are shown in Figure 2. Surface defects observed and documented during visual inspection primarily included transverse cracks, diagonal cracks, and longitudinal cracks (see photos 3 through 5). There are numerous small chips observed near the saw-cut grooves in the deck (photo 6). Apparent signs of other serious damages such as freeze-thaw, D-cracking, alkali-silica reaction, and alkali-aggregate reaction were not observed. Longitudinal cracks were observed at span ends of cast-in-place deck.

A total of 143 cracks (101 traverse cracks, 12 longitudinal cracks, and 30 diagonal cracks) were recorded during visual survey of the bridge decks (see Figure 2). The sum of crack lengths was 889.5 ft over a bridge deck area of 6,760 ft^2. Crack density (total crack length / deck area) for the bridge was calculated to be 0.132 ft/ft^2.

It is noted that the number of transverse crack accounts for majority of cracks recorded (71%), and the total length is 712.5 ft. The 30 diagonal cracks have a total length of 128.5 ft. The total length for longitudinal cracks is 48.5 ft. Span 1 and Span 3 have similar crack counts (i.e., 53 cracks measured on Span 1, and 51 cracks measured on Span 3). Span 2 only has 39 transverse cracks.

All cracks measured are hairline crack with a width of less than 0.031 in. Typical crack patterns on the bridge decks are shown in photos 3 through 5.

Diagonal cracks were typically limited at span ends. Transverse cracks were typically found in the traffic lanes and shoulders. Small surface spalls, which either occurred due to breaking of tined edges or the crack edges, were observed. Figure 2 also illustrates the locations of drilled cores.

The number, length and density of cracks for entire bridge decks in both directions are shown in Tables 8 through 10, and described below according to the crack type.

Transverse Cracks: Figure 2 illustrates the transverse cracks that were identified on the surface of the bridge decks. Table 8 provides the detailed information regarding transverse cracks identified on the bridge decks. The crack densities (crack length per deck area) range from 0.0944 to 0.1156 ft/ft^2 for the 3 spans investigated.

TABLE 8: Measured Transverse Cracks on the Bridge Decks

Traverse Cracks	Count	Length Range (feet)	Mean Length of Cracks (feet)	Median Length of Cracks (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span 1	27	1 to 25	7.6	5	204	2160	0.0944
Span 2	35	1.5 to 24	7.2	6	282	2440	0.1156
Span 3	39	1 to 16	6.5	7	226.5	2160	0.1049

Diagonal Cracks: The diagonal crack densities (crack length per deck area) range from 0.0285 to 0.0310 ft/ft^2 for the 3 spans investigated. Diagonal cracks in the bridge decks typically present near the joints.

TABLE 9: Measured Diagonal Cracks on the Bridge Decks

Traverse Cracks	Count	Length Range (feet)	Mean Length of Cracks (feet)	Median Length of Cracks (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span 1	16	1 to 13	3.8	3	61.5	2160	0.0285
Span 2	0	N/A	N/A	N/A	N/A	2440	N/A
Span 3	14	1 to 11	4.8	3.75	67	2160	0.0310

Longitudinal Cracks: The length of longitudinal cracks is insignificant. Several of the longitudinal cracks were along the beams and at the boundaries of the precast deck panels. The longitudinal crack densities (crack length per deck area) range from 0.0021 to 0.0204 ft/ft^2 for the 3 spans investigated.

TABLE 10: Measured Longitudinal Cracks on the Bridge Decks

Traverse Cracks	Count	Length Range (feet)	Mean Length of Cracks (feet)	Median Length of Cracks (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span 1	10	1 to 7	4.4	3.5	44	2160	0.0204
Span 2	0	NA	NA	NA	NA	2440	NA
Span 3	2	2 to 2.5	2.3	2.3	4.5	2160	0.0021

Maximum Crack Width

The maximum width of transverse cracks was measured to be 0.01 in. According to ACI 201, these crack widths are classified as hairline cracks. The fine width cracks were generally located at span ends and some exhibited spalling due to the breaking of the edges.

General Condition of the Deck Underside

The underside of the deck exhibits no signs of distress. Photos 7 and 8 show the general and close up view of the underside of the deck.

General Condition of the Girders

The girders were inspected without the aide of any access equipment. No signs of distress were noted on any of the girders.

Concrete Core Samples

Seven cores, 3-¾ inches in diameter, were retrieved from the decks. The core sample locations are shown on Figure 2. The locations were evenly distributed along each shoulder of the bridge. The cores were labeled SDNB-1 through SDNB-6, with the exception of SDNB-5a and SDNB-5b, and were transferred to FHWA for further analysis.

TABLE 11: Core Dimensions

Sample	SDNB-1	SDNB-2	SDNB-3	SDNB-4	SDNB-5a	SDNB-5b	SDNB-6
Diameter (in.)	3¾	3¾	3¾	3¾	3¾	3¾	3¾
Length (in.)	2¾	3	2¾	2¼	1½	2½	3½

Preliminary Conclusions

The construction of the I-29 Northbound Bridge was the first time of using high performance concrete (HPC) in an entire superstructure by South Dakota Department of Transportation (SDDOT). It was built in the summer of 1999.

The visual inspection of the bridge decks was performed about four and half years after the bridge opened to traffic. A total of 143 cracks (101 traverse cracks, 12 longitudinal cracks, and 30 diagonal cracks) were recorded during visual survey of the bridge decks (see Figure 2). The sum of crack lengths was 889.5 ft over a bridge deck area of 6,760 ft^2. Crack density (total crack length / deck area) for the bridge was calculated to be 0.132 ft/ft^2. The crack density as compared to other HPC bridge decks is relatively high. Majority of the cracks observed is transverse cracks, which were typically found in the traffic lanes and shoulders.

The longitudinal cracks were very limited and tend to connect to the diagonal cracks near the span joints. The relatively flexible bridge structural system combined with the heavy ADT on the bridge might have contributed to the development of some cracks.

In general, the top surface of I-29 northbound bridge was in good condition, with only hairline cracks found, showing that HPC designs provide significantly higher strength that can lead to more efficient designs and improved durability.

Petrographic examination of the concrete samples was performed on seven concrete cores that were retrieved from the I-29 Northbound Bridge. The dimension of the concrete cores was 3.75 in. in diameter and 1- to 3-in. long. The identification on the cores was as following: SDNB-1, SDNB-2, SDNB-3, SDNB-4, SDNB-5A, SDNB-5B, and SDNB-6. All of the cores showed evidence of being broken off, and not being drilled all the way through. One core (SDNB-3) was split longitudinally into halves. The rest of the cores appeared intact.

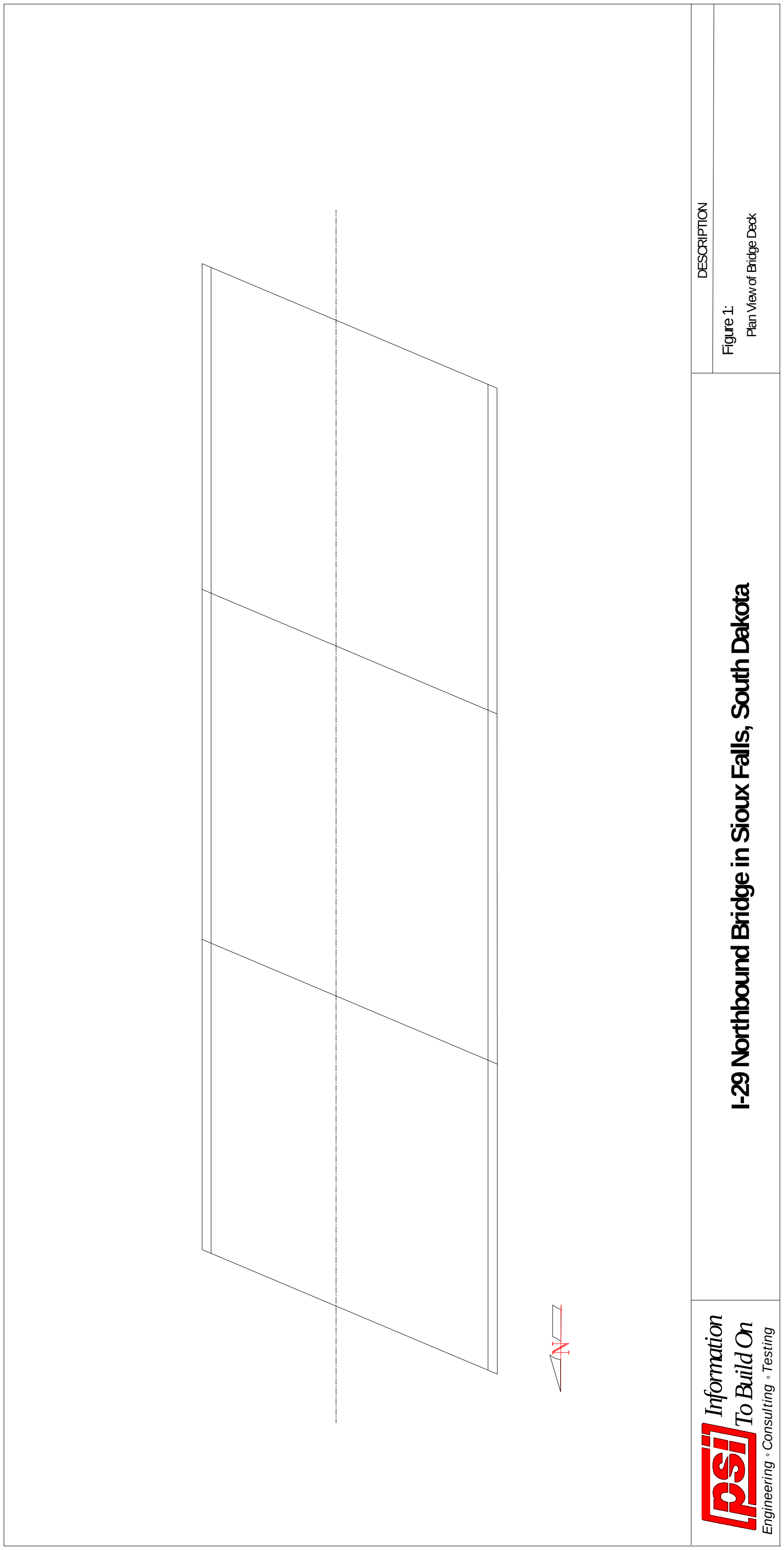
The coarse aggregate in the concrete was crushed quartzite, and the aggregate particles were mostly angular. The maximum size, measured from the prepared concrete samples, was about 1 inch. Preferential orientation of coarse aggregate particles was not observed in this concrete, nor was segregation. The natural sand fine aggregate fraction was mainly composed of quartz, with small amount of quartzite, limestone, dolomite, sandstone, chert, and feldspar. The fine aggregate particles appeared rounded to angular.

The cement was reasonably hydrated with respect to the age of the concrete. The paste contained some unhydrated cement particles. Fly ash particles and ground granulated blast-furnace slag particles were also present in the paste matrix.

The concrete was air entrained. Small, spherical air voids were well distributed in the concrete. Entrapped air voids were also present in the concrete. The paste/aggregate bond appeared to be good. Ettringite was found sporadically in some air voids.

Cracks of microscopical scale were found sporadically in the concrete. These cracks were very small in size and existed mainly in the paste. Examination of polished specimens revealed that cracks of greater sizes were present in the concrete. Cracks were also found at the interfacial region between coarse aggregate and paste. These cracks were mostly found in the 1-½ in. zone from the exposed surface.

Respectfully Submitted,
Professional Service Industries, Inc.
Structural Investigation & Petrography Department


DESCRIPTION
Figure 1:
Plan View of Bridge Deck
I-29 Northbound Bridge in Sioux Falls, South Dakota
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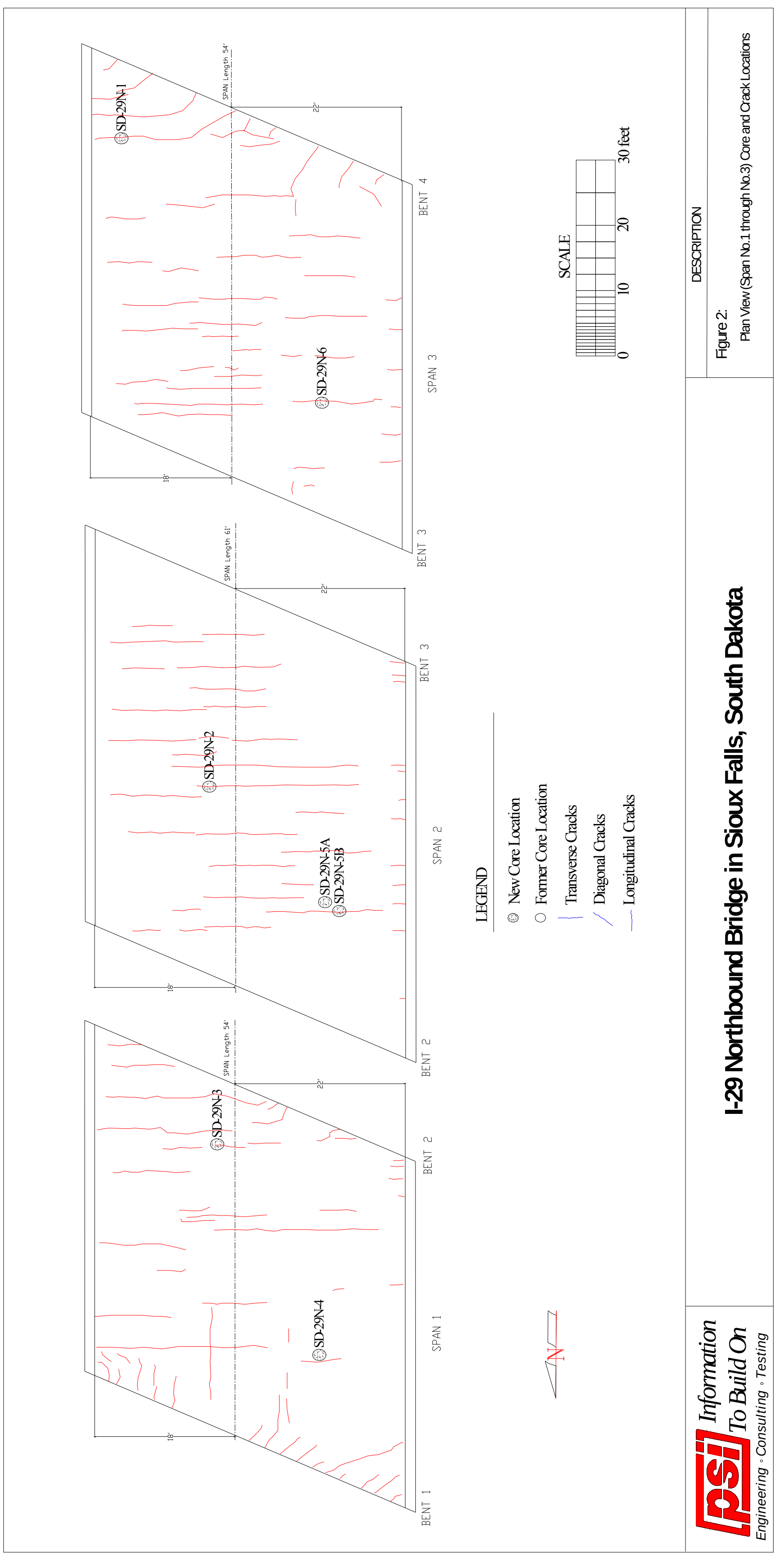
Information
To Build On
Engineering • Consulting • Testing



SD-29N-1
SD-29N-6
SD-29N-2
SD-29N-5A
SD-29N-5B
SD-29N-3
SD-29N-4
BENT 1
SPAN 1
BENT 2
SPAN 2
BENT 3
SPAN 3
BENT 4
SPAN Length 54'
SPAN Length 61'
SPAN Length 54'
22'
18'
N
LEGEND
New Core Location
Former Core Location
Transverse Cracks
Diagonal Cracks
Longitudinal Cracks
SCALE
0
10
20
30 feet
DESCRIPTION
Figure 2:
Plan View (Span No.1 through No.3) Core and Crack Locations
I-29 Northbound Bridge in Sioux Falls, South Dakota
psi
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Photographic Documentation

	Photo 1: General Overview looking of the I-29 Northbound Bridge from the side.
	Photo 2: View looking north from the southern end of bridge.
	Photo 3: Cracks on the north side approach slab on the I-29 Northbound Bridge.

Photo 4: Close up view of diagonal crack near the joint.

Photo 5: Close up view of transverse crack.

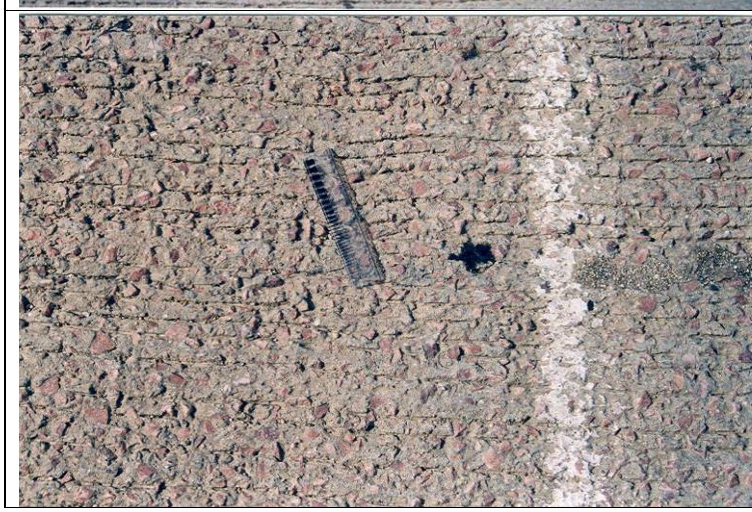


Photo 6: small chips observed near the saw-cut grooves.

	Photo 7: General overview of underside of the bridge.
	Photo 8: Close up view of underside of the bridge. Note the crack and efflorescence.

APPENDIX K – Supplement 1

I-29 Northbound Bridge, South Dakota
Petrographic Report

PETROGRAPHIC EXAMINATION OF CONCRETE CORES FROM A BRIDGE IN SOUTH DAKOTA (SD29N)

A report submitted to the Structures Laboratory at TFHRC

Rongtang Liu
Petrographic Laboratory
FHWA TFHRC
(Reviewed by Richard Meininger, PE; Concrete Laboratory; printed in color 9-6-2006)

August 23, 2006

1. Introduction

Seven concrete cores were received from the Structures Laboratory of the Turner-Fairbank Highway Research Center for petrographic examination. These cores were collected from a concrete bridge in South Dakota.

The dimension of the concrete cores was 3.75-in. diameter, 1 to 3-in. long. The identification on the cores was as following: SDNB-1, SDNB-2, SDNB-3, SDNB-4, SDNB-5A, SDNB-5B, and SDNB-6 (Figure K1-1).

Thin sections, as well as polished slabs, were made from the concrete cores for microscopic examination. Petrographic examination was performed on these samples using optical microscopes.

All of the cores showed evidence of being broken off, and not being drilled all the way through. One core (SDNB-3) was split longitudinally into halves. The findings from microscopic examination indicate that the concrete has entrained air voids, and the air content is estimated to be at a normal level; the hydration of the cement was reasonable, and the presence of unhydrated cement particles was observed in the cement paste; fly ash and ground granulated blast-furnace slag particles were also found in the concrete; cracks were observed in the concrete samples.

2. Laboratory Procedures

Petrographic examination of the concrete samples was performed in accordance with ASTM C 856, "Standard Practice for Petrographic Examination of Hardened Concrete." Sections were polished and examined using a stereomicroscope at magnifications up to 200×. Small rectangular blocks were cut from concrete samples. One surface of each block was polished, dried, placed on petrographic slide with low−viscosity epoxy resin, and reduced to a thickness of approximately 25 micrometers (0.001 in.). These thin sections were examined using a polarized−light microscope at magnification up to 400×, to determine aggregate mineralogy, paste characteristics, and microstructure.

Two ¾ inch thick slabs were sawn from the concrete cores. The slabs were ground and polished to obtain a smooth and plane surface. They were examined using a stereomicroscope at magnifications up to 200×.

3. Findings

Six thin section samples taken from the received cores were examined using the polarized light microscope. Two polished sections were examined using the stereomicroscope. The findings are summarized as follows:

Aggregate

The coarse aggregate in the concrete is crushed quartzite. Coarse aggregate particles are mostly angular, and the maximum size is about 1 inch. Preferential orientation of coarse aggregate particles is not observed in this concrete, nor is segregation.

The fine aggregate fraction is mainly composed of quartz, with small amounts of quartzite, limestone, dolomite, sandstone, chert, and feldspar. The fine aggregate is from natural sand and the particles appear rounded to angular.

Cement Paste

The cement is reasonably hydrated and the paste contains some unhydrated cement particles as seen under the microscope (Figure K1-2). Fly ash particles (Figure K1-3) and silica fume particles (Figure K1-4) are also present in the paste matrix.

Air Voids

Small, spherical air voids are observed in the concrete (Figure K1-5), hence the concrete was air entrained. Entrained air voids are well distributed in the concrete. The air content is estimated to be at a normal level. Entrapped air voids are also present in the concrete; one is shown in Figure K1-6.

Cement-Aggregate Bonding

The paste/aggregate bond appears to be good.

Cracking

Cracks of microscopical scale were found sporadically in the concrete, as shown in Figure K1-7. These cracks are normally very small in size and exist mainly in the paste.

Examination of polished specimens revealed that cracks of greater sizes are present in the concrete. Figure K1-8 shows a crack that is located about ½-in. below the exposed surface. Cracks also form at the interfacial region between coarse aggregate and paste, as shown in Figures K1-9 and K1-10. These cracks are mostly found in the 1-½ in. zone from the exposed surface.

Secondary Deposit

Ettringite, which is common to see as secondary deposit in concrete, was found sporadically in some air voids (Figure K1-11).

4. Summary

The concrete is air entrained and the air content is estimated to be at a normal level. The entrained air voids are well distributed in the concrete. Cement was reasonably hydrated and unhydrated cement particles are present in the concrete. Fly ash and slag particles are also found in the concrete. The bond between the aggregate and paste is good.

Micro-cracks are also found in the concrete samples. They exist mainly in the cement paste. Cracks of much greater sizes are visible under the microscope, but they are mainly found in the 1-½ in. zone from the exposed surface. It is speculated that shrinkage may be the cause of the cracking.

Ettringite forms sporadically in some air voids. It is common to see ettringite as a secondary deposit in concrete.

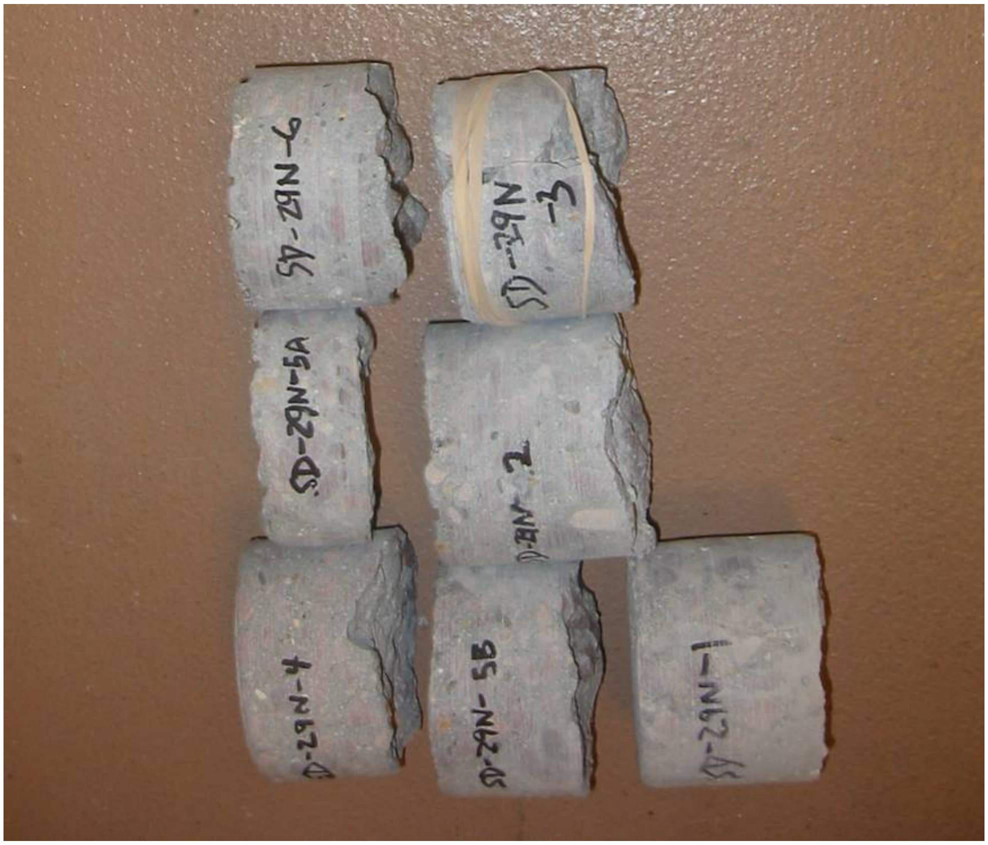


Figure K1-1: Seven concrete cores as received.

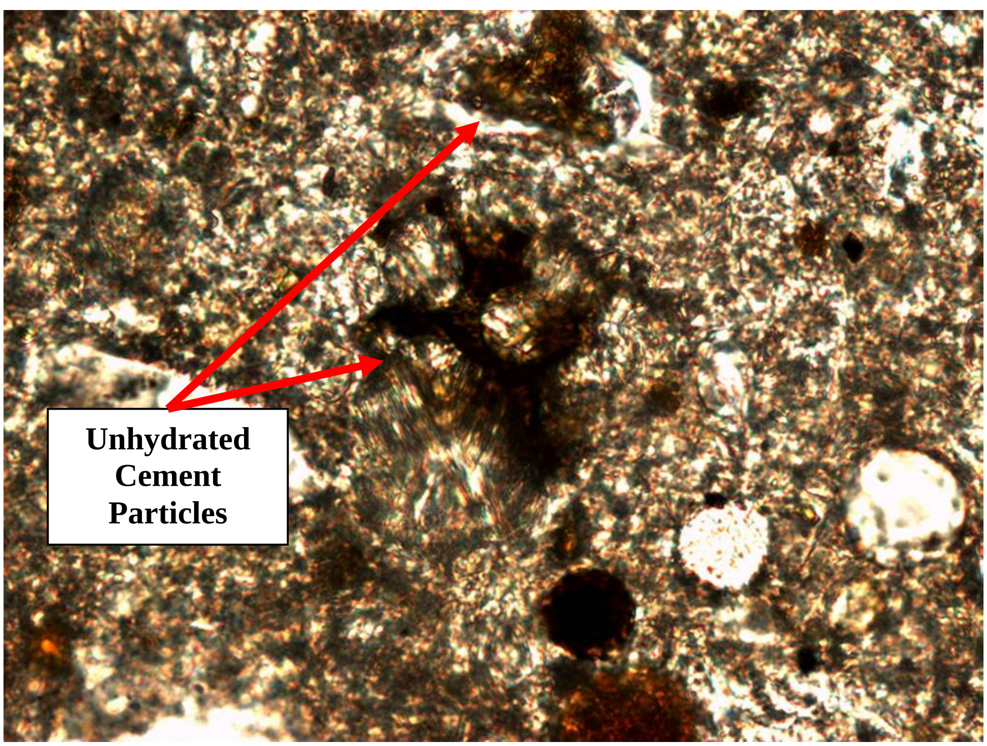


Figure K1-2: Unhydrated cement particles in the paste. Width of field is 0.165 mm. Thin section image.

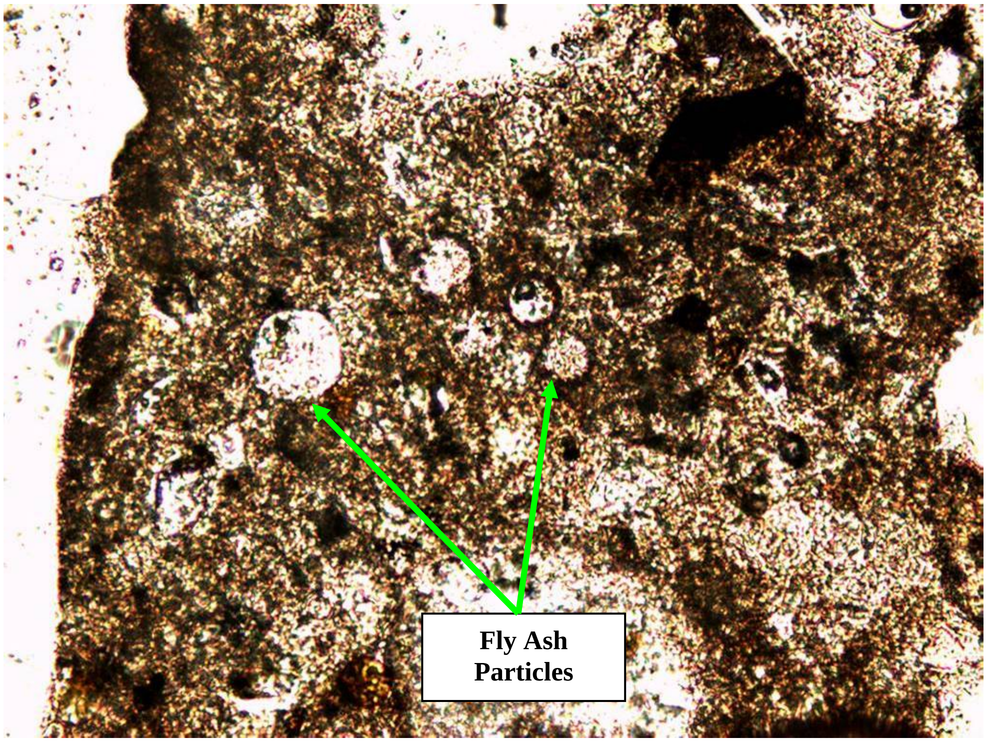


Figure K1-3: Fly ash particles in the cement matrix. Width of field is 0.33 mm. Thin section image.

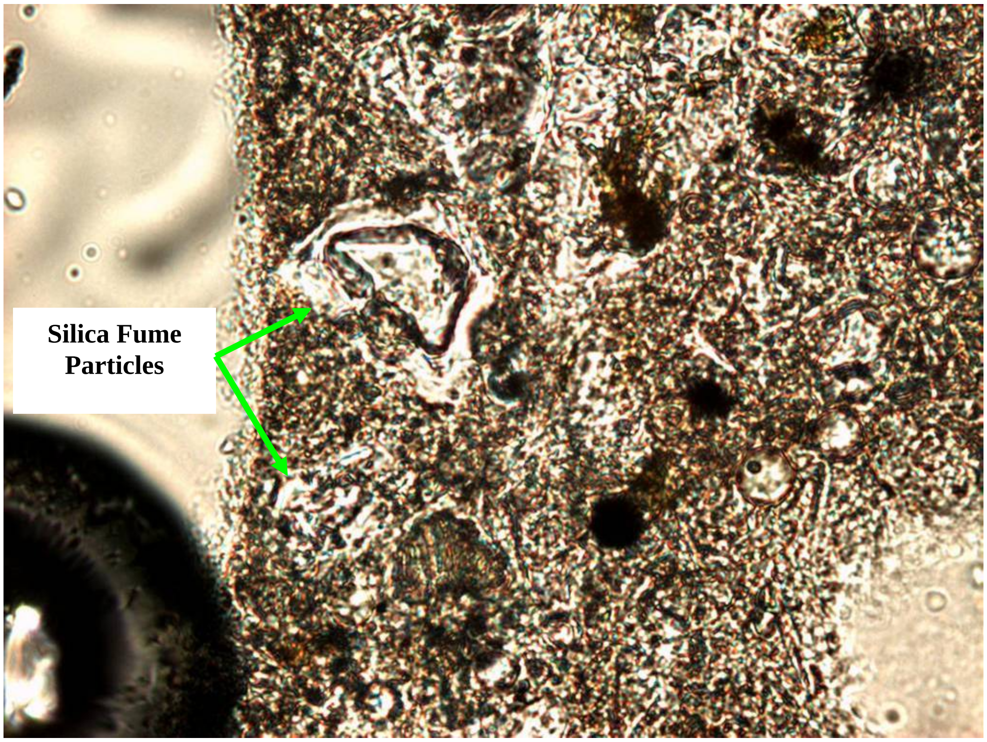


Figure K1-4: Ground granulated blast furnace slag particles in the paste matrix. Width of field is 0.165 mm. Thin section image.

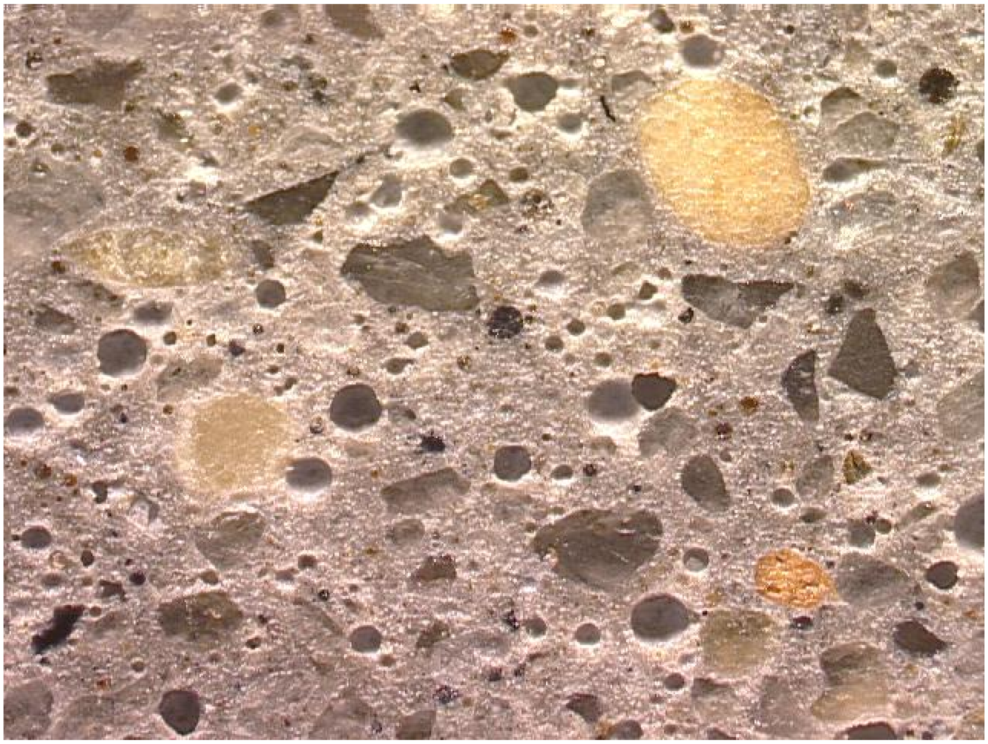


Figure K1-5: Entrained air voids in the concrete. Width of field is 4.0 mm. Polished surface image.

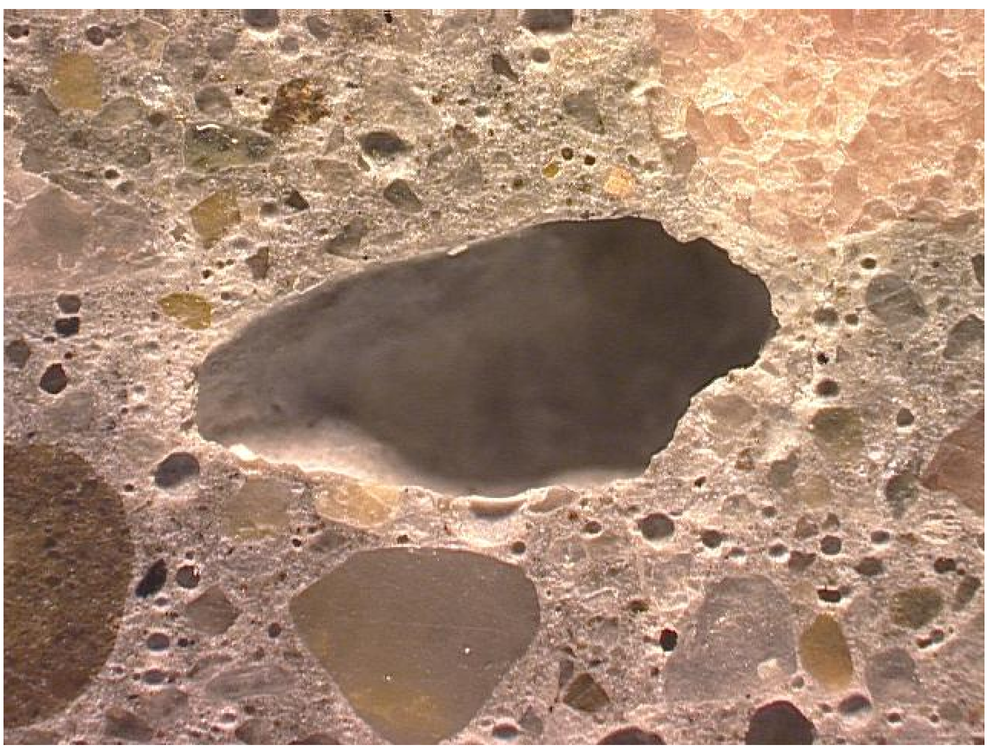


Figure K1-6: An entrapped air void. Width of field is 6.5 mm. Polished surface image.

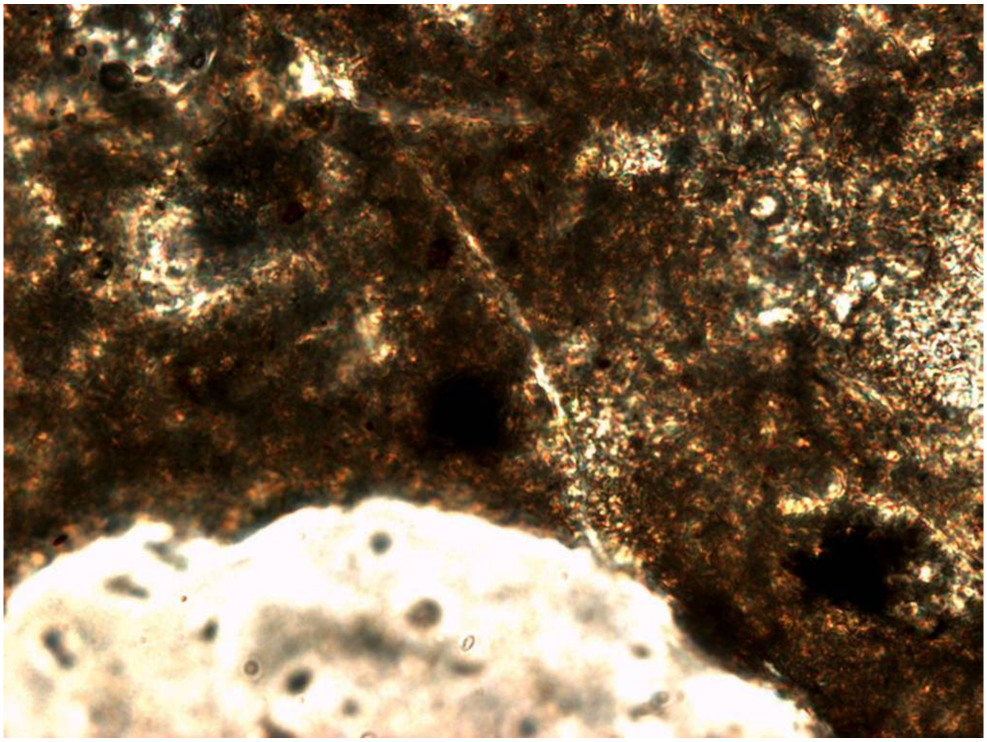


Figure K1-7: A crack in cement paste. Width of field is 0.165 mm. Thin section image.

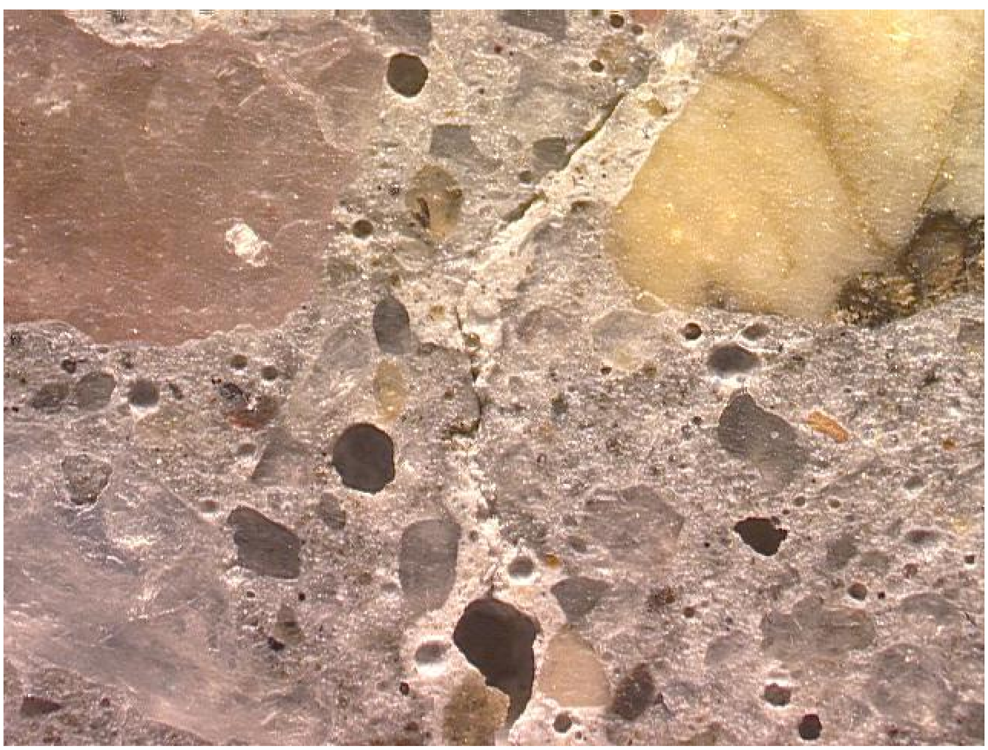


Figure K1-8: A crack located about ½ in. below the exposed surface. Width of field is 4.0 mm. Polished surface image.

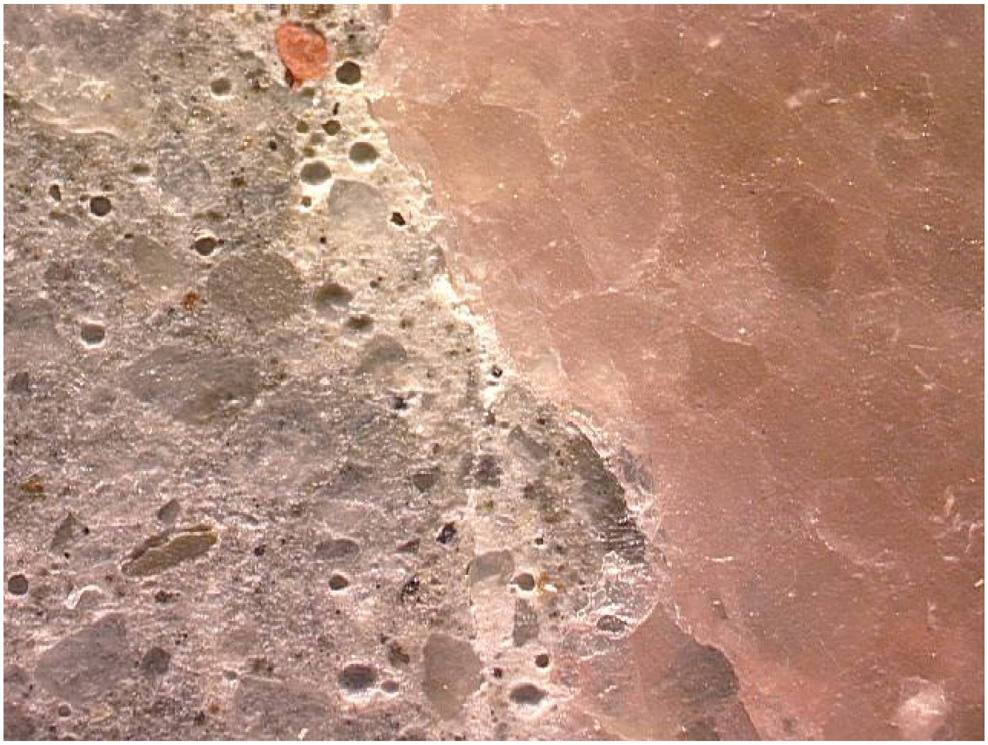


Figure K1-9: Cracks at the interfacial region between a coarse aggregate particle and paste. Width of field is 4.0 mm. Polished section image.

Figure K1-10: Another image showing cracking at the interfacial region between a coarse aggregate particle and the paste. Width of field is 2.0 mm. Polished section image.

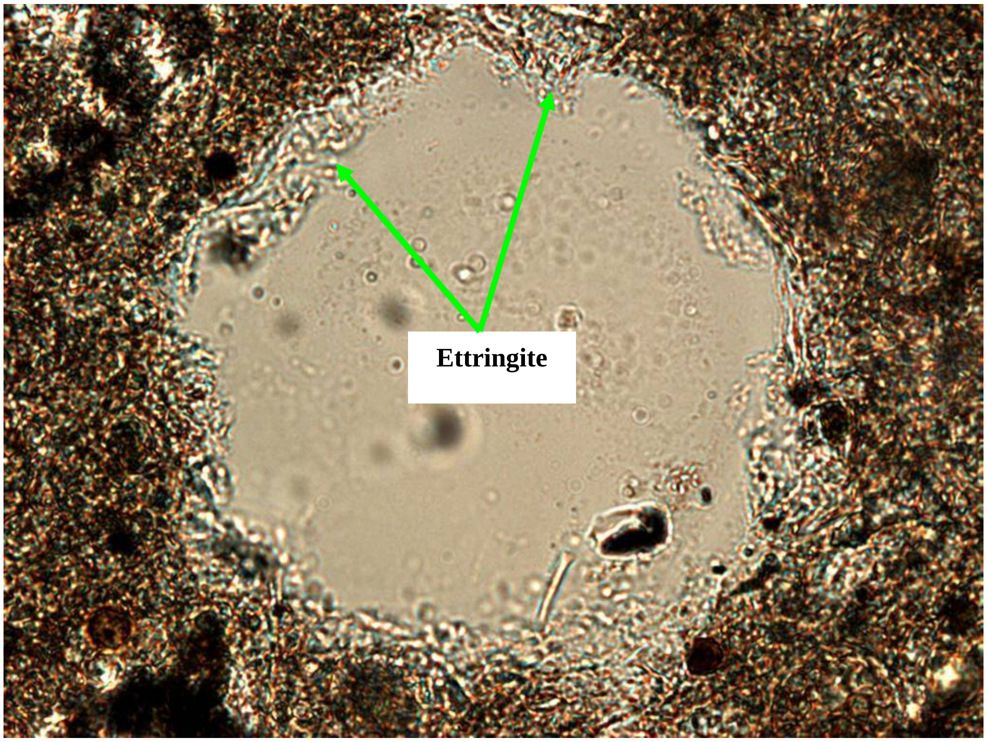


Figure K1-11: Ettringite in a void. Width of field is 0.165 mm. Thin section image.

APPENDIX K – Supplement 2

I-29 Northbound Bridge, South Dakota
Survey Checklist

Checklist

The following checklist is adapted from 201.1R-92, and provided to facilitate a thorough survey. The definition of terms and associated photographs in 201.1 R-92, are utilized to standardize the reporting of the condition of the concrete in the structures.

1. Description of structure or pavement
    - 1.2 Name, location, type, and size: I-29 Northbound Bridge in Minnehaha County, near Sioux Falls was built in the summer of 1999. It is a railroad overpass structure. The bridge consisted of typical three-span precast, prestressed concrete girders with standard integral abutments and integral bent diaphragms. Total length is 172 ft long. The clear width of the deck is 40 ft, including two through-traffic lanes and two shoulders.
    - 1.2 Owner, project engineer, contractor, when built
      Owner- South Dakota Department of Transportation. This bridge is part of a demonstration project for HPC in bridge structures which were co-sponsored by the Federal Highway Administration (FHWA) and the South Dakota Department of Transportation.
    - 1.3 Design
        - 1.3.1 Architect and/or engineer: the South Dakota Department of Transportation
        - 1.3.2 Intended use and history of use: To carry high volume of traffic on I-29 Northbound.
        - 1.3.9 Special features: HPC deck with specified strength of 4500 psi at 28days was used. The girder concrete was 9900 psi (68.3 MPa) at 28 days and 8250 psi (56.9 MPa) at release of the strands. fly ash (17%) and silica fume (8%) was used.
    - 1.4 Construction
        - 1.4.1 Contractor-general, NA
        - 1.4.2 Subcontractors concrete placement: NA
        - 1.4.3 Concrete Supplier: NA
        - 1.4.4 Agency responsible for testing: the South Dakota Department of Transportation, South Dakota School of Mines and Technology, and South Dakota State University
        - 1.4.5 Other subcontractors: NA
    - 1.5 Photographs
        - 1.5.1 General view Photos 1 through 2
        - 1.5.2 Detailed close up of condition of area Photos 3 through 8
    - 1.16 Sketch map-orientation showing sunny and shady and well and poorly drained regions N/A

2. Present condition of structure Date of Evaluation: The week of June 14, 2004
    - 2.1 Overall alignment of structure No signs of misalignment
        - 2.1.1 Settlement
        - 2.1.2 Deflection
        - 2.1.3 Expansion
        - 2.1.4 Contraction

2.2 Portions showing distress (beams, columns, pavement, walls, etc., subjected to strains and pressures) None Observed

2.3 Surface condition of concrete

2.3.1 General (good, satisfactory, poor, dusting, chalking, blisters) Good

2.3.2 Cracks Longitudinal, transverse, and diagonal

2.3.2.1 Location and frequency See Figure 2

2.3.2.20 Type and size (see Definitions) See Figure 2

Transverse Observed

Width (from Crack comparator) Hairline

Hairline	(Less than 1/32 in.)
Fine	(1/32 in. - 1/16 in.)
Medium	(1/16 – 1/8 in.)
Wide	(Greater than 1/8 in.)

Craze N/A

Width (from Crack comparator) ____________

Hairline	(Less than 1/32 in.)
Fine	(1/32 in. - 1/16 in.)
Medium	(1/16 – 1/8 in.)
Wide	(Greater than 1/8 in.)

Map N/A

Width (from Crack comparator) ________________

Hairline	(Less than 1/32 in.)
Fine	(1/32 in. - 1/16 in.)
Medium	(1/16 – 1/8 in.)
Wide	(Greater than 1/8 in.)

D-Cracking N/A

Width (from Crack comparator) ____________

Hairline	(Less than 1/32 in.)
Fine	(1/32 in. - 1/16 in.)
Medium	(1/16 – 1/8 in.)
Wide	(Greater than 1/8 in.)

Diagonal NA

Width (from Crack comparator) NA

Hairline	(Less than 1/32 in.)
Fine	(1/32 in. - 1/16 in.)
Medium	(1/16 – 1/8 in.)
Wide	(Greater than 1/8 in.)

2.3.2.21 Leaching, stalactites N/A

2.3.3 Scaling N/A

2.3.3.1 Area, depth ____________________

2.3.3.11 Type (see Definitions) ____________________

Light	(Less than 1/8 in.)
Medium	(1/8 in. – 3/8 in.)
Severe	(3/8 in. – 3/4 in.)
Very Severe	(Greater than 3/4 in.)

2.3.4 Spalls and popouts None observed
- 2.3.4.1 Number, size, and depth NA
- 2.3.4.11 Type (see Definitions) NA
  - Spalls
    - Small (Less than 3/4 in. depth)
    - Large (Greater than 3/4 in. depth)
  - Popouts
    - Small (Less than 3/8 in. diameter)
    - Medium (3/8 in. – 2 in. diameter)
    - Large (Greater than 2 in. diameter)

2.3.5 Extent of corrosion or chemical attack, abrasion, impact, cavitation N/A

2.3.6 Stains, efflorescence observed at underside of the bridge

2.3.7 Exposed reinforcement None

2.3.8 Curling and warping N/A

2.3.9 Previous patching or other repair N/A

2.3.10 Surface coatings N/A
- 2.3.10.1 Type and thickness N/A
- 2.3.10.2 Bond to concrete N/A
- 2.3.10.3 Condition N/A

2.3.11 Abrasion N/A

2.3.12 Penetrating sealers ___
- 2.3.12.1 Type N/A
- 2.3.12.2 Effectiveness N/A
- 2.3.12.12 Discoloration N/A

2.4 Interior condition of concrete (in situ and samples) N/A

2.4.1 Strength of cores

2.4.2 Density of cores

2.4.3 Moisture content

2.4.4 Evidence of alkali-aggregate or other reactions N/A

2.4.5 Bond to aggregate, reinforcing steel, joints N/A

2.4.6 Pulse velocity

2.4.7 Volume change

2.4.8 Air content and distribution

2.4.9 Chloride-ion content

2.4.10 Cover over reinforcing steel

2.4.11 Half-cell potential to reinforcing steel.

2.4.12 Evidence of reinforcement corrosion

2.4.13 Evidence of corrosion of dissimilar metals

2.4.23 Delaminations N/A
- 2.4.23.1 Location N/A
- 2.4.23.2 Number, and size N/A

2.4.15 Depth of carbonation

2.4.16 Freezing and thawing distress (frost damage)

2.4.17 Extent of deterioration

2.4.27 Aggregate proportioning, and distribution

3. Nature of loading and detrimental elements
  3.1 Exposure
    3.1.1 Environment (arid, subtropical, marine, freshwater, industrial, etc.) arid
    3.1.2 Weather-(July and January mean temperatures, Max 110ºF, min. -36ºF
     mean annual rainfall and 25.1-in
     months in which 60 percent of it occurs) June
    3.1.3 Freezing and thawing Significant
    3.1.4 Wetting and drying Significant
    3.1.8 Drying under dry atmosphere N/A
    3.1.6 Chemical attack-sulfates, acids, chloride N/A
    3.1.7 Abrasion, erosion, cavitation, impact N/A
    3.1.8 Electric currents N/A
    3.1.9 Deicing chemicals which contain chloride ions N/A
    3.1.10 Heat from adjacent sources N/A
  3.2 Drainage N/A
    3.2.1 Flashing ______
    3.2.2 Weepholes ______
    3.2.3 Contour ______
    3.2.4 Elevation of drains ______
  3.3 Loading Research Test Data Available in Compilation CD Version 3
    3.3.1 Dead ______
    3.3.2 Live ______
    3.3.3 Impact ______
    3.3.4 Vibration ______
    3.3.5 Traffic index ______
    3.3.6 Other ______
  3.4 Soils (foundation conditions) ______
    3.4.1 Compressibility
    3.4.2 Expansive soil
    3.4.3 Settlement
    3.4.4 Resistivity
    3.4.5 Evidence of pumping
    3.4.6 Water table (level and fluctuations)

4. Original condition of structure Good
  4.1 Condition of formed and finished surfaces Good
    4.1.1 Smoothness ______
    4.1.2 Air pockets ("bugholes") ______
    4.1.3 Sand streaks ______
    4.1.4 Honeycomb ______
    4.1.5 Soft areas (retarded hydration) ______
    4.1.6 Cold joints ______
    4.1.25 Staining ______
    4.1.26 Sand pockets ______
  4.2 Defects N/A
    4.2.1 Cracking ______

4.2.1.1	Plastic shrinkage	________________
4.2.1.2	Thermal shrinkage	________________
4.2.1.3	Drying shrinkage	________________
4.2.11	Curling	________________

5. Materials of Construction — See Table 1

6. Construction Practices — See Report pg. 3, 5 and 6

APPENDIX L

I-29 Southbound Bridge, South Dakota

HIGH PERFORMANCE CONCRETE BRIDGE DECK INVESTIGATION

I-29 Southbound Bridge in Sioux Falls, South Dakota

I. BACKGROUND

Following the success of the I-29 Northbound Bridge in Minnehaha County, near Sioux Falls, the South Dakota Department of Transportation (SDDOT) decided to construct another HPC bridge – the I-29 Southbound Bridge, less than a half mile away from the I-29 Northbound Bridge. I-29 Southbound Bridge was built in the summer of 2000. HPC was used in girders, deck, and bent diaphragms. The I-29 Southbound Bridge would serve for comparison purposes and additional research.

I-29 Southbound Bridge is at a 27º skew to the railroad. The bridge consisted of typical three-span precast, prestressed concrete girders with standard integral abutments and integral bent diaphragms. AASHTO Type II girders were used for the 54-ft (16.5-m) long end spans and the 61-ft (18.6-m) long main span. The total length of the I-29 Southbound Bridge is 172 ft (52.4 m). There are two traffic lanes and two shoulders for a clear deck width of 40 ft (12.2 m). The deck of I-29 Southbound Bridge is composed of 9-in. thick cast-in-place concrete.

The use of HPC allowed designers to reduce the number of girders in each span from five to four. Design compressive strength of the girder concrete was 9900 psi (68.3 MPa) at 28 days and 8250 psi (56.9 MPa) at release of the strands. The deck utilized a 4500-psi (31-MPa) compressive strength concrete. To improve durability, the cementitious materials in the deck concrete consisted of fly ash. The girders had a low water-cementitious materials-ratio of 0.25. Curing was required for a minimum of seven days.

The I-29 Southbound Bridge is also part of a demonstration project for HPC in bridge structures and was co-sponsored by the Federal Highway Administration (FHWA) and the South Dakota Department of Transportation (SDDOT). South Dakota School of Mines and Technology did trial batches and testing to optimize mix designs for the girders and the deck. South Dakota State University instrumented, monitored, and tested the girder and deck concrete during and after construction.

II. SCOPE OF SERVICES

Professional Service Industries (PSI) is under contract with the Federal Highway Administration (FHWA) to conduct bridge deck inspections for HPC bridges. Our scope of services on this bridge included a series of tasks and sub-tasks, which are described as follows:

- Collect all available information relevant to the construction of each bridge, including;
    - Deck Concrete Properties

  - Specified Deck Concrete Construction Procedures
  - Approved Concrete Mix Proportions
  - Measured Properties from QC
  - Other Measured Properties
  - Actual Method of Deck Placement
  - Average Daily Traffic (ADT)
  - Exposure Condition of the Bridge
  - Any Performed Maintenance
  - Any Inspection Reports
- Visually inspect the bridge, and obtain the following:
  - General condition of the deck top surface
  - Determination of the maximum crack width
  - General condition of the deck underside
  - General condition of the girders
  - Photograph areas of significant deterioration
  - Prepare drawings locating defects
  - Extract 6 concrete core samples

III. COMPILATION OF BRIDGE CONSTRUCTION INFORMATION

Information sources used for this report include bridge drawings, inspection reports, bridge summary data sheets, and technical information contained in FHWA's "Compilation and Evaluation of Results from High Performance Concrete Bridge Projects" version 3.

Deck Concrete Properties

The concrete mixture design for cast-in-place bridge decks of the I-29 Southbound Bridge is based on 4500 psi compressive strength at 28 days. Fly ash and Type II cement were specified. Ten percent of the cement was required to be replaced with Class F fly ash at a ratio of fly ash to cement of 1.9:1.0 by weight.

Specified Deck Concrete Construction Procedures

In the construction of the I-29 Southbound Bridge, it was specified that a test pour shall be conducted a minimum of 30 days prior to the deck pour. The test pour shall consist of an unreinforced slab 40-ft wide, 36-ft long, and 9-in. thick. The test pour shall be set up, placed, finished, and cured in the same manner the bridge deck will be done. The last 10 ft of the test pour shall not be fogged but instead shall have an evaporation retardant applied immediately behind the carpet drag on the finish machine. Following the initial application of the evaporation retardant, the test pour slab shall be given a grooved finish using a metal tine, and the evaporation retardant shall be reapplied. Application of the evaporation retardant shall be in accordance with the manufacturer's instructions. During the curing process, linseed oil based emulsion curing compound was followed by wet burlap, soaker hoses, and polyethylene sheeting for 7 days.

Approved Concrete Mix Proportions

The approved proportions for cast-in-place deck are shown in Table 1. Note that the selected mix design was chosen based on performance during trial batching.

TABLE 1: Approved Mix Proportions for the I-29 Southbound Bridge

Mix Parameters	Cast-in-Place Deck
Cement Type:	II
Cement Quantity:	590 lb/yd^3
Fly Ash Brand:	Coal Creek, Underwood
Fly Ash Quantity:	124 lb/yd^3
Fine Aggregate Quantity:	1222 lb/yd^3
Fine Aggregate SG:	2.65
Coarse Aggregate SG:	2.62
Coarse Aggregate Quantity:	1634 lb/yd^3
Water:	255 lb/yd^3
Water Reducer Brand:	WR-91 (Brett Admixtures)
Water Reducer Quantity:	22 fl oz /yd^3
Air Entrainment Brand:	AE-92 (Brett Admixtures)
Air Entrainment Quantity:	NA
Water/Cementitious Materials Ratio:	0.36
Average Air Content:	6.5%
Average Unit Weight:	141.7 lb/ft^3

Measured Properties from QC Tests of Production Concrete

Cast-in-Place Deck

Measured properties of the deck concrete mix from QC tests are summarized in Table 2. Air content, slump, and compressive strengths meet the specifications.

TABLE 2: Measured Properties of QC Tests of the Production Concrete Mixes for the Cast-in-Place Deck

Properties	Value
Average Slump	2.5 in.
Average Air Content	6.8%
Average Unit Weight	143 lb/ft^3
Compressive Strength	6170 psi

Measured Properties from Research Tests of Production Concrete for the Deck

Research tests of the production concrete showed that the compressive strength of the I-29 Southbound Bridge had 28-day strengths greater than 6500 psi, well above the specified 4500 psi. All results are the average values from three 6x12-in. cylinders.

TABLE 3: Measured Compressive Strength from Research Tests of Production Concrete for the Cast-in-place Deck

Series No.	Concrete Age, days				
	3	7	14	28	90
1	4450	5030	5840	6860	8120
2	3990	4440	5400	6330	7030
3	4420	5000	5870	6950	7930
4	4270	4970	5610	6530	7520
5	4420	5150	6070	7110	8360
Average	4310	4920	5760	6760	7770

TABLE 4: Measured Modulus of Elasticity from Research Tests of Production Concrete for the Cast-in-place Deck

Concrete Age, days	3	7	14	28	90
Compressive Strength, psi	4230	4800	5840	6640	7480
Modulus of Elasticity, ksi	4400	4550	5230	5620	5670

TABLE 5: Measured Chloride Permeability from Research Tests of Production Concrete for the Cast-in-place Deck (AASHTO T277)

Span	Permeability, coulombs
South	844, 654, 827, 804
Middle	1548, 1396, 1279, 1394
North	877, 944, 1052, 1079
Average	1058

Actual Method of Deck Placement

Construction of the Southbound I-29 bridge began in summer of 2000. Placing High Performance Concrete in the bridge deck was not allowed until successful completion of a test pour. Successful completion was defined as achieving concrete compressive strengths in excess of the minimum specified and demonstrating a successful fogging and curing operation with minimal cracking of the test pour.

As soon as the bridge deck concrete were struck off and finished by the finish machine, it was given carpet drag finish with the carpet drag attached to the finish machine. Fogging with approved fogging equipment began immediately behind the finish machine until wet burlap was applied. Fogging was considered inadequate when the relative humidity was less than 85% within 6 in. above the deck surface. In this case fogging was immediately applied and the area of coverage was increased. The engineer monitored relative humidity in the field. Wet burlap was placed as soon as the concrete surface will support it without deformation. The burlap was kept continuously and thoroughly wet with soaker hoses for not less than seven days after placing the concrete. Polyethylene sheeting was placed over the wet burlap and soaker hoses as soon as the concrete can be walked on without damaging it. Bridge deck grooving was sawed into the surface at least 14 days after the deck pour. Grooving was cut transverse to the centerline of the roadway, succeeding passes did not overlap and grooving terminated one foot from the barrier curb. Curing compound was not allowed unless fogging was ineffective due to high winds or equipment malfunction. Because wind can cause fogging to be ineffective, the contractor attempted to pour the bridge deck when light winds were forecast. The contractor was not allowed to pour when winds were in excess of 20 mph at the start of the pour.

Average Daily Traffic (ADT)

Average daily traffic for both northbound and southbound lanes was calculated based on a count of all vehicles crossing the bridge during a 10 minutes period beginning at 1055 hrs on June 15, 2004. The southbound ADT was 14,544, including 11,808 cars and 2,736 trucks per day. The estimation of traffic flow was made while the PSI inspection crew was on-site.

Exposure Condition of the Bridge

The I-29 Southbound Bridge in Minnehaha County, near Sioux Falls, South Dakota is a railroad overpass structure. It experiences high volume of traffic and a wide range of climate conditions throughout the year. The mean daily maximum temperatures for Sioux Falls range from 66°F in January to a high of 110°F in July. Mean daily minimum temperatures in Sioux Falls vary between a low -36°F in January and 38°F in July. Sioux Falls's mean daily minimum temperatures range from 5°F in January to 62°F in July. The Sioux Falls area experiences about 25.1 in. precipitation per year, implies that the bridge experiences many wet/dry cycles. The temperature history throughout the year indicates a considerable number of freeze-thaw cycles.

Performed Maintenance

No documents were found which would indicate any maintenance had been performed since the bridge was constructed.

Inspection Reports

As part of the project, South Dakota State University instrumented, monitored, and tested the girder and deck concrete during and after construction. No research report has been available.

IV. BRIDGE DECK INSPECTION

PSI personnel performed a visual inspection of the bridge decks during the week of June 14, 2004. The results of the inspection are summarized as follows.

General Condition of the Deck Top Surface

Figure 1 shows the general layout of the decks for the I-29 Southbound bridge. Results of visual inspection of the decks are shown in Figure 2. Surface defects observed and documented during visual inspection primarily included transverse cracks, diagonal cracks, and longitudinal cracks (see photos 3 through 5). There are numerous locations where pattern crack (see photo 6) and honeycomb defects (see photo 7) were observed. Apparent signs of other serious damages such as freeze-thaw, D-cracking, alkali-silica reaction, and alkali-aggregate reaction were not observed. Longitudinal cracks were observed at span ends of cast-in-place deck.

A total of 119 cracks (75 traverse cracks, 2 longitudinal cracks, and 42 diagonal cracks) were recorded during visual survey of the bridge decks (see Figure 2). The sum of crack lengths was 1,121 ft over a bridge deck area of 6,760 ft^2. Crack density (total crack length / deck area) for the eastbound and westbound bridges combined was calculated to be 0.166 ft/ft^2.

It is noted that the number of transverse crack accounts for majority of cracks recorded (63%), and the total length is 872 ft. The 42 diagonal cracks have a total length of 245 ft. The total length for longitudinal cracks is only 4 ft. Span 1 and Span 2 have similar crack counts (i.e., 34 cracks measured on Span 1, and 32 cracks measured on Span 2). Span 3 has 53 cracks. On Span 2 only transverse cracks were found.

All cracks measured are hairline crack with a width of less than 0.031 in. Typical crack patterns on the bridge decks are shown in photos 3 through 5.

Diagonal cracks were typically limited at span ends. Transverse cracks were typically found in the traffic lanes and shoulders. Small surface spalls, which either occurred due to breaking of tined edges or the crack edges, were observed. Figure 2 also illustrates the locations of drilled cores.

The number, length and density of cracks for entire bridge decks in both directions are shown in Tables 6 through 8, and described below according to the crack type.

Transverse Cracks: Figure 2 illustrates the transverse cracks that were identified on the surface of the bridge decks. Table 8 provides the detailed information regarding transverse cracks identified on the bridge decks. The crack densities (crack length per deck area) range from 0.0077 to 0.1998 ft/ft^2 for the 3 spans investigated.

TABLE 6: Measured Transverse Cracks on the Bridge Decks

Traverse Cracks	Count	Length Range (feet)	Mean Length of Cracks (feet)	Median Length of Cracks (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span 1	21	1 to 19	7.9	8	166	2160	0.0077
Span 2	32	3 to 40	15.2	13.3	487.5	2440	0.1998
Span 3	22	4 to 15	9.9	10	218.5	2160	0.1011

Diagonal Cracks: The diagonal crack densities (crack length per deck area) range from 0.0248 to 0.0887 ft/ft^2 for the 3 spans investigated. Diagonal cracks in the bridge decks typically present near the joints.

TABLE 7: Measured Diagonal Cracks on the Bridge Decks

Traverse Cracks	Count	Length Range (feet)	Mean Length of Cracks (feet)	Median Length of Cracks (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span 1	12	2 to 14	4.5	2.5	53.5	2160	0.0248
Span 2	0	NA	NA	NA	NA	2440	NA
Span 3	30	1 to 13.5	6.4	7.3	191.5	2160	0.0887

Longitudinal Cracks: The length of longitudinal cracks is insignificant. Several of the longitudinal cracks were along the beams and at the boundaries of the precast deck panels. The longitudinal crack densities (crack length per deck area) range from 0.0021 to 0.0204 ft/ft^2 for the 3 spans investigated.

TABLE 8: Measured Longitudinal Cracks on the Bridge Decks

Traverse Cracks	Count	Length Range (feet)	Mean Length of Cracks (feet)	Median Length of Cracks (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span 1	1	2	2	NA	2	2160	0.00093
Span 2	0	NA	NA	NA	NA	2440	NA
Span 3	1	2	2	NA	2	2160	0.00093

Maximum Crack Width

The maximum width of transverse cracks was measured to be 0.01 in. According to ACI 201, these crack widths are classified as hairline cracks. The fine width cracks were generally located at span ends and some exhibited spalling due to the breaking of the edges.

General Condition of the Deck Underside

The underside of the deck exhibits no signs of distress. Photo 8 shows the general view of the underside of the deck.

General Condition of the Girders

The girders were inspected without the aide of any access equipment. No signs of distress were noted on any of the girders.

Concrete Core Samples

Six cores, 3-¾ inches in diameter, were retrieved from the decks. The core sample locations are shown on Figure 2. The locations were evenly distributed along each shoulder of the bridge. The cores were labeled SDSB-1 through SDSB-6, and were transferred to FHWA for further analysis.

TABLE 9: Core Dimensions

Sample	SDSB-1	SDSB-2	SDSB-3	SDSB-4	SDSB-5	SDSB-6
Diameter (in.)	3¾	3¾	3¾	3¾	3¾	3¾
Length (in.)	3	2¾	2½	2¾	2½	3½

Preliminary Conclusions

The construction of the I-29 Southbound Bridge was the first use of high performance concrete (HPC) in an entire superstructure by South Dakota Department of Transportation (SDDOT). It was built in the summer of 2000.

The visual inspection of the bridge decks as part of our study was performed about four years after the bridge opened to traffic. A total of 119 cracks (75 traverse cracks, 2 longitudinal cracks, and 42 diagonal cracks) were recorded during visual survey of the bridge decks (see Figure 2). The sum of crack lengths was 1,121 ft over a bridge deck area of 6,760 ft^2. Crack density (total crack length / deck area) for the bridges was calculated to be 0.166 ft/ft^2. The crack density as compared to other HPC bridge decks is relatively high. Majority of the cracks observed is transverse cracks, which were typically found in the traffic lanes and shoulders.

Diagonal cracks were typically limited at span ends. Small surface spalls, which either occurred due to breaking of tined edges or the crack edges, were observed. The longitudinal cracks were very limited and tend to connect to the diagonal cracks near the span joints. The relatively flexible bridge structural system combined with the heavy ADT on the bridge might have contributed to the development some cracks. All cracks measured are hairline crack with a width of less than 0.031 in. Typical crack patterns on the bridge decks are shown in photos 3 through 5. In general, the top surface of I-29 southbound bridge was in good condition, with only hairline cracks found, showing that HPC designs provide significantly higher strength that can lead to more efficient designs and improved durability.

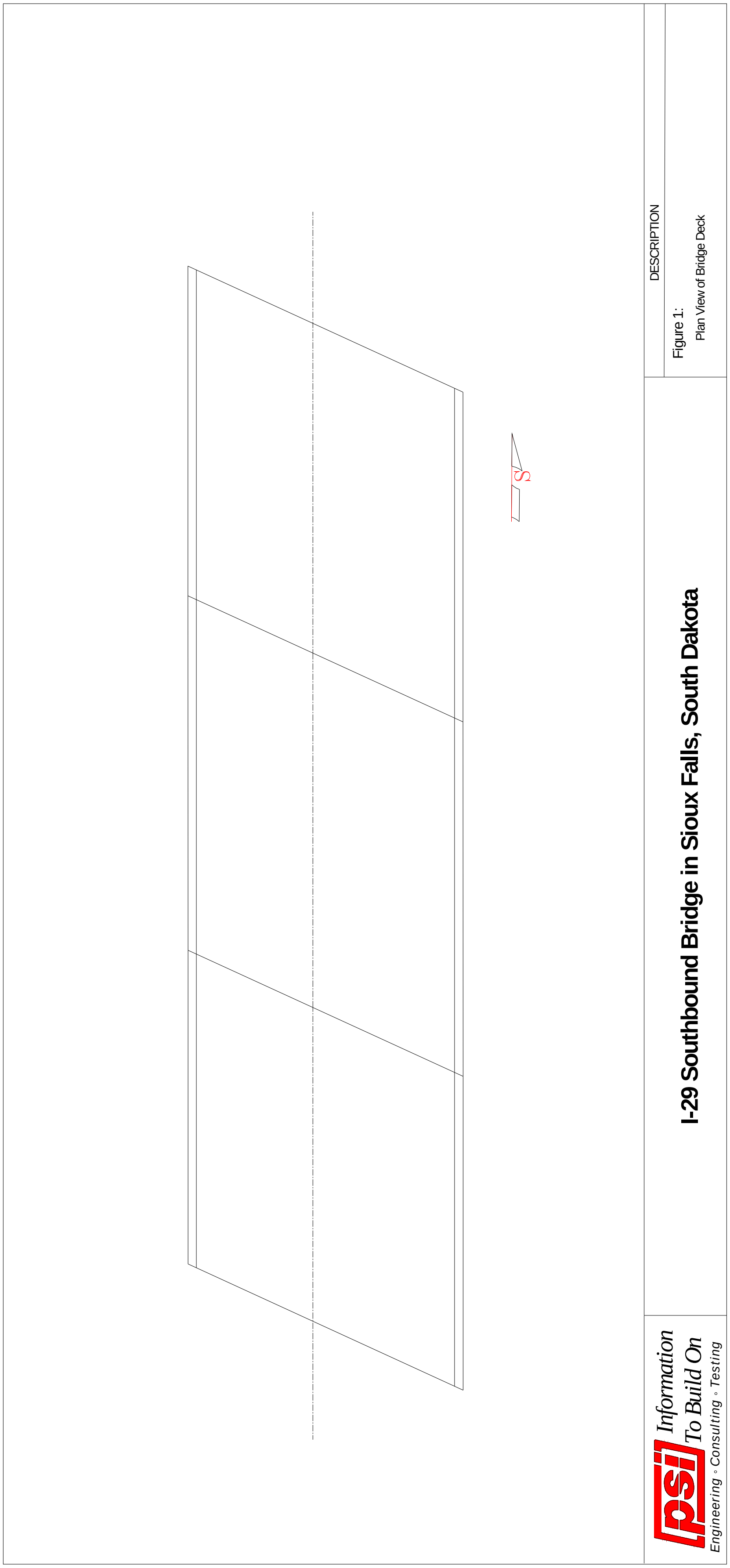
Petrographic examination was performed on core samples retrieved from the I-29 southbound bridge; six concrete cores of 3-3/4-in. diameter, 2- to 3-1/2-in. long were received by the Petrographic Laboratory. The identification on the cores was as follows: SDSB-1, SDSB-2, SDSB-3, SDSB-4, SDSB-5, and SDSB-6.

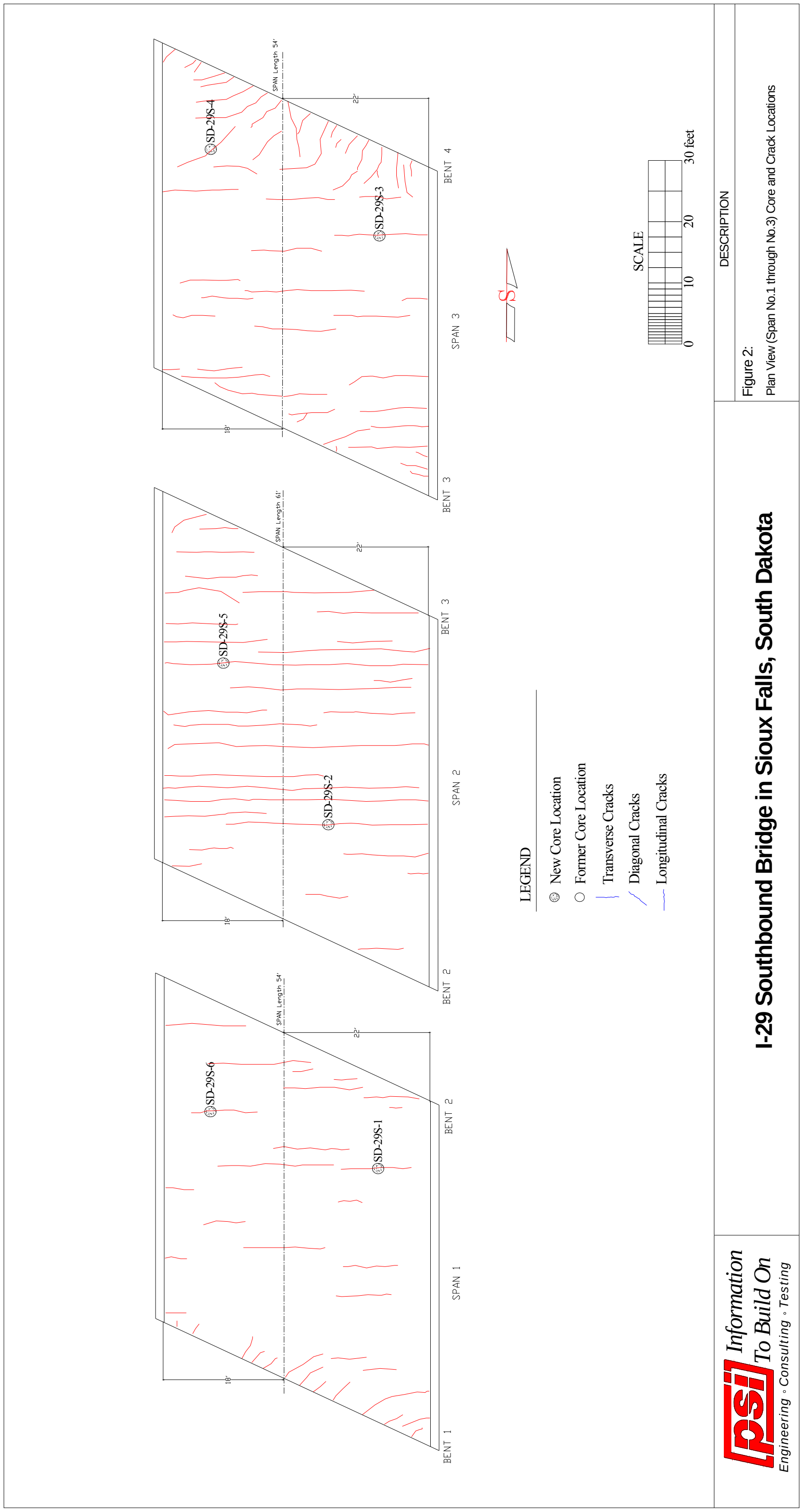
The coarse aggregate in the concrete is crushed quartzite. Coarse aggregate particles are mostly angular, and the maximum size is about 1 inch. Preferential orientation of coarse aggregate particles is not observed in this concrete, nor is segregation. The fine aggregate fraction is mainly composed of quartz, with small amounts of quartzite, limestone, dolomite, sandstone, and chert. The fine aggregate is from natural sand and the particles appear rounded to angular. Both coarse aggregate and fine aggregate contain chalcedony, which is commonly considered as a potentially alkali-silica reactive constituent. However, no evidence of alkali-silica reaction is found in the concrete samples.

In general, the bond between the aggregate and the paste appears moderately strong. However, cracks do exist at the paste-aggregate interface. The cause of the cracking in three of the cores is uncertain. It is speculated that shrinkage, among other mechanisms, may be the major cause of the cracking. Chalcedony, a potentially alkali-silica reactive form of silica, is present in both coarse aggregate and fine aggregate. However, there is no evidence of alkali-silica reaction in the concrete.

Ettringite crystals formed in air voids. Often, ettringite filled part of a void. But voids fully filled with ettringite are also found in the concrete. There was no evidence of deterioration associated with the existence of the ettringite in the concrete. It is very common to see ettringite as a secondary deposit in concrete.

Respectfully Submitted,
Professional Service Industries, Inc.
Structural Investigation & Petrography Department


DESCRIPTION
Figure 1:
Plan View of Bridge Deck
S
I-29 Southbound Bridge in Sioux Falls, South Dakota
Information
To Build On
Engineering • Consulting • Testing


I-29 Southbound Bridge in Sioux Falls, South Dakota
Figure 2:
Plan View (Span No.1 through No.3) Core and Crack Locations
DESCRIPTION
LEGEND
New Core Location
Former Core Location
Transverse Cracks
Diagonal Cracks
Longitudinal Cracks
SCALE
0
10
20
30 feet
S
SPAN 1
SPAN 2
SPAN 3
BENT 1
BENT 2
BENT 3
BENT 4
SD-29S-1
SD-29S-2
SD-29S-3
SD-29S-4
SD-29S-5
SD-29S-6
psi
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Photographic Documentation

	Photo 1: General Overview looking of the I-29 Southbound Bridge from the side.
	Photo 2: View looking north from the southern end of the Southbound Bridge.
	Photo 3: Crack patterns on the Southbound Bridge.

	Photo 4: Close up view of diagonal crack near the joint.
	Photo 5: Close up view of transverse cracks.
	Photo 6: Close up view of crack pattern on Span 2 of Southbound Bridge.

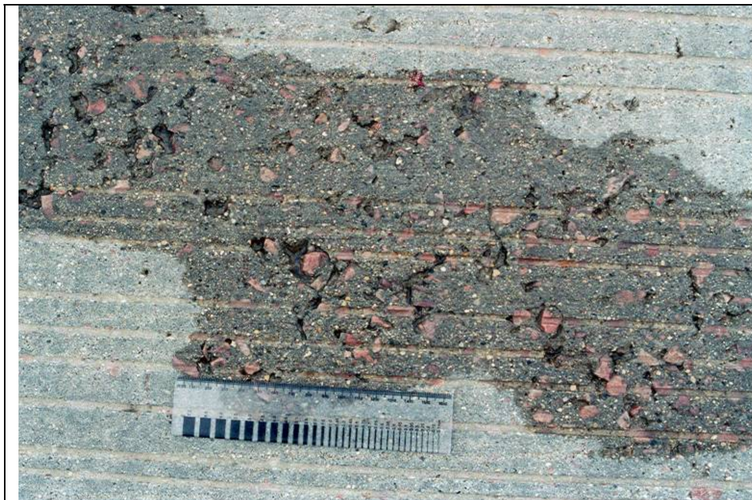


Photo 7: Honeycomb defects observed on Span 2 of Southbound Bridge.

Photo 8: General overview of underside of the bridge.

Photo 9: Close up view of patched core location on Southbound Bridge. Note the crack over the core.

APPENDIX L – SUPPLEMENT 1

I-29 Southbound Bridge, South Dakota
Petrographic Report

PETROGRAPHIC EXAMINATION OF CONCRETE CORES FROM A BRIDGE IN SOUTH DAKOTA (SD 29S)

A report submitted to the Structures Laboratory at TFHRC

Rongtang Liu
Petrographic Laboratory
FHWA TFHRC
(Reviewed by Richard Meininger, PE; Concrete Laboratory; printed in color 9-12-2006)

June 16, 2006

Abstract

Six concrete cores were received from the Structures Laboratory of the Turner-Fairbank Highway Research Center for petrographic examination. Reportedly, the cores were collected from a concrete bridge in South Dakota.

Petrographic examination was performed on samples using optical microscopes. Thin sections, as well as polished slabs, were made from the concrete cores for microscopic examination.

Visual inspection of the concrete cores revealed that two cores (SDSB-3 and SDSB-4) have cracked longitudinally. No defects were observed in the other cores. The findings from microscopic examination indicate that the concrete has entrained air voids; the hydration of the cement was reasonable; the presence of some unhydrated cement particles was also observed in the cement paste; cracks existed in the paste as well as at the paste-aggregate interface; ettringite as secondary deposit formed in some of the air voids; no evidence of alkali-silica reaction was observed.

Introduction

The Petrographic Laboratory of the FHWA Turner-Fairbank Highway Research Center was asked by the Structures Laboratory to examine a set of concrete cores retrieved from a bridge in South Dakota. Six concrete cores of 3-3/4-in. diameter, 2- to 3-1/2-in. long were received by the Petrographic Laboratory. The identification on the cores was as follows: SDSB-1, SDSB-2, SDSB-3, SDSB-4, SDSB-5, and SDSB-6.

Laboratory Procedures

Petrographic examination of the concrete samples was performed in accordance with ASTM C 856, “Standard Practice for Petrographic Examination of Hardened Concrete.” Sections were polished and examined using a stereomicroscope at magnifications up to 200×. Small rectangular blocks were cut from concrete samples. One surface of each block was polished, dried, placed on petrographic slide with low−viscosity epoxy resin,

and reduced to a thickness of approximately 25 micrometers (0.001 in.). These thin sections were examined using a polarized–light microscope at magnification up to 400×, to determine aggregate mineralogy, paste characteristics, and microstructure.

Two ¾-inch thick slabs were sawn from the concrete cores. The slabs were ground and polished to obtain a smooth and plane surface. They were examined using a stereomicroscope at magnifications up to 200×.

Findings

Six thin section samples taken from the received cores were examined using the polarized light microscope. Two polished sections were examined using the stereomicroscope. The findings are summarized as follows:

Aggregates

The coarse aggregate in the concrete is crushed quartzite. Coarse aggregate particles are mostly angular, and the maximum size is about 1 inch. Preferential orientation of coarse aggregate particles is not observed in this concrete, nor is segregation.

The fine aggregate fraction is mainly composed of quartz, with small amounts of quartzite, limestone, dolomite, sandstone, and chert. The fine aggregate is from natural sand and the particles appear rounded to angular.

Both coarse aggregate and fine aggregate contain chalcedony, which is commonly considered as a potentially alkali-silica reactive constituent. However, no evidence of alkali-silica reaction is found in the concrete samples.

Cement Paste

The cement is well hydrated and the paste contains some unhydrated cement particles as seen under the microscope (Figure L1-1). Fly ash (as shown in Figure L1-2) is present in the cement matrix.

Air Voids

Small, spherical air voids are observed in the concrete (Figure L1-3), hence the concrete was air entrained. Entrained air voids are well distributed in the concrete. A small amount of entrapped air voids are also present in the concrete.

Cement-Aggregate Bonding

In general, the cement/aggregate interface is moderately dense and fairly strong, as shown in Figure L1-4.

Secondary Deposit

Ettringite is observed in some air voids in the concrete. Very often, ettringite crystals filled up a portion of a void, as shown in Figures L1-5, L1-6, and L1-7. Occasionally, voids fully filled with ettringite are also found in the concrete (Figure L1-7, left side).

Cracking

There are visible cracks in two of the cores (SDSB-3 and SDSB-4). The cause of the cracking in two of the cores is uncertain. It is speculated that shrinkage may be the cause of the cracking. There is no evidence of any material related deterioration in the concrete.

Cracks are also observed under the microscope. Micro-cracking is present in the cement paste, as shown in Figure L1-8 and Figure L1-9. Cracks are also found at the paste-aggregate interface (Figure L1-10 and Figure L1-11). These cracks are probably due to drying shrinkage, although other mechanisms may also contribute to the distress.

Summary

The concrete was air entrained, and the entrained air voids were well distributed in the concrete. In general, the bond between the aggregate and the paste appears moderately strong. However, cracks do exist at the paste-aggregate interface. The cause of the cracking in three of the cores is uncertain. It is speculated that shrinkage, among other mechanisms, may be the major cause of the cracking. Chalcedony, a potentially alkali-silica reactive form of silica, is present in both coarse aggregate and fine aggregate. However, there is no evidence of alkali-silica reaction in the concrete.

Ettringite crystals formed in air voids. Often, ettringite filled part of a void. But voids fully filled with ettringite are also found in the concrete. There was no evidence of deterioration associated with the existence of the ettringite in the concrete. It is very common to see ettringite as a secondary deposit in concrete.

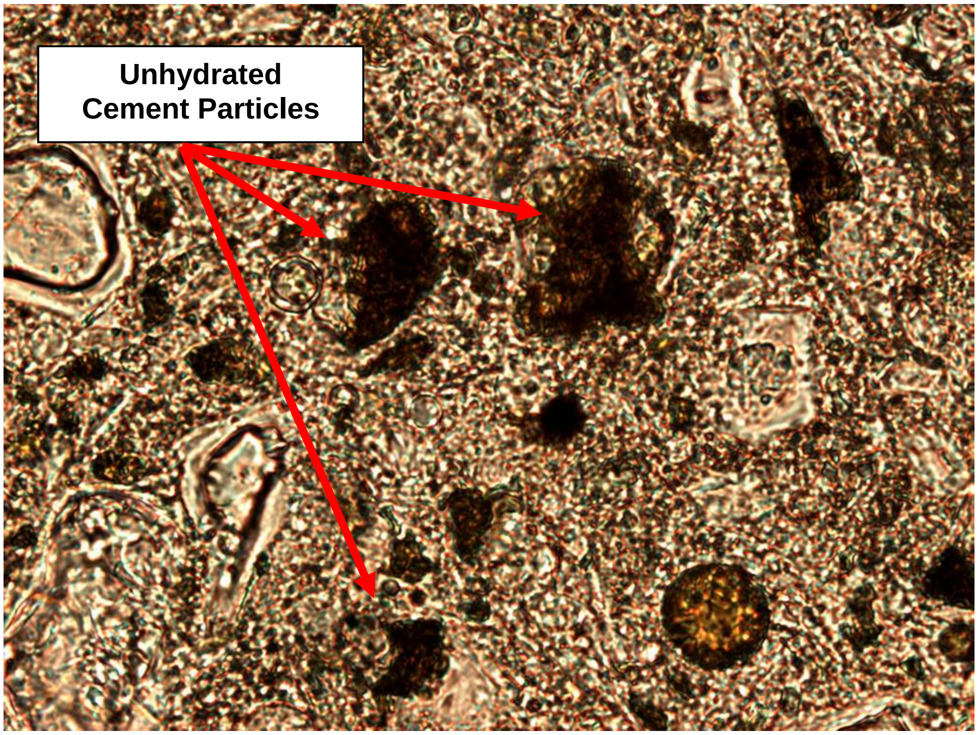


Figure L1-1: Unhydrated cement particles in paste. Width of field is 0.33 mm. Thin section image.

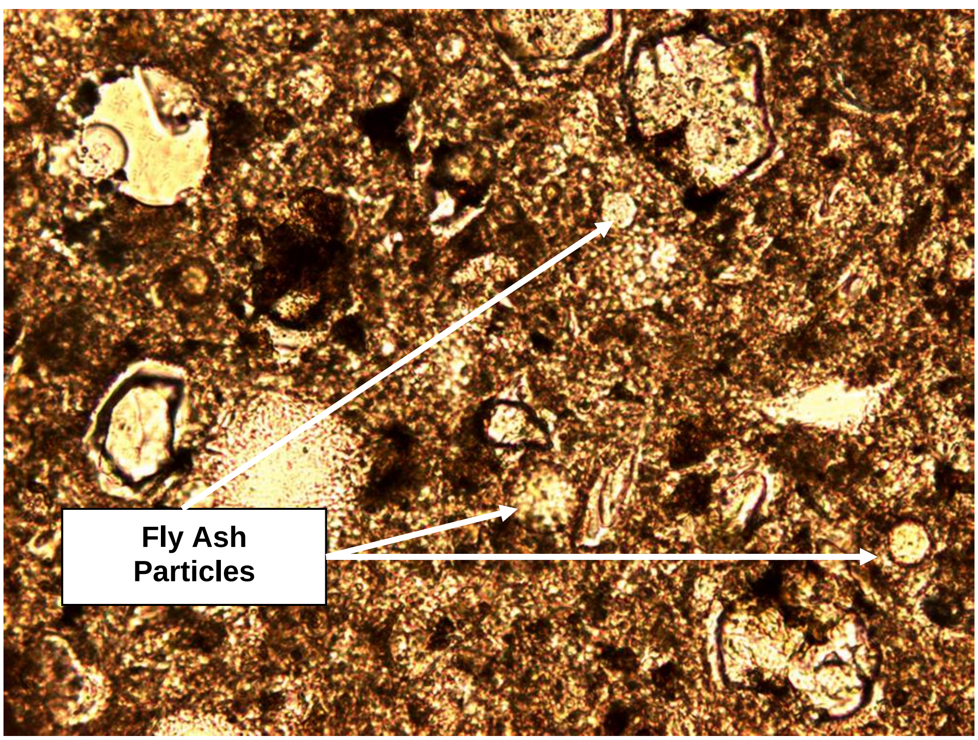


Figure L1-2: Fly ash particles in the cement matrix. Width of field is 0.33 mm. Thin section image.

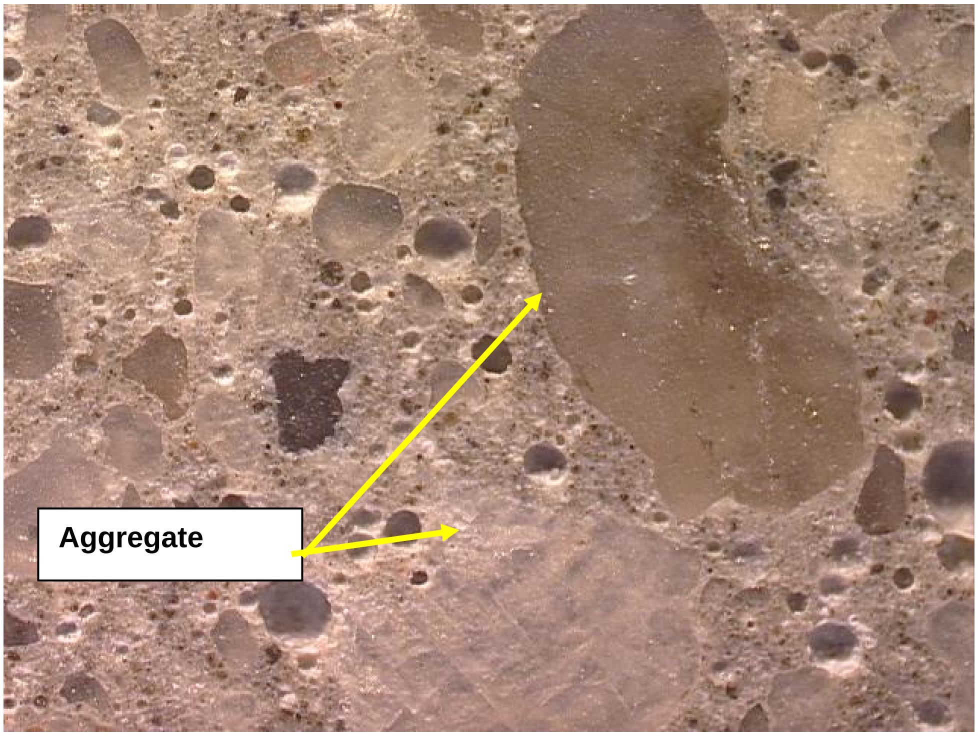


Figure L1-3 Entrained air voids in the concrete. Width of field is 4.0 mm. Polished surface image.

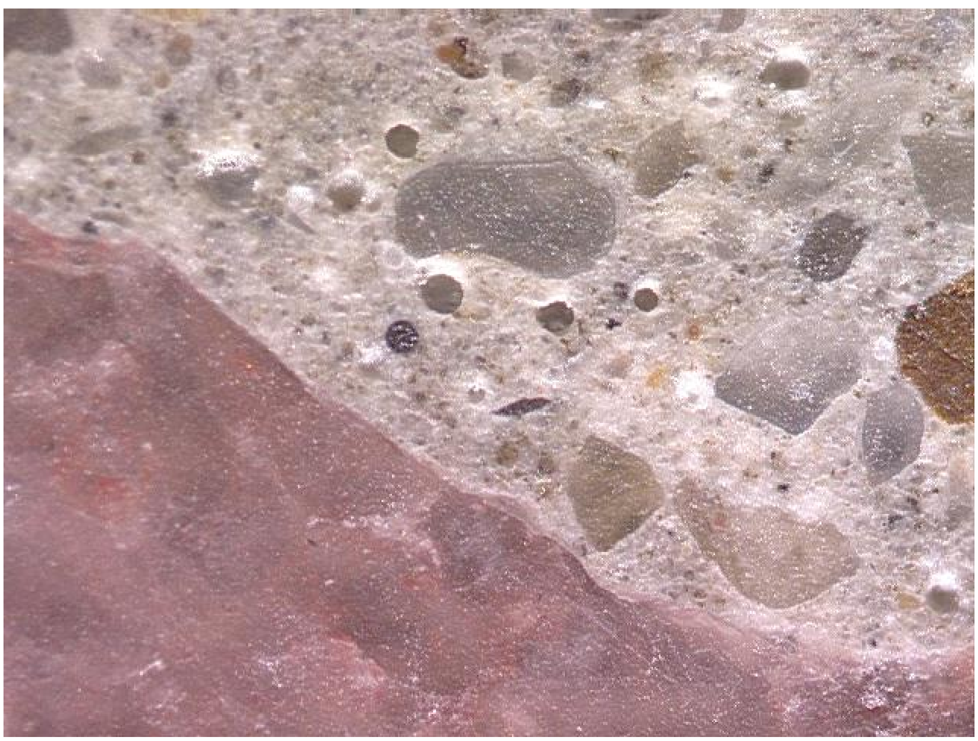


Figure L1-4: The bonding between aggregate and cement paste is moderately strong. Width of field is 2.0 mm. Polished surface image.

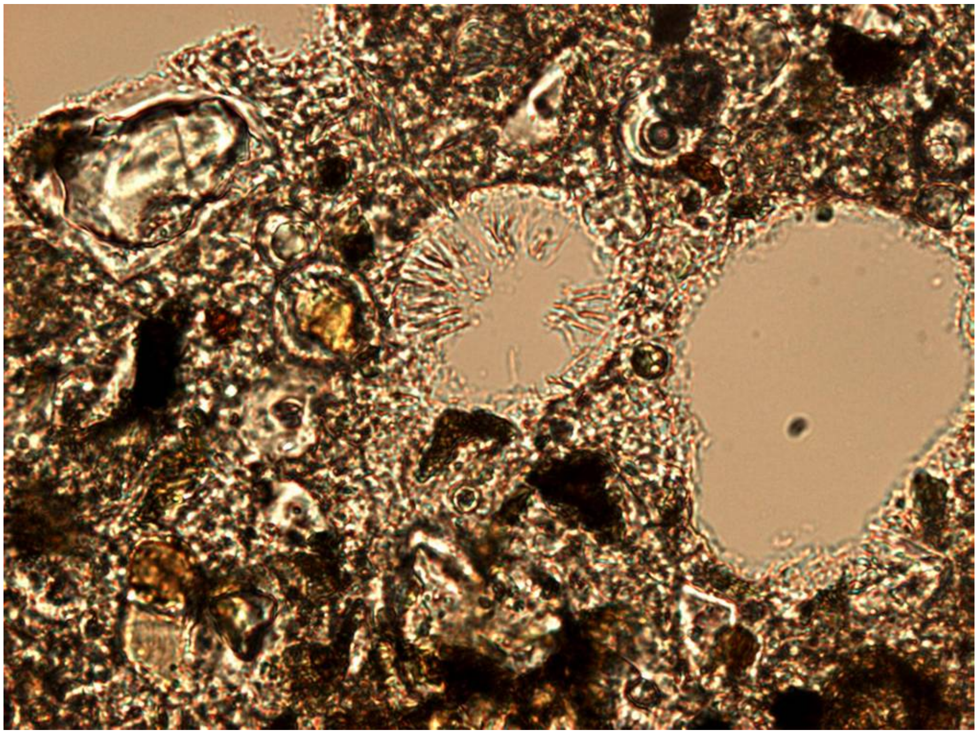


Figure L1-5: Image of ettringite in an air void. Width of field is 0.165 mm. Thin section image 0.165 mm.

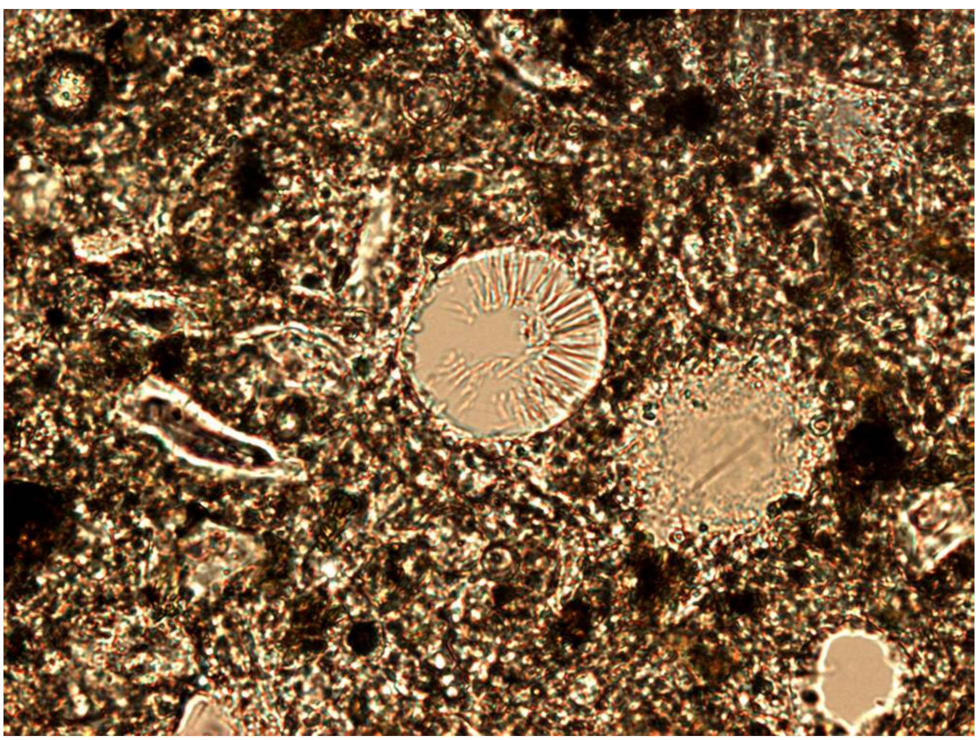


Figure L1-6: Ettringite in an air void. Width of field is 0.165 mm. Thin section image.

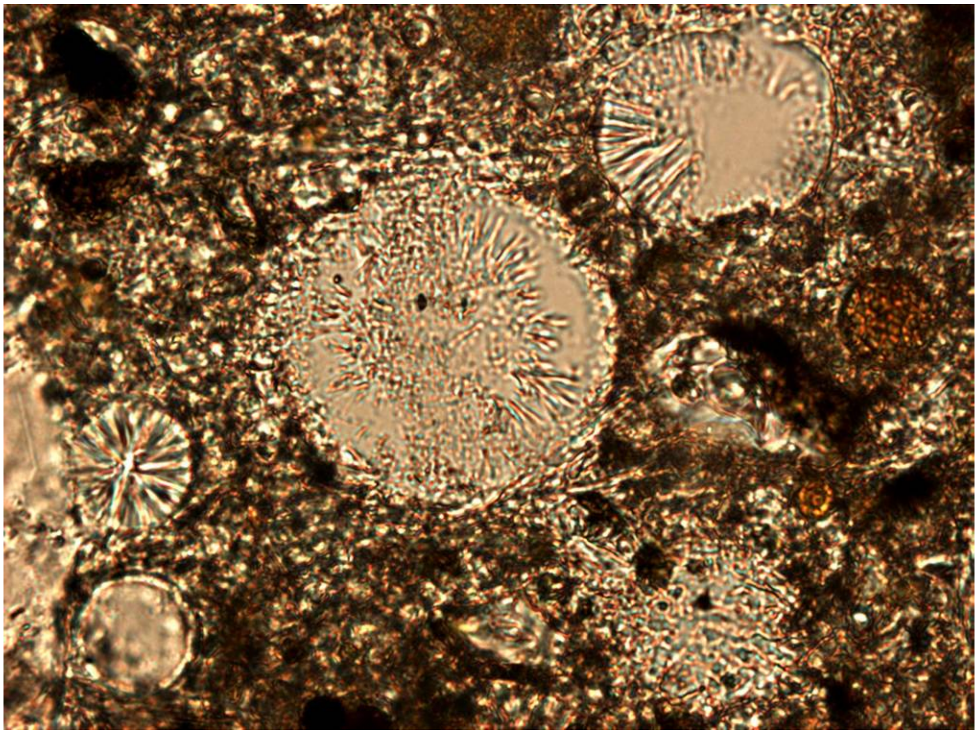


Figure L1-7: Ettringite in air voids. Width of field is 0.165 mm. Thin section image. Note the void on the left is fully filled with ettringite.

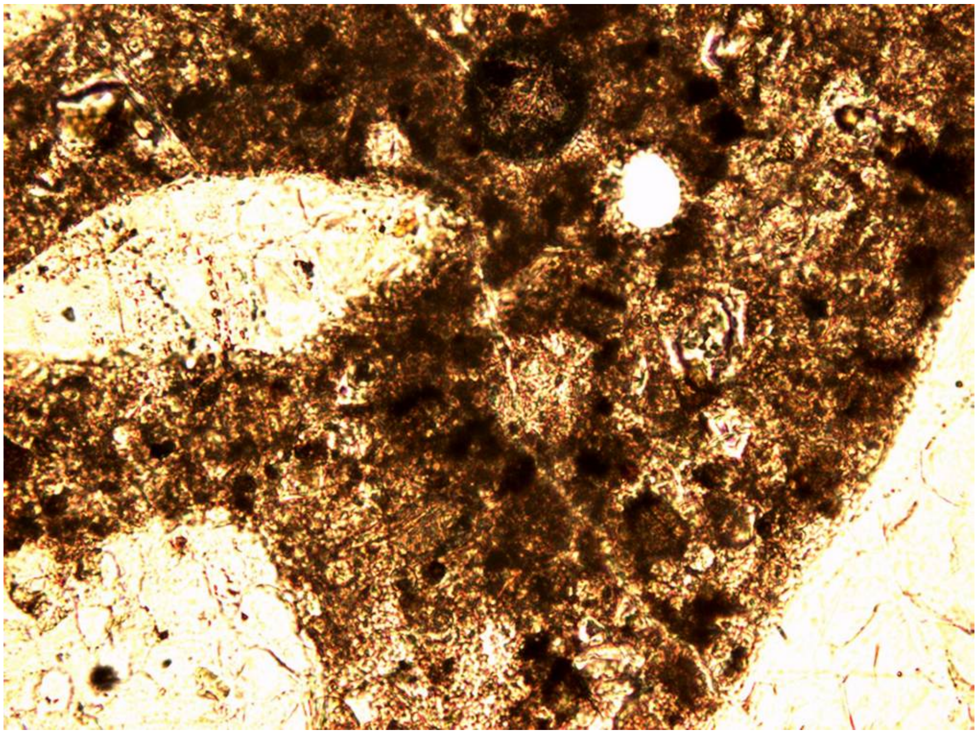


Figure L1-8: Crack in the cement paste matrix. Width of field is 0.33 mm. Thin section image.

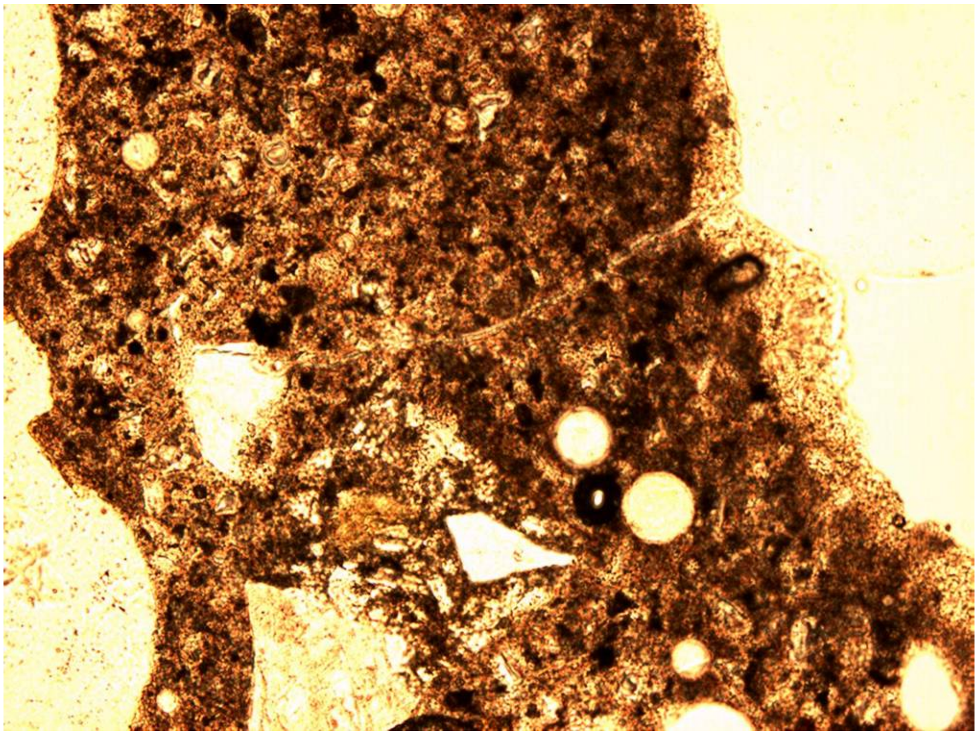


Figure L1-9: Another crack in the cement paste matrix. Width of field is 0.33 mm. Thin section image.

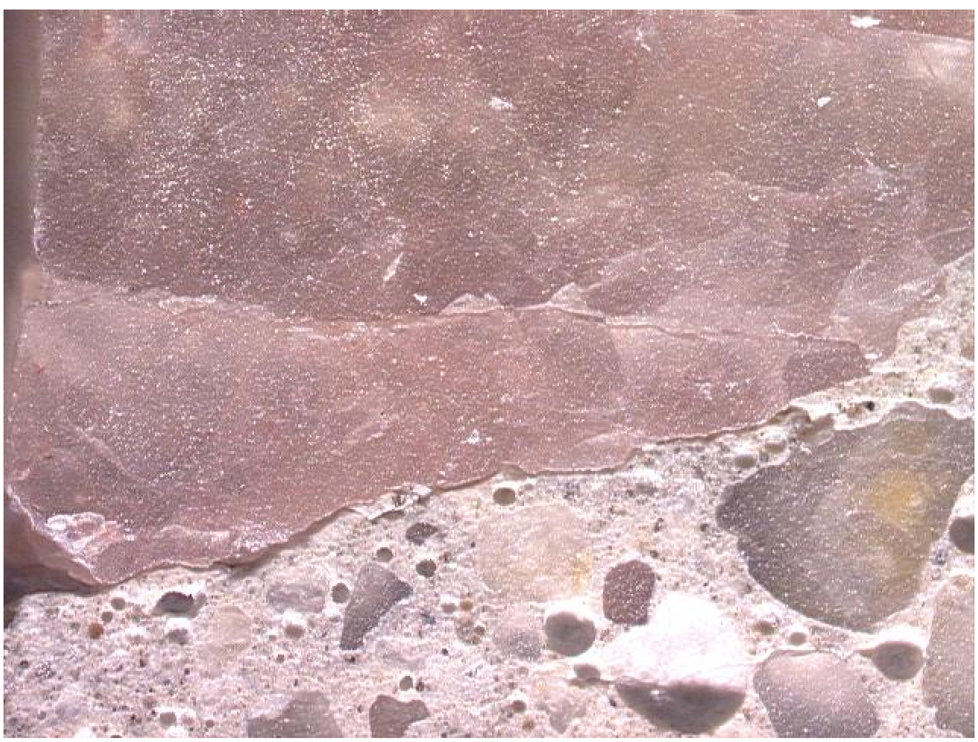


Figure L1-10: Crack at the aggregate-paste interface. Width of field is 4.0 mm. Polished surface image.

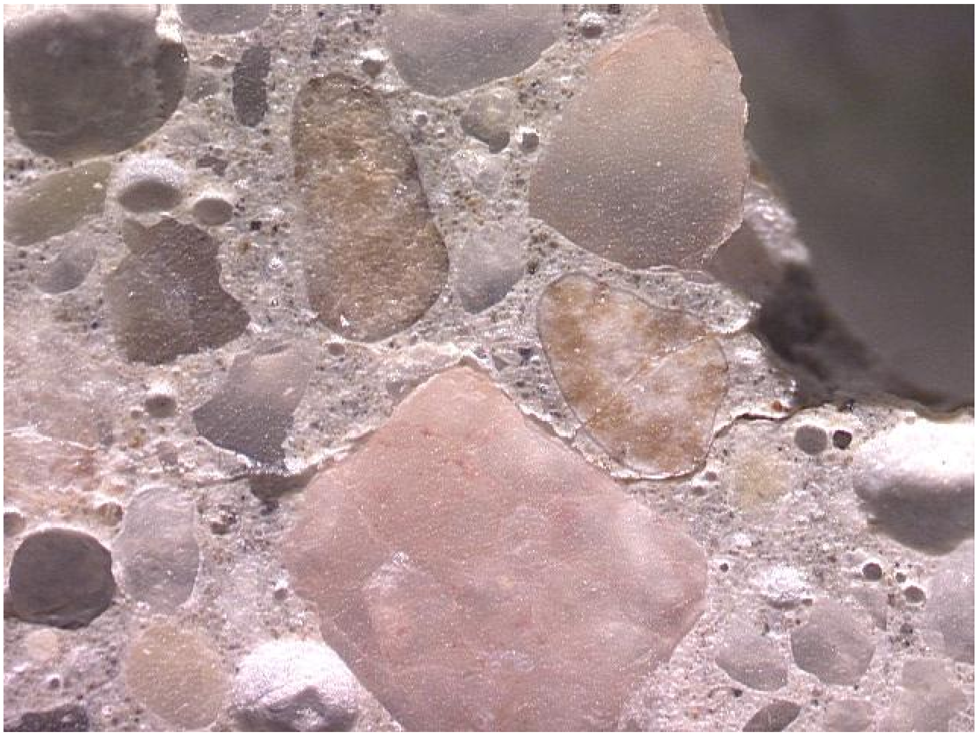


Figure L1-11: Crack at the aggregate-paste interface (crushed pink quartzite). Width of field is 4.0 mm. Polished surface image. Also a crack through an aggregate particle (a rounded natural sand particle with a weathering rim).

APPENDIX L – Supplement 2

I-29 Southbound Bridge, South Dakota
Survey Checklist

Checklist

The following checklist is adapted from 201.1R-92, and provided to facilitate a thorough survey. The definition of terms and associated photographs in 201.1 R-92, are utilized to standardize the reporting of the condition of the concrete in the structures.

1. Description of structure or pavement
    - 1.1 Name, location, type, and size I-29 Southbound Bridge in Minnehaha County, near Sioux Falls was built in the summer of 2000. It is less than a half mile away from the I-29 Northbound Bridge, and also a railroad overpass structure. The bridge consisted of typical three-span precast, prestressed concrete girders with standard integral abutments and integral bent diaphragms. Total length is 172 ft long. The clear width of the deck is 40 ft, including two through-traffic lanes and two shoulders.
    - 1.2 Owner, project engineer, contractor, when built
      Owner: South Dakota Department of Transportation. This bridge is part of a demonstration project for HPC in bridge structures which were co-sponsored by the Federal Highway Administration (FHWA) and the South Dakota Department of Transportation.
    - 1.3 Design
        - 1.3.1 Architect and/or engineer: the South Dakota Department of Transportation
        - 1.3.2 Intended use and history of use: To carry high volume of traffic on I-29 Southbound.
        - 1.3.10 Special features: HPC deck with specified strength of 4500 psi at 28days was used. The girder concrete was 9900 psi (68.3 MPa) at 28 days and 8250 psi (56.9 MPa) at release of the strands. fly ash (17%) was used.
    - 1.4 Construction
        - 1.4.1 Contractor-general: NA
        - 1.4.2 Subcontractors concrete placement: NA
        - 1.4.3 Concrete Supplier: NA
        - 1.4.4 Agency responsible for testing: the South Dakota Department of Transportation, South Dakota School of Mines and Technology, and South Dakota State University
        - 1.4.5 Other subcontractors: NA
    - 1.5 Photographs
        - 1.5.1 General view Photos 1 through 2
        - 1.5.2 Detailed close up of condition of area Photos 3 through 9
    - 1.17 Sketch map-orientation showing sunny and shady and well and poorly drained regions N/A

2. Present condition of structure Date of Evaluation: The week of June 14, 2004
    - 2.1 Overall alignment of structure No signs of misalignment
        - 2.1.1 Settlement
        - 2.1.2 Deflection
        - 2.1.3 Expansion

2.1.4 Contraction
2.2 Portions showing distress (beams, columns, pavement, walls, etc., subjected to strains and pressures) none observed
2.3 Surface condition of concrete
2.3.1 General (good, satisfactory, poor, dusting, chalking, blisters) Good
2.3.2 Cracks Longitudinal, transverse, and diagonal
2.3.2.1 Location and frequency See Figure 2

2.3.2.22 Type and size (see Definitions) See Figure 2

Transverse Observed
Width (from Crack comparator) Hairline

- Hairline (Less than 1/32 in.)
- Fine (1/32 in. - 1/16 in.)
- Medium (1/16 – 1/8 in.)
- Wide (Greater than 1/8 in.)

Craze N/A
Width (from Crack comparator) ____________

- Hairline (Less than 1/32 in.)
- Fine (1/32 in. - 1/16 in.)
- Medium (1/16 – 1/8 in.)
- Wide (Greater than 1/8 in.)

Map N/A
Width (from Crack comparator) ____________

- Hairline (Less than 1/32 in.)
- Fine (1/32 in. - 1/16 in.)
- Medium (1/16 – 1/8 in.)
- Wide (Greater than 1/8 in.)

D-Cracking N/A
Width (from Crack comparator) ____________

- Hairline (Less than 1/32 in.)
- Fine (1/32 in. - 1/16 in.)
- Medium (1/16 – 1/8 in.)
- Wide (Greater than 1/8 in.)

Diagonal NA
Width (from Crack comparator) NA

- Hairline (Less than 1/32 in.)
- Fine (1/32 in. - 1/16 in.)
- Medium (1/16 – 1/8 in.)
- Wide (Greater than 1/8 in.)

2.3.2.23 Leaching, stalactites N/A
2.3.3 Scaling N/A
2.3.3.1 Area, depth ____________
2.3.3.12 Type (see Definitions) ____________

- Light (Less than 1/8 in.)
- Medium (1/8 in. – 3/8 in.)
- Severe (3/8 in. – 3/4 in.)

Very Severe (Greater than 3/4 in.)

2.3.4 Spalls and popouts none observed

2.3.4.1 Number, size, and depth NA

2.3.4.12 Type (see Definitions) NA

Spalls

Small (Less than 3/4 in. depth)

Large (Greater than 3/4 in. depth)

Popouts

Small (Less than 3/8 in. diameter)

Medium (3/8 in. – 2 in. diameter)

Large (Greater than 2 in. diameter)

2.3.5 Extent of corrosion or chemical attack, abrasion, impact, cavitation N/A

2.3.6 Stains, efflorescence observed at underside of the bridge

2.3.7 Exposed reinforcement none

2.3.8 Curling and warping N/A

2.3.9 Previous patching or other repair N/A

2.3.10 Surface coatings N/A

2.3.10.1 Type and thickness N/A

2.3.10.2 Bond to concrete N/A

2.3.10.3 Condition N/A

2.3.11 Abrasion N/A

2.3.12 Penetrating sealers ___

2.3.12.1 Type N/A

2.3.12.2 Effectiveness N/A

2.3.12.13 Discoloration N/A

2.4 Interior condition of concrete (in situ and samples) N/A

2.4.1 Strength of cores

2.4.2 Density of cores

2.4.3 Moisture content

2.4.4 Evidence of alkali-aggregate or other reactions N/A

2.4.5 Bond to aggregate, reinforcing steel, joints N/A

2.4.6 Pulse velocity

2.4.7 Volume change

2.4.8 Air content and distribution

2.4.9 Chloride-ion content

2.4.10 Cover over reinforcing steel

2.4.11 Half-cell potential to reinforcing steel.

2.4.12 Evidence of reinforcement corrosion

2.4.13 Evidence of corrosion of dissimilar metals

2.4.24 Delaminations N/A

2.4.24.1 Location N/A

2.4.24.2 Number, and size N/A

2.4.15 Depth of carbonation

2.4.16 Freezing and thawing distress (frost damage)

2.4.17 Extent of deterioration

2.4.28 Aggregate proportioning, and distribution

3. Nature of loading and detrimental elements
 3.1 Exposure
 3.1.1 Environment (arid, subtropical, marine, freshwater, industrial, etc.) Arid
 3.1.2 Weather-(July and January mean temperatures, Max 110ºF, min. -36ºF
 Mean annual rainfall and 25.1-in
 Months in which 60 percent of it occurs) June
 3.1.3 Freezing and thawing Significant
 3.1.4 Wetting and drying Significant
 3.1.9 Drying under dry atmosphere N/A
 3.1.6 Chemical attack-sulfates, acids, chloride N/A
 3.1.7 Abrasion, erosion, cavitation, impact N/A
 3.1.8 Electric currents N/A
 3.1.9 Deicing chemicals which contain chloride ions N/A
 3.1.10 Heat from adjacent sources N/A
 3.2 Drainage N/A
 3.2.1 Flashing ____
 3.2.2 Weepholes ____
 3.2.3 Contour ____
 3.2.4 Elevation of drains ____
 3.3 Loading Research Test Data Available in Compilation CD Version 3
 3.3.1 Dead ____
 3.3.2 Live ____
 3.3.3 Impact ____
 3.3.4 Vibration ____
 3.3.5 Traffic index ____
 3.3.6 Other ____
 3.4 Soils (foundation conditions) ____
 3.4.1 Compressibility
 3.4.2 Expansive soil
 3.4.3 Settlement
 3.4.4 Resistivity
 3.4.5 Evidence of pumping
 3.4.6 Water table (level and fluctuations)

4. Original condition of structure Good
 4.1 Condition of formed and finished surfaces Good
 4.1.1 Smoothness ____
 4.1.2 Air pockets ("bugholes") ____
 4.1.3 Sand streaks ____
 4.1.4 Honeycomb ____
 4.1.5 Soft areas (retarded hydration) ____
 4.1.6 Cold joints ____
 4.1.27 Staining ____
 4.1.28 Sand pockets ____

4.2	Defects		N/A
4.2.1	Cracking		
4.2.1.1		Plastic shrinkage	
4.2.1.2		Thermal shrinkage	
4.2.1.3		Drying shrinkage	
4.2.12	Curling		

5. Materials of Construction — See Table 1

6. Construction Practices — See Report pg. 2, 3 and 5

APPENDIX M

Porter Road, Tennessee

HIGH PERFORMANCE CONCRETE
BRIDGE DECK INVESTIGATION

PORTER ROAD
Dickson County, Tennessee

I. BACKGROUND

Porter Road bridge over State Route 840 in Dickson County was constructed in 2000. The structure is 318-ft long and 32-ft wide (see photos 1 through 3, and Figure 1). It carries one eastbound lane and one westbound lane of Porter Road. The structure consists of 8-¼-in. thick concrete deck with stay-in-place forms on two 159-ft long continuous span concrete bulb-tee prestressed superstructure, on one concrete pier and two concrete abutments. The structure was built with a 27^{o} skew at both abutments and the pier. Four precast bulb-tee girders, BT-72, on 8 ft - 4 in. centers support each span. The concrete stub abutments are separated from the State Route 840 with loose riprap slope protection. The concrete pier is comprised of a cast-in-place concrete hammerhead cap on a cast-in-place pier stem.

The retaining wall, abutments, bent, girders and deck were constructed with high performance concrete (HPC). The factors that led to the use of HPC in this bridge included longer span length and a more durable structure. If HPC was not used, structural steel was the only other viable option, which would have resulted in an additional cost of about $500,000. The cost of HPC in this bridge was $160/ linear ft for the beams, $315/yd^3 for the deck and $240/yd^3 for the substructure concrete.

II. SCOPE OF SERVICES

Our scope of services on this bridge included a series of tasks and sub-tasks, which are described as follows:

- Collect all available information relevant to the construction of each bridge, including;
    - Deck Concrete Properties
    - Specified Deck Concrete Construction Procedures
    - Approved Concrete Mix Proportions
    - Measured Properties from QC
    - Other Measured Properties
    - Actual Method of Deck Placement
    - ADT & ADTT
    - Exposure Condition of the Bridge
    - Any Performed Maintenance
    - Any Inspection Reports
- Visually inspect the bridge, for the following information:
    - General condition of the deck top surface

- Determination of maximum crack width
- General condition of the deck underside
- General condition of the girders
- Photograph areas of significant deterioration
- Prepare drawings locating defects
- Extract cores

III. COMPILATION OF BRIDGE CONSTRUCTION INFORMATION

Information sources used for this report include bridge drawings, FHWA newsletters and reports, and technical information contained in FHWA's "Compilation and Evaluation of Results from High Performance Concrete Bridge Projects" version 3.

Approved Concrete Mix Proportions

Table 1. As Specified Concrete Properties

Property	Deck
Minimum Cementitious Materials Content:	658 lb/yd^3
Max. Water/Cementitious Materials Ratio:	0.43
Min. Percentage of Silica Fume:	3 %
Max. Percentage of Silica Fume:	8 %
Slump:	3 ± 1 in. (1)
Air Content:	6 ± 2%
Design Compressive Strength:	5000 psi at 28 days
Chloride Permeability (AASHTO T 277):	<1500 coulombs at 28 days (2)

(1) Specified slump is prior to addition of a high-range water-reducer if used. The maximum slump with a high-range water-reducer added shall be 8 in.
(2) One week moist cured at 73 ^oF followed by 3 weeks at 100 $^oF \pm 10$ ^oF.

Specified Concrete Deck Construction Procedures

Curing:	Fogging followed by membrane curing, wet burlap, and vapor barrier. Wet cure for 7 days.
Cylinder Curing:	Fog room
Cylinder Size:	6x12 in.
Flexural Strength:	—
Other QA/QC Requirements:	Two trial batches with a test slab

Approved Concrete Mix Proportions

Table 2. Approved Mix Proportions

	CIP Deck
Cement Brand:	Lonestar
Cement Type:	I
Cement Quantity:	293 kg/m^3
Fly Ash Brand:	Mineral Resources
Fly Ash Type:	C
Fly Ash Quantity:	91 kg/m^3
Silica Fume Brand:	Rheomac SF 100
Silica Fume Quantity:	30 kg/m^3
Fine Aggregate Type:	Sand
Fine Aggregate SG:	2.61
Fine Aggregate Quantity:	662 kg/m^3
Coarse Aggregate, Max. Size:	No. 57
Coarse Aggregate Type:	Crushed limestone
Coarse Aggregate SG:	2.63
Coarse Aggregate Quantity:	1074 kg/m^3
Water:	138 l/m^3
Water/Cementitious Materials Ratio:	0.36

Measured Properties from QC Tests of Production Concrete for Decks

Compressive Strength: (AASHTO T 22)
Curing Procedure for Cylinders: Moist cured

Table 3. Measured Properties from QC

Batch No.	Compressive Strength, psi			
	7 days	28 days	35 days	56 days
	6x12 in	6x12 in	4x8 in	6x12 in
1	5290	8700	7190	8810
	5030	8870	6730	9680
2	4690	7860	6730	8430
	4700	7960	7540	8570
3	5030	8130	8330	8030
	4930	8070	7890	8760
Average	4945	8265	7402	8713

Measured Properties from Research Tests of Production Concrete for Decks

These values were obtained from research tests of production concrete for the deck.

Table 4. Other Measured Properties

Batch No.	Compressive Strength(4), psi			Modulus of Elasticity(5), ksi	Splitting Tensile Strength(6), psi
	28 days		mc Location		
	m	mc		28 days	28 days
1	8487	—	—	4557	800
	7869	—		4554	738
2	8038	9455	Center	4394	788
	7738	—		4247	736
3	8784	9284	Edge	4784	757
	8870	9370		4468	755
Average	8289	9370	—	4501	762

m = moist cured, mc = match cured.
(4) AASHTO T 22. 4x8-in moist cured cylinders.
(5) ASTM C 469. 4x8-in moist cured cylinders.
(6) AASHTO T 198. 4x8-in moist cured cylinders.

Table 5. Chloride Permeability (AASHTO T 277)

Batch No.	Chloride Permeability(7), coulombs	
	28 days	56 days
1	3049	1254
	3105	1215
2	3913	1551
	3901	1538
3	2679	1168
	3045	1056
Match Cure Edge	2721	1008
Match Cure Center	2911	1046

(7) All specimens were moist cured.

Actual Method of Deck Placement

Construction of the deck occurred in January 2000, with the concrete for the deck pumped from a truck located below on the Route 840 alignment. Surface finishing consisted of motorized screed pan with a burlap drag. The deck was cured using water soaked burlap covered with plastic for seven days. Due to the winter weather the entire bridge was draped with plastic and heated with space heaters. Fogging with additional water through pressurized jets was also provided. The milling operation and saw cutting the transverse grooves were performed in September 2001. The ambient temperature at the time of placement was 35-40^{o}F.

Average Daily Traffic (ADT) and Average Daily Truck Traffic (ADTT)

The district identifies this bridge as carrying 400 vehicles per day, with 0% trucks. While the PSI inspection crew was on-site only 10 cars were noted in 4 hours. No trucks were witnessed during this time period. This represents an ADT of approximately 60.

Exposure Condition of the Bridge

The surrounding area is currently residential combined with agricultural land use. The National Weather Service reports that the mean July temperature is 77.8°F, while the mean January temperature is 35.1°F. The mean annual rainfall is 54 in., 64% of the annual amount falls from November to May. The bridge is exposed to freezing and thawing as well as wetting and drying on a seasonal cycle basis.

Performed Maintenance

No documentation of any maintenance performed since construction was found.

Inspection Reports

This bridge, along with the other bridges on the new alignment of Route 840 is not currently entered into the inventory system.

IV. BRIDGE DECK INSPECTION

The bridge deck received a close visual inspection October 1 through 3, 2002, the findings of this inspection are summarized as follows.

General Condition of the Deck Top Surface

Defects on the top surface include transverse cracks, map cracks, diagonal corner cracks in the acute corners, small gouges, one small sand pocket and a large area of surface milling.

Transverse Cracks: Transverse cracks were found primarily along the centerline of the roadway. Figure 2 illustrates the 815 ft of transverse cracks that were identified on the surface of the deck (see Table 6 below). The crack widths ranged from 0.003 to 0.020 in. for the 90 cracks (see photos 4 and 5).

Table 6. Transverse Cracks

Distance from Abutment 1 (ft)	Number	Length Range (ft)	Total Length of Cracks (ft)	Deck Area (ft^2)	Crack Density (ft/ft^2)
2	1	9	9	64	0.1406
10 – 42	14	6-14	140	1,024	0.1367
80 – 110	10	2-4	30	960	0.0313
122 – 168	28	5-15	280	1,472	0.1902
170 – 218	16	5-15	160	1,536	0.1042
225 – 262	10	6-10	80	1,184	0.0676
279 – 315	11	6-15	116	1,152	0.1007
Cumulative	90	2-15	815	10,080	0.0809

Map Cracks: Map cracks were found primarily along the centerline and Eastbound roadway. Figure 2 illustrates the 1,200 ft^2 of map cracks that were identified on the surface of the deck (see Table 7). The crack widths ranged from 0.003 to 0.010 in. and the cracks were generally 8 in. by 8 in. apart (see photos 6, 7 and 8).

Table 7. Map Cracks

Distance from Abut. 1 (ft)	Width (in.)	Area (ft^2)
122 – 218	0.005-0.010	900
255 – 275	0.003	300
Cumulative	0.003-0.010	1,200

Diagonal Cracks: Diagonal cracks were primarily found in the acute corners, SE and NW, of the bridge deck. Figure 2 illustrates the 45 ft of diagonal cracks that were identified on the surface of the deck (see Table 8). The crack widths ranged from 0.002 to 0.010 in. for the 10 cracks (see photo 8).

Table 8. Diagonal Corner Cracks

Distance from Abutment 1 (ft)	Number	Length Range (ft)	Total Length of Cracks (ft)	Deck Area (ft^2)	Crack Density (ft/ft^2)
0-4	6	4-8	30	128	0.2344
315-318	4	2-4	15	96	0.1563
Cumulative	10	2-8	45	10080	0.0045

Longitudinal Cracks: No longitudinal cracks were observed.

In addition to the different types of cracks noted, a few isolated small defects were found. These defects included two gouges ½-in. deep into the deck; both were 1-in. wide, one was 12-in. long, while the other was 6-in. long. The other defect noted was one sandball, caused by inadequate mixing at the time of construction. The sandball was 2-in. long, 1-in. wide, and ¼-in. deep. Approximately 4,700 ft^2 of deck surface has been milled.

MAXIMUM CRACK WIDTH

The maximum width of transverse cracks was measured to be 0.020 in., while the maximum width of map cracks and diagonal cracks was measured to be 0.010 in.

GENERAL CONDITION OF THE DECK UNDERSIDE

The underside of the deck was not visible due to the presence of stay-in-place deck forms. However, the exterior cantilever portions of the deck were exposed, which exhibit no signs of distress. Photo 9 shows a general view of the underside of the deck.

General Condition of the Girders

The girders were inspected from the ground, without the aid of any access equipment. No signs of distress were noted on any of the girders. Girders 3 (3B1) and 4 (3C1) in Span 1 are instrumented with both electrical strain gauges and a mechanical string line (see photos 10 and 11).

EXTRACTED CORES

Six cores were retrieved from the deck, Figure 1 illustrates their locations. The locations were selected to distribute the samples along each shoulder of the bridge, since Tennessee DOT had requested that the coring and patching operation avoid the traveled lanes. The cores were labeled TNP-1 through TNP-6 (see Table 9) and transferred to FHWA on January 7, 2003, for further investigation.

Table 9. Core Dimensions

Sample	TNP-1	TNP-2	TNP-3	TNP-4	TNP-5	TNP-6
Diameter (in.)	4	4	4	4	4	4
Length (in.)	4	3-½	3-¼	3-½	3-¾	4

Preliminary Conclusions

The construction of the Porter Road Bridge was part of a demonstration project for HPC in bridge structures. It was completed in May 2000.

The visual inspection of the bridge deck as part of the study was performed about a year and a half after the bridge was opened to traffic. A total of 90 transverse cracks were observed. There were 10 diagonal corner cracks and the deck exhibited map cracks primarily along the centerline and Eastbound roadway. The map cracking encompassed about 1,200 ft^2 of the deck surface and the crack widths ranged from 0.003 to 0.010 in. and the cracks were generally 8 in. by 8 in. apart.

In general, the work on the bridge showed that HPC designs provided significantly higher strength that can lead to more efficient designs and improved durability.

Petrographic analysis of six core samples retrieved from the deck was performed at Turner-Fairbank Highway Research Center.

The coarse aggregate in the concrete was crushed stone, and was primarily limestone, with small amount of sandstone and dolomite. The shape of coarse aggregate particles was angular, and the maximum size was about 1/2 inch. Preferential orientation of coarse aggregate particles was not observed in this concrete, nor was segregation. The fine aggregate fraction was mainly composed of quartz, with a small portion of chert, sandstone, and quartzite. The fine aggregate was from natural sand and the particles appeared rounded to angular.

The cement was reasonably hydrated and the paste contained some unhydrated cement particles. Small, spherical air voids were observed in the concrete; however, the air content was estimated to be low.

Isolated cracks in cement paste were sporadically observed in the concrete. In general, the paste/aggregate interface appeared solid and dense. A small amount of ettringite was found sporadically in air voids in the concrete.

The concrete was air entrained, and the air content was low. Sporadic and isolated micro-cracks existed in cement paste, as well as in the aggregate/paste interface. Despite the defects in microscopical scale, the concrete appeared solid and sound.

Ettringite crystals form in some air voids. There was no evidence of deterioration related with the existence of the ettringite in the concrete. It is very common to see ettringite as secondary deposit in concrete.

Respectfully Submitted,
Professional Service Industries, Inc.
Structural Investigation & Petrography Department

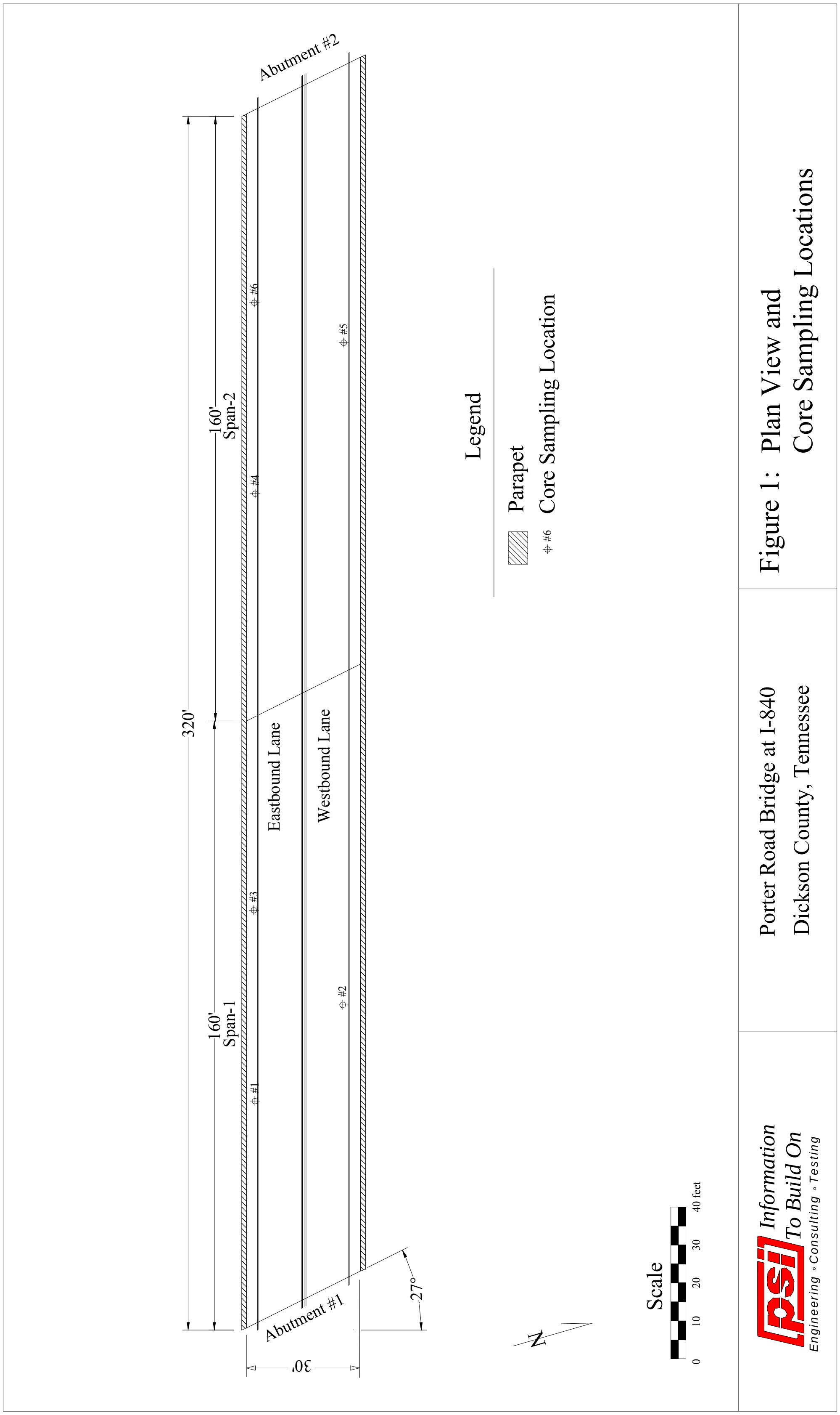


Figure 1: Plan View and Core Sampling Locations

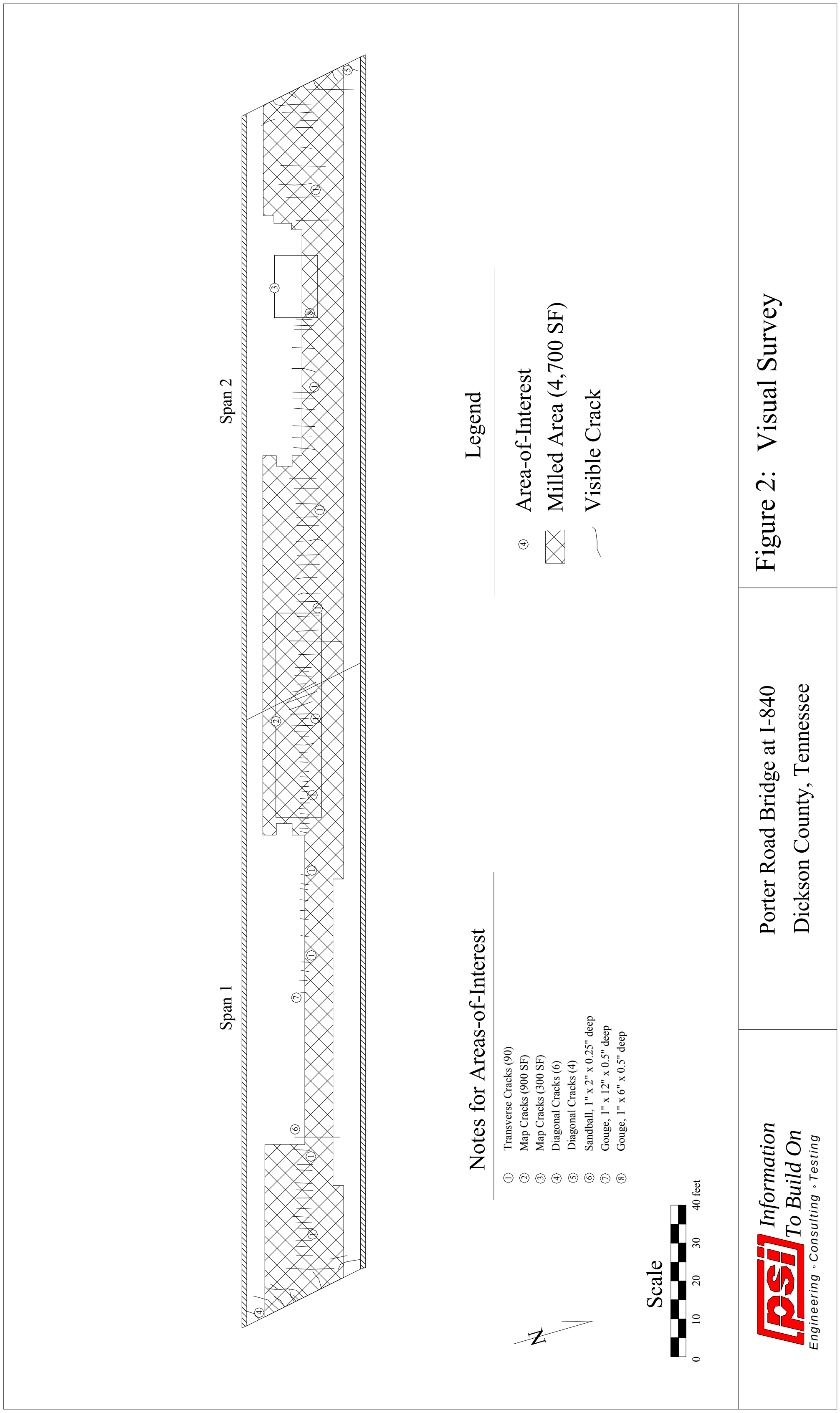


Figure 2: Visual Survey

Photographic Documentation

	Photo 1. General view, South Elevation.
	Photo 2. General view, East Approach.
	Photo 3. General view, West Approach.

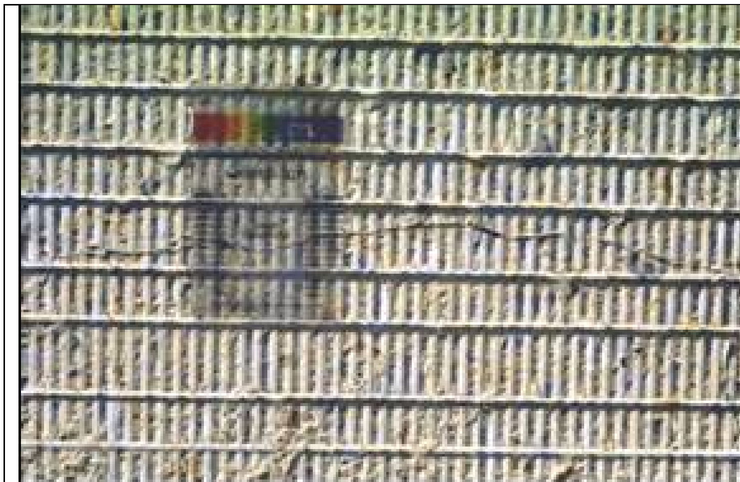
	Photo 4. Typical Transverse Crack (0.020 in. width).
	Photo 5. Typical Transverse Crack (0.016 in. width).

Photo 6. Typical Map Crack
General view.

Photo 7. Typical Map Crack
(Detail view 0.010 in).

Photo 8. Typical Diagonal Crack
(Detail view 0.010 in).

Photo 9. Underside of Deck (Note Stay-in-Place Forms and Connections for Instrumentation.

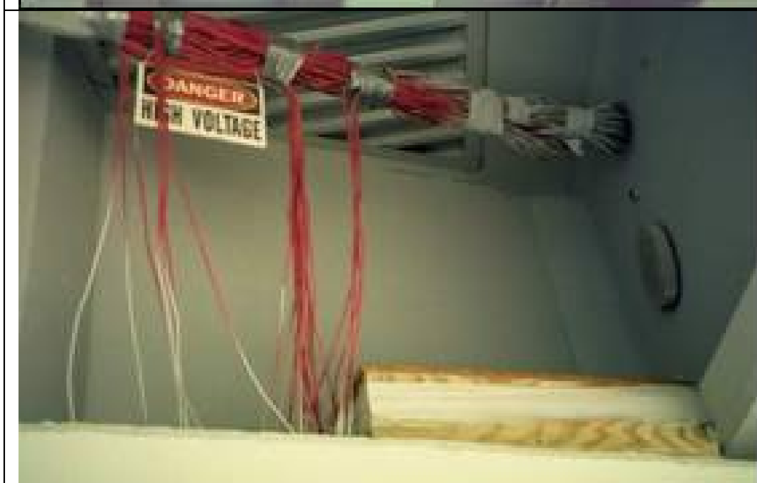


Photo 10. Underside of Deck (Note Instrumentation at Abutment 1, Span 1) Interior Girder 3 on Right.

Photo 11. Underside of Deck (Note Instrumentation at Abutment 1, Span 1) Exterior Girder 4 on Left.

APPENDIX M – Supplement 1

Porter Road, Tennessee
Petrographic Examination

PETROGRAPHIC EXAMINATION OF SIX CONCRETE CORES FROM TENNESSEE (TNP)

A report submitted to the Structures Laboratory at TFHRC

Rongtang Liu
Petrographic Laboratory
FHWA TFHRC

April 4, 2005

Abstract

Six concrete cores were received from the Structures Laboratory of the Turner-Fairbank Highway Research Center for petrographic examination. Reportedly, the concrete cores were collected from a concrete bridge in Tennessee.

Petrographic examination was performed on samples using optical microscopes. Thin sections, as well as polished slabs, were made from the concrete cores for microscopic examination.

The concrete appears sound and solid. Visual inspection of the concrete cores revealed no defects. The findings from microscopic examination indicate that the concrete has entrained air voids, but the air content is low. The hydration of the cement was reasonable. The presence of unhydrated cement particles was observed in the cement paste. Sporadic and isolated micro- cracks were present in the paste, as well as in the paste/aggregate interfacial region.

Introduction

The Petrographic Laboratory of the FHWA Turner-Fairbank Highway Research Center was asked by the Structures Laboratory to examine a set of concrete cores retrieved from a bridge in Tennessee. Six concrete cores of 4-in. diameter, 3-1/2- to 4-in. long were received by the Petrographic Laboratory. The identification on the cores was as follows: TNP-1, TNP-2, TNP-3, TNP-4, TNP-5, and TNP-6.

Laboratory Procedures

Petrographic examination of the concrete samples was performed in accordance with ASTM C 856, “Standard Practice for Petrographic Examination of Hardened Concrete.” Sections were polished and examined using a stereomicroscope at magnifications up to 350×. Small rectangular blocks were cut from concrete samples. One surface of each block was polished, dried, placed on petrographic slide with low–viscosity epoxy resin, and reduced to a thickness of approximately 25 micrometers (0.001 in.). These thin sections were examined using a polarized–light microscope at magnification up to 400×, to determine aggregate mineralogy, paste characteristics, and microstructure.

Two ¾-inch thick slabs were sawn from the concrete cores. The slabs were ground and polished to obtain a smooth and plane surface. They were examined using a stereomicroscope at magnifications up to 200×.

Findings

Twelve thin section samples taken from the received cores were examined using the polarized light microscope. Two polished sections were examined using the stereomicroscope. The findings are summarized as follows:

Aggregate

The coarse aggregate in the concrete is crushed stone, and the rocks are primarily limestone, with small amount of sandstone and dolomite. The shape of coarse aggregate particles is angular, and the maximum size is about 1/2 inch. Preferential orientation of coarse aggregate particles is not observed in this concrete, nor is segregation.

The fine aggregate fraction is mainly composed of quartz, with a small portion of chert, sandstone, and quartzite. The fine aggregate is from natural sand and the particles appear rounded to angular.

Cement Paste

The cement is reasonably hydrated and the paste contains some unhydrated cement particles as seen under the microscope (Figure M1-1).

Air Voids

Small, spherical air voids are observed in the concrete (Figure M1-2). However, the air content is estimated to be low.

Cracks

Isolated cracks in cement paste are sporadically observed in the concrete. Figure M1-3 shows a crack connecting two air voids, while the crack in Figure M1-4 spans the cement paste between two fine aggregate particles. In general, cracks do not propagate in aggregate particles (Figure M1-5), but they may find their existence in the aggregate/paste interface (Figure M1-6). These cracks appear isolated and short, and no cracking network is formed.

Paste/Aggregate Interface

In general, the paste/aggregate interface appears solid and dense, as shown in Figure M1-7.

Secondary Deposit

Small amount of ettringite was found sporadically in air voids in the concrete. Figure M1-8 shows the internal wall of a void is partially lined with ettringite crystals.

Summary

The concrete was air entrained, and the air content was low. Sporadic and isolated microcracks exist in cement paste, as well as in the aggregate/paste interface. Despite the defects in microscopical scale, the concrete appeared solid and sound.

Ettringite crystals form in some air voids. There was no evidence of deterioration related with the existence of the ettringite in the concrete. It is very common to see ettringite as secondary deposit in concrete.

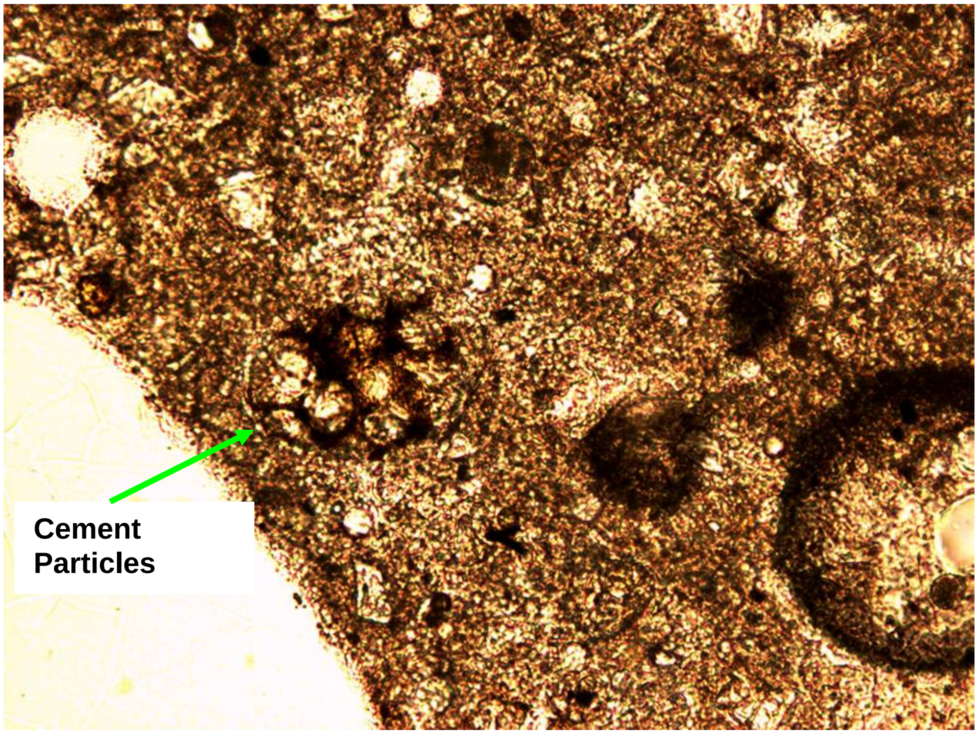


Figure M1-1: Unhydrated cement particles in paste. Width of field is 0.33 mm. Thin section image.

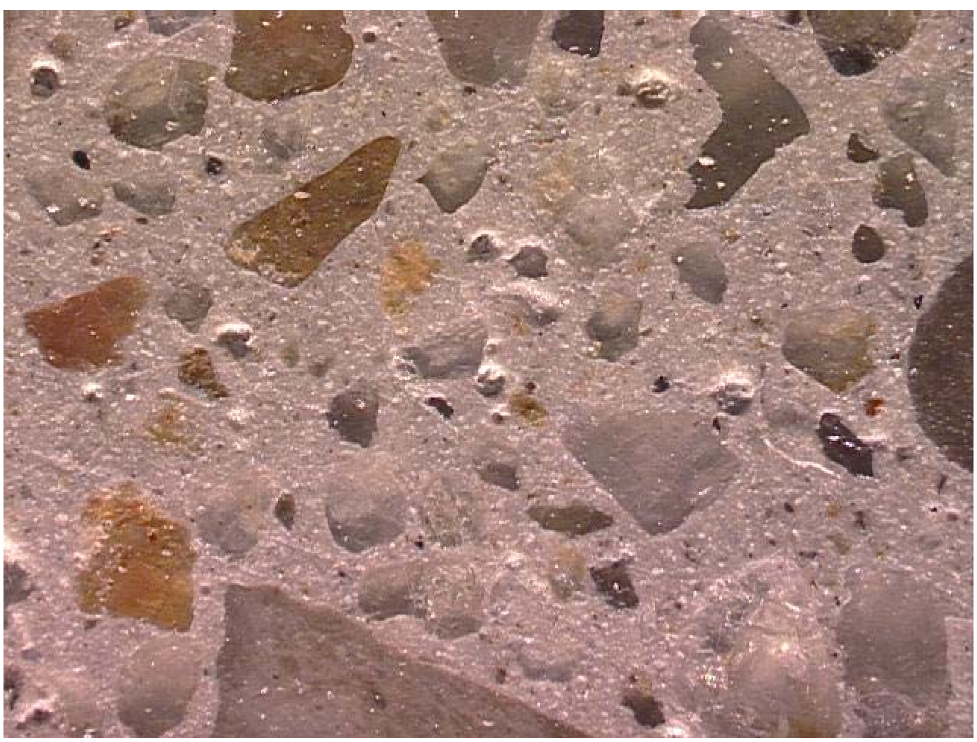


Figure M1-2: Air voids in the concrete. Width of field is 4.0 mm. Polished concrete surface image.

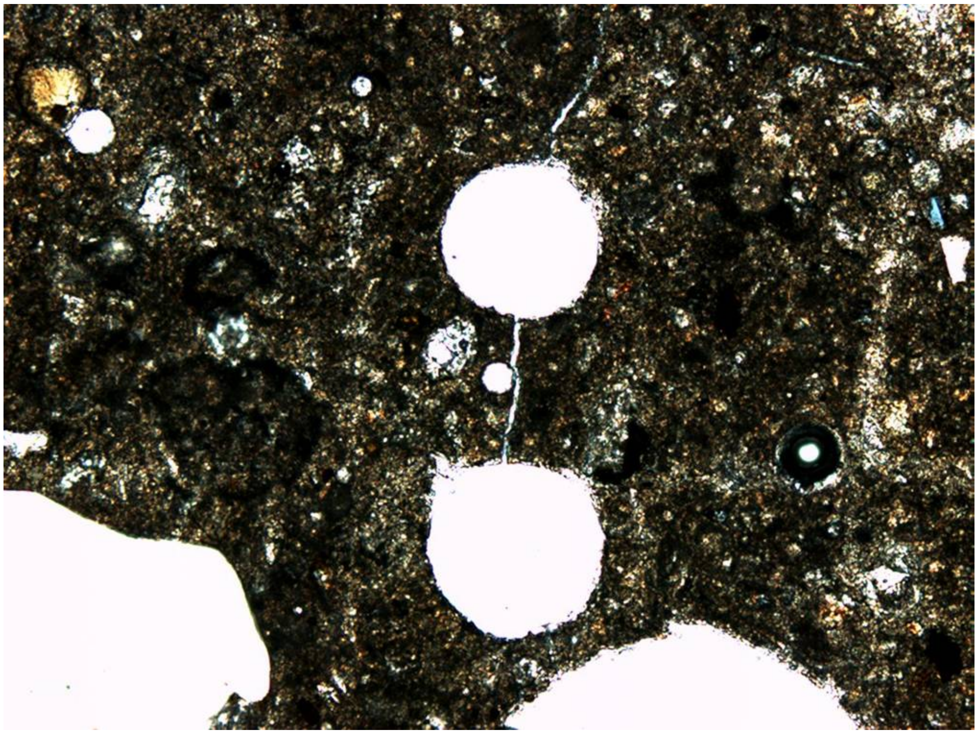


Figure M1-3: A crack connecting air voids. Width of field is 0.65 mm. Thin section image.

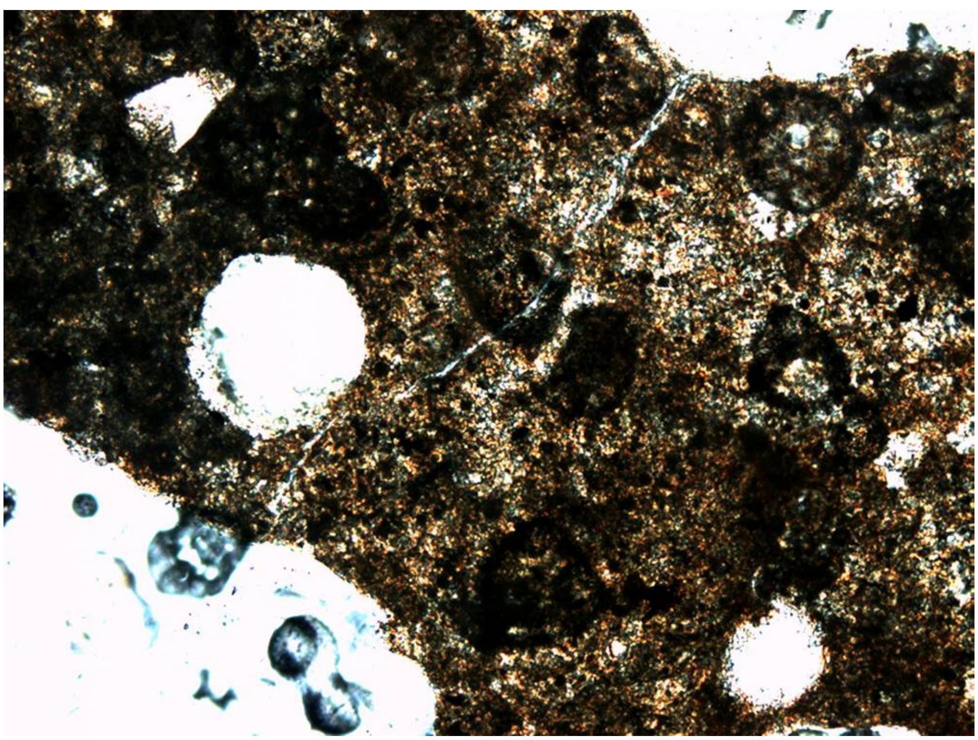


Figure M1-4: A crack connecting two fine aggregate particles. Width of field is 0.65 mm. Thin section image.

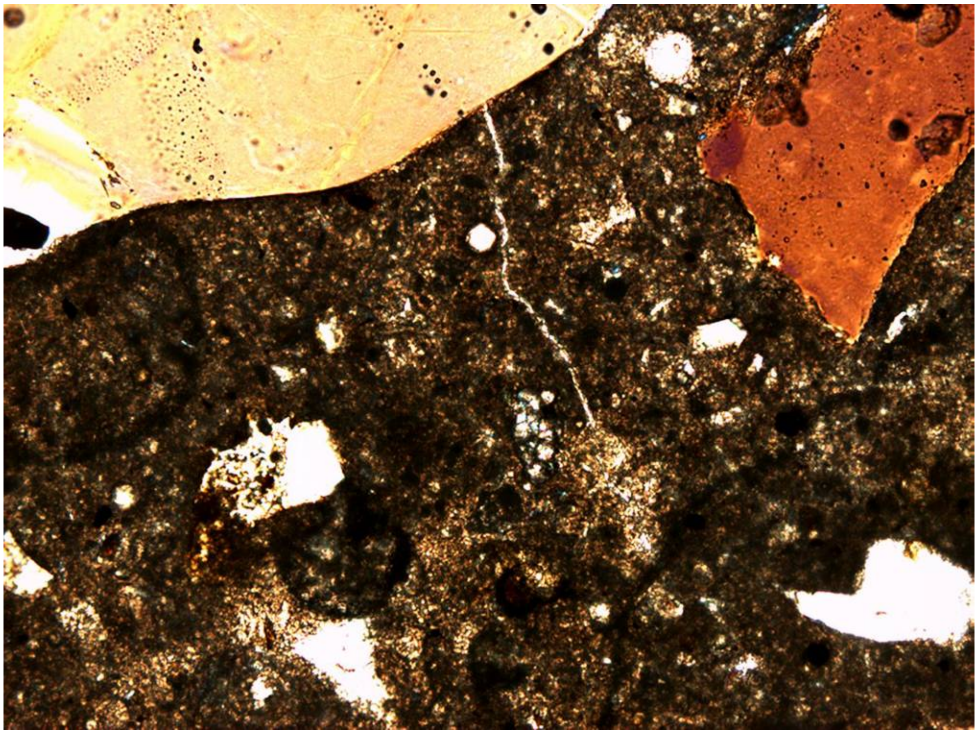


Figure M1-5: A crack close to a coarse aggregate particle. Width of field is 0.65 mm. Thin section image.

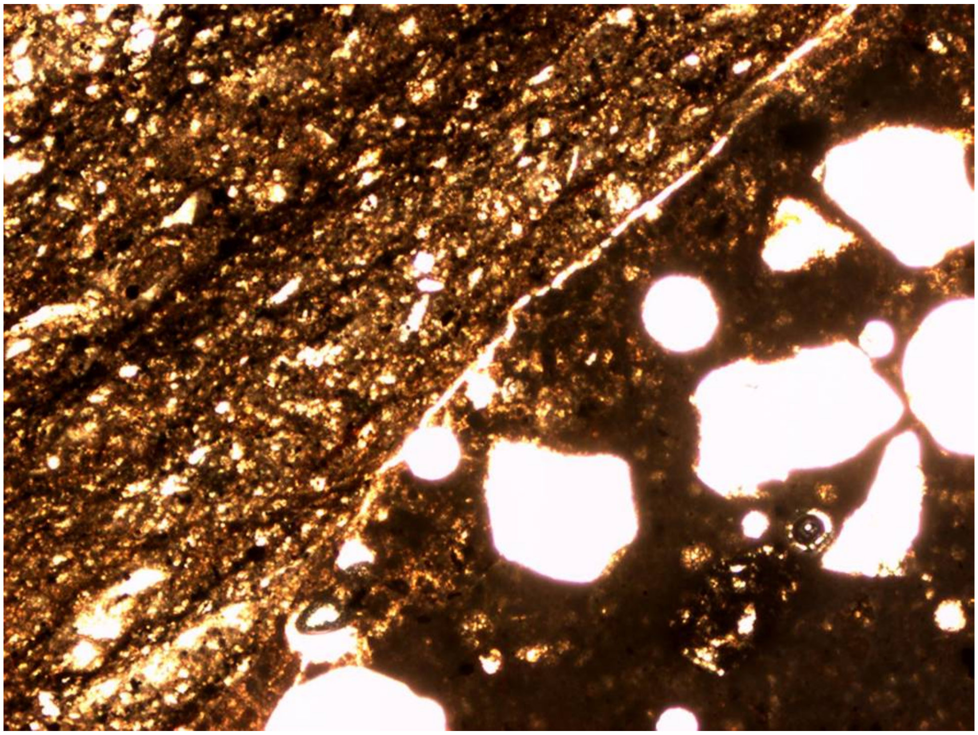


Figure M1-6: Crack in the coarse aggregate/paste interface. Width of field is 0.65 mm. Thin section image.

Figure M1-7: An image of aggregate/paste interface. Width of field is 2.0 mm. Polished surface image.

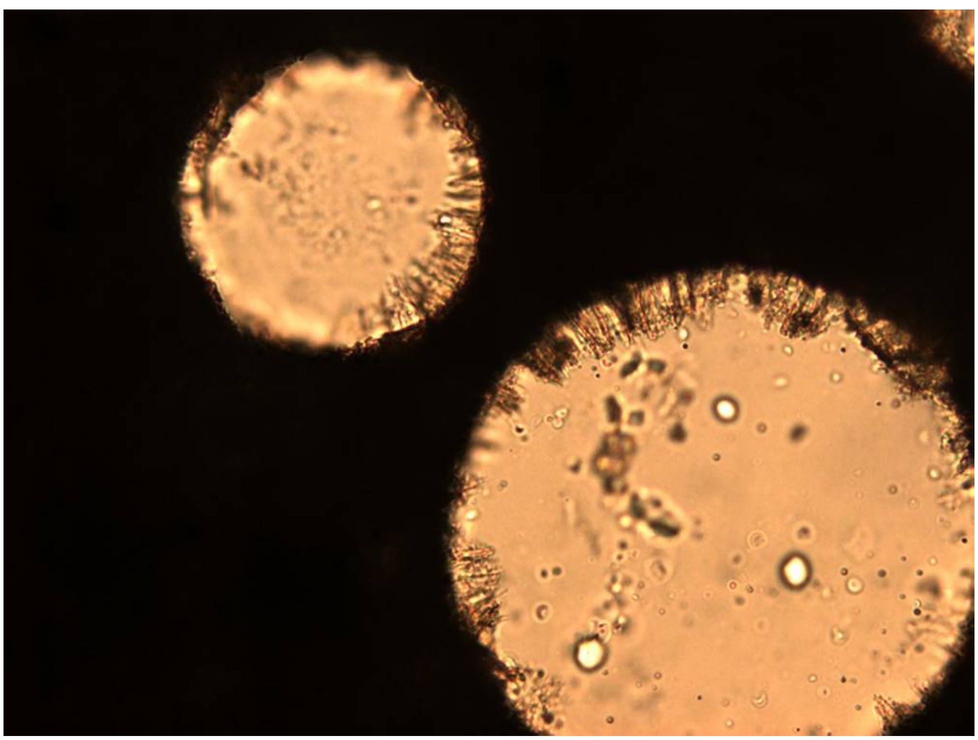


Figure M1-8: Ettringite crystals in air voids. Width of field is 0.33 mm. Thin section image.

APPENDIX M – Supplement 2

Porter Road, Tennessee
Survey Checklist

Checklist

The following checklist is adapted from 201.1 R-2, and provided to facilitate a thorough survey. The definition of terms and associated photographs in 201.1 R-2, are utilized to standardize the reporting of the condition of the concrete in the structures.

1. Description of structure or pavement
    - 1.2 Name, location, type, and size: Bridge 3, Porter Road over I-840, Dickson Co., Two Span Continuous Prestressed Girder, 320 Ft
    - 1.2 Owner, project engineer, contractor, when built
      TNDOT, Built 2000
    - 1.3 Design
        - 1.3.1 Architect and/or engineer
        - 1.3.2 Intended use and history of use
        - 1.3.3 Special features
    - 1.4 Construction ____________
        - 1.4.1 Contractor-general
        - 1.4.2 Subcontractors concrete placement
        - 1.4.3 Concrete supplier
        - 1.4.4 Agency responsible for testing
        - 1.4.5 Other subcontractors
    - 1.5 Photographs
        - 1.5.1 General view
          Yes
        - 1.5.2 Detailed close up of condition of area
          Yes
    - 1.18 Sketch map-orientation showing sunny and shady and well and poorly drained regions ____________

2. Present condition of structure — Date of Evaluation 10/1/02
    - 2.1 Overall alignment of structure — Good
        - 2.1.I Settlement — None
        - 2.1.2 Deflection — None
        - 2.1.3 Expansion — None
        - 2.1.4 Contraction — None
    - 2.2 Portions showing distress (beams, columns, pavement, walls, etc., subjected to strains and pressures) Parapets-Vert. HL,Deck-Map HL
    - 2.3 Surface condition of concrete
        - 2.3.1 General (good, satisfactory, poor, dusting, chalking, blisters
          Good
        - 2.3.2 Cracks — Diag/Map/Transverse
            - 2.3.2.1 Location and frequency — See Figure 2
            - 2.3.2.24 Type and size (see Definitions) — See Figure 2
              Longitudinal — N/A
              Width (from Crack comparator) ____ in.
              Hairline (Less than 1/32 in.)

Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)
Transverse Along Roadway CL
Width (from Crack comparator) 0.003-0.020 in.
Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)
Craze N/A
Width (from Crack comparator) ____ in.
Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)
Map Travel Lanes
Width (from Crack comparator) 0.003-0.010 in.
Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)
D-Cracking N/A
Width (from Crack comparator) ____ in.
Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)
Diagonal At Skew Ends
Width (from Crack comparator) 0.002-0.010 in.
Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)

2.3.2.25 Leaching, stalactites None/SIP Forms

2.3.3 Scaling None

2.3.3.1 Area, depth n/a

2.3.3.13 Type (see Definitions) n/a
Light (Less than 1/8 in.)
Medium (1/8 in. – 3/8 in.)
Severe (3/8 in. – 3/4 in.)
Very Severe (Greater than 3/4 in.)

2.3.4 Spalls and popouts None

2.3.4.1 Number, size, and depth n/a

2.3.4.13 Type (see Definitions) n/a
Spalls
Small (Less than 3/4 in. depth)

Large (Greater than 3/4 in. depth)
Popouts
Small (Less than 3/8 in. diameter)
Medium (3/8 in. – 2 in. diameter)
Large (Greater than 2 in. diameter)

2.3.5 Extent of corrosion or chemical attack, abrasion, impact, cavitation ___None___
2.3.6 Stains, efflorescence ___None___
2.3.7 Exposed reinforcement ___None___
2.3.8 Curling and warping ___None___
2.3.9 Previous patching or other repair ___None___
2.3.10 Surface coatings ___None___
- 2.3.10.1 Type and thickness ______
- 2.3.10.2 Bond to concrete ______
- 2.3.10.3 Condition ______

2.3.11 Abrasion ___Surface Milled___
2.3.12 Penetrating sealers ___None___
- 2.3.12.1 Type ______
- 2.3.12.2 Effectiveness ______
- 2.3.12.14 Discoloration ______

2.4 Interior condition of concrete (in situ and samples) ______
- 2.4.1 Strength of cores
- 2.4.2 Density of cores
- 2.4.3 Moisture content
- 2.4.4 Evidence of alkali-aggregate or other reactions ______
- 2.4.5 Bond to aggregate, reinforcing steel, joints ______
- 2.4.6 Pulse velocity
- 2.4.7 Volume change
- 2.4.8 Air content and distribution
- 2.4.9 Chloride-ion content
- 2.4.10 Cover over reinforcing steel
- 2.4.11 Half-cell potential to reinforcing steel.
- 2.4.12 Evidence of reinforcement corrosion
- 2.4.13 Evidence of corrosion of dissimilar metals
- 2.4.25 Delaminations ___None___
  - 2.4.25.1 Location ___n/a___
  - 2.4.25.2 Number, and size ___n/a___
- 2.4.15 Depth of carbonation
- 2.4.16 Freezing and thawing distress (frost damage)
- 2.4.17 Extent of deterioration
- 2.4.29 Aggregate proportioning, and distribution

3. Nature of loading and detrimental elements
- 3.1 Exposure
  - 3.1.1 Environment (arid, subtropical, marine, freshwater, industrial, etc.) ___agricultural / residential___
  - 3.1.2 Weather-(July and January mean temperatures, ___77.8/35.1___

mean annual rainfall and	54 in.
months in which 60 percent of it occurs)	Nov.-May (64%)
3.1.3 Freezing and thawing	Yes
3.1.4 Wetting and drying	Yes
3.1.10 Drying under dry atmosphere	No
3.1.6 Chemical attack-sulfates, acids, chloride	None
3.1.7 Abrasion, erosion, cavitation, impact	Milled
3.1.8 Electric currents	None
3.1.9 Deicing chemicals which contain chloride ions	unknown
3.1.10 Heat from adjacent sources	None
3.2 Drainage	
3.2.1 Flashing	None
3.2.2 Weepholes	Good
3.2.3 Contour	Good
3.2.4 Elevation of drains	Good
3.3 Loading	No signs of distress
3.3.1 Dead	
3.3.2 Live	
3.3.3 Impact	
3.3.4 Vibration	
3.3.5 Traffic index	
3.3.6 Other	
3.4 Soils (foundation conditions)	n/a
3.4.1 Compressibility	
3.4.2 Expansive soil	
3.4.3 Settlement	
3.4.4 Resistivity	
3.4.5 Evidence of pumping	
3.4.6 Water table (level and fluctuations)	
4. Original condition of structure	Good
4.1 Condition of formed and finished surfaces	Good
4.1.1 Smoothness	Fair
4.1.2 Air pockets ("bugholes")	Good
4.1.3 Sand streaks	Good
4.1.4 Honeycomb	Good
4.1.5 Soft areas (retarded hydration)	Good
4.1.6 Cold joints	Good
4.1.29 Staining	Good
4.1.30 Sand pockets	Fair
4.2 Defects	Cracking
4.2.1 Cracking	Fair
4.2.1.1 Plastic shrinkage	Fair
4.2.1.2 Thermal shrinkage	Fair
4.2.1.3 Drying shrinkage	Fair
4.2.13 Curling	None

5. Materials of Construction Good

6. Construction Practices Fair

APPENDIX N

Hickman Road, Tennessee

HIGH PERFORMANCE CONCRETE BRIDGE DECK INVESTIGATION

Hickman Road
Dickson County, Tennessee

I. BACKGROUND

Hickman Road Bridge over State Route 840 in Dickson County was constructed in 2000. The structure is 290 ft - 8 in. long and 32-ft wide (see photos 1 through 3, and Figure 1). It carries one eastbound lane and one westbound lane of Hickman Road. The structure consists of 8-¼-in. thick concrete deck with stay-in-place forms on 139 ft - 4 in. and 151 ft - 4 in. long continuous span concrete bulb-tee prestressed superstructure, on one concrete pier and two concrete abutments. The structure was built with a 17.5^{o} skew at both abutments and the pier. Four precast bulb-tee girders, BT-72, on 8 ft - 4 in. centers support each span. The concrete stub abutments are separated from the State Route 840 with loose riprap slope protection. The concrete pier is comprised of a cast-in-place concrete hammerhead cap on a cast-in-place pier stem.

The retaining wall, abutments, bent, girders and deck were constructed with high performance concrete (HPC). The factors that led to the use of HPC in this bridge included longer span length and a more durable structure. If HPC was not used, structural steel was the only other viable option, which would have resulted in an additional cost of about $500,000. The cost of HPC in this bridge was $160/ linear ft for the beams, $315/yd^3 for the deck and $240/yd^3 for the substructure concrete.

II. SCOPE OF SERVICES

Professional Service Industries Inc. (PSI) is under contract with the Federal Highway Administration (FHWA) to conduct bridge deck inspections for HPC bridges. Our scope of services on this bridge included a series of tasks and sub-tasks, which are described as follows:

- Collect all available information relevant to the construction of each bridge, including;
    - Deck Concrete Properties
    - Specified Deck Concrete Construction Procedures
    - Approved Concrete Mixture Proportions
    - Measured Properties from QC
    - Other Measured Properties
    - Actual Method of Deck Placement
    - Average Daily Traffic (ADT)
    - Exposure Condition of the Bridge
    - Any Performed Maintenance
    - Any Inspection Reports
- Visually inspect the bridge, and obtain the following:

- General condition of the deck top surface
- Determination of the maximum crack width
- General condition of the deck underside
- General condition of the girders
- Photograph areas of significant deterioration
- Prepare drawings locating defects
- Extract 6 concrete core samples

III. COMPILATION OF BRIDGE CONSTRUCTION INFORMATION

Information sources used for this report include bridge drawings, inspection reports, bridge summary data sheets, and technical information contained in FHWA's "Compilation and Evaluation of Results from High Performance Concrete Bridge Projects" version 3.

DECK CONCRETE PROPERTIES

The specified concrete properties for the bridge deck construction are listed in Table 1.

TABLE 1. Specified Concrete Properties for Deck Construction

Property	Deck
Minimum Cementitious Materials Content:	658 lb/yd^3
Max. Water/Cementitious Materials Ratio:	0.43
Min., Max. Percentage of Fly Ash:	—, —
Min., Max. Percentage of Silica Fume:	3, 8
Min., Max. Percentage of GGBFS:	—, —
Maximum Aggregate Size:	—
Slump:	3 ± 1 in
Air Content:	6 ± 2%
Compressive Strength (Design):	5000 psi at 28 days
Chloride Permeability (AASHTO T 277):	<1500 coulombs at 28 days
ASR or DEF Prevention:	—
Freeze-Thaw Resistance:	—
Deicer Scaling:	—
Abrasion Resistance:	—

NOTES:
(1) Specified slump is prior to addition of a high-range water-reducer if used. The maximum slump with a high-range water-reducer added shall be 8 in.
(2) One week moist cured at 73 ^{o}F followed by 3 weeks at 100 ^{o}F $\pm$ 10 ^{o}F.

SPECIFIED DECK CONCRETE CONSTRUCTION PROCEDURES

For the cast-in-place deck, a wet curing for 7 days was specified. Fogging was applied, and then it was followed by membrane curing, wet burlap, and vapor barrier.

Concrete cylinders, 6 x 12 inch in dimension and cured in fog room, were specified for quality control testing.

APPROVED CONCRETE MIX PROPORTIONS

The approved concrete mix proportions are shown in Table 2.

TABLE 2. Approved Mix Proportions

	Cast-In-Place Deck
Cement Brand:	Lonestar
Cement Type:	I
Cement Composition:	—
Cement Fineness:	—
Cement Quantity:	293 kg/m^3
GGBFS Brand:	—
GGBFS Quantity:	—
Fly Ash Brand:	Mineral Resources
Fly Ash Type:	C
Fly Ash Quantity:	91 kg/m^3
Silica Fume Brand:	Rheomac SF 100
Silica Fume Quantity:	30 kg/m^3
Fine Aggregate Type:	Sand
Fine Aggregate FM:	—
Fine Aggregate SG:	2.61
Fine Aggregate Quantity:	662 kg/m^3
Coarse Aggregate, Max. Size:	No. 57
Coarse Aggregate Type:	Crushed limestone
Coarse Aggregate SG:	2.63
Coarse Aggregate Quantity:	1074 kg/m^3
Other Aggregate, Max. Size:	—
Other Aggregate Type:	—
Other Aggregate SG:	—
Other Aggregate Quantity:	—
Water:	138 l/m^3
Water Reducer Brand:	Polyhead N
Water Reducer Type:	A
Water Reducer Quantity:	—
High-Range Water-Reducer Brand:	Rheobuild 1000
High-Range Water-Reducer Type:	A and F
High-Range Water-Reducer Quantity:	—
Retarder Brand:	—
Retarder Type:	—
Retarder Quantity:	—

Continued

TABLE 2 (continued). Approved Mix Proportions

	Cast-In-Place Deck
Corrosion Inhibitor Brand:	—
Corrosion Inhibitor Type:	—
Corrosion Inhibitor Quantity:	—
Air Entrainment Brand:	Micro-Air
Air Entrainment Type:	Surfactant
Air Entrainment Quantity:	—
Water/Cementitious Materials Ratio:	0.36

NOTES: Retarder to be added when ambient temperature is 75 ^oF or higher.

MEASURED PROPERTIES — QC TESTS

The measured properties from QC tests of the HPC production concrete used in the cast-in-place concrete are shown in Table 3. The cylinders were moist cured.

TABLE 3. Measured Properties from QC Testing

Batch No.	**Compressive Strength (1), psi**			
	7 days	**14 days**	**28 days**	**56 days**
1	5040	6220	6940	7740
	4810	6290	6780	7770
2	3880	5250	5950	6790
	3670	5110	5840	6780
3	4230	5570	7020	6950
	4090	5670	6230	7150
Average	4287	5685	6460	7197

(1) 6 x12 in. cylinders.

MEASURED PROPERTIES — RESEARCH TESTS OF PRODUCTION CONCRETE

The modulus of elasticity and splitting tensile strength were obtained from research tests of production concrete for the deck. These measured properties are shown in Table 4.

TABLE 4. Other Measured Properties

Batch No.	**Modulus of Elasticity (1), ksi**		**Splitting Tensile Strength (2), psi**	
	28 days	**56 days**	**28 days**	**56 days**
1	4866	5129	826	720
	4718	5284	797	737
2	4211	4511	647	693
	4099	4469	665	709
3	4415	4545	762	686
	4622	4600	797	658
Average	4489	4756	749	701

(1) ASTM C 469. 4 x 8-in moist cured cylinders.
(2) AASHTO T 198. 4 x 8-in moist cured cylinders.

The rapid chloride penetration test (AASHTO T 277) was performed on the concrete samples. The Coulomb values for each concrete batch and the curing conditions are shown in Table 5.

TABLE 5. Chloride Permeability (AASHTO T 277)

Batch No.	**Chloride Permeability [1], Coulombs**	
	28 days	**56 days**
1	289 [2]	237 [2]
	1311	550
2	356 [2]	311 [2]
	1658	832
3	306 [2]	259 [2]
	1407	739
Match Cure Edge	508 [2]	748
	3972	
Match Cure Center	660 [2]	1502
	4055	

[1] All specimens were moist cured except as noted.
[2] Cured in water at 100 °F.

ACTUAL METHOD OF DECK PLACEMENT

Construction of the deck occurred in May 2000, with the concrete for the deck pumped from a truck located below on the Route 840 alignment. Surface finishing consisted of motorized screed pan with a burlap drag. The deck was cured using water soaked burlap covered with plastic for seven days. Fogging with additional water through pressurized jets was also provided. The ambient temperature at the time of placement was 70°F. The milling operation and saw cutting the transverse grooves were performed in September 2001.

Average Daily Traffic (ADT) and Average Daily Truck Traffic (ADTT)

The district identifies this bridge as carrying 300 vehicles per day, with 3% trucks. While the PSI inspection crew was on-site only 20 cars were noted in 4 hours. No trucks were witnessed during this time period. This represents an ADT of approximately 120.

EXPOSURE CONDITION OF THE BRIDGE

The surrounding area is currently residential combined with agricultural land use. The National Weather Service reports that the mean July temperature is 77.8°F, while the mean January temperature is 35.1°F. The mean annual rainfall is 54 in., 64% of the annual amount falls from November to May. The bridge is exposed to freezing and thawing as well as wetting and drying on a seasonal cycle basis.

PERFORMED MAINTENANCE

No documentation of any maintenance performed since construction was found. Several patches were made at the time of construction, reportedly to repair the sand pocket inclusions due to the inadequate mixing of the silica fume.

INSPECTION REPORTS

This bridge, together with the other bridges along the new alignment of Route 840, is not currently entered into the inventory system.

IV. BRIDGE DECK INSPECTION

The bridge deck received a close visual inspection on October 1 through 3, 2002. The findings of this inspection are summarized as follows.

GENERAL CONDITION OF THE DECK TOP SURFACE

Defects in the top surface include transverse cracks, map cracks, diagonal corner cracks in the acute corners, patches, small sand pockets and an area of surface milling.

Transverse Cracks: Transverse cracks were found primarily along the centerline of the roadway. Figure 2 illustrates the 84 ft of transverse cracks that were identified on the surface of the deck (see Table 6). The crack widths ranged from 0.007 to 0.025 in. for the 10 cracks (see photo 4).

TABLE 6. Transverse Cracks

Distance from Abutment 1 (ft)	Width (in.)	Number	Length Range (ft)	Total Length of Cracks (ft)
5	0.009	1	6	6
115-135	0.009-0.025	8	6-10	73
285	0.007	1	5	5
Cumulative	0.007-0.025	10	5-10	84

Map Cracks: Map cracks were found primarily along the centerline and Eastbound roadway near the pier. Figure 2 illustrates the 480 ft^2 of map cracks that were identified on the surface of the deck (see Table 7). The crack widths ranged from 0.003 to 0.010 in. and the cracks were generally 8 in. by 8 in. apart (see photos 5 and 6).

TABLE 7. Map Cracks

Distance from Abutment 1 (ft)	Width (in.)	Area (ft^2)
110-125	0.003-0.009	180
135-160	0.005-0.010	300
Cumulative	0.003-0.010	480

Diagonal Cracks: Diagonal cracks were found primarily in the acute corners, SE and NW, of the bridge deck. Figure 2 illustrates the 26 ft of diagonal cracks that were identified on the surface of the deck (see Table 8). The crack widths ranged from 0.005 to 0.016 in. for the 6 cracks (see photo 7).

TABLE 8. Diagonal Corner Cracks

Distance from Abutment 1 (ft)	Width (in.)	Number	Length Range (ft)	Total Length of Cracks (ft)
0-5	0.009-0.016	2	4-7	11
285-290	0.005-0.016	4	2-6	15
Cumulative	0.005-0.016	6	2-7	26

In addition to the different types of cracking noted, a few isolated small defects were found. These defects included patches, sand pockets, inclusions, a small spall and a pattern of shallow embossing. Approximately 688 ft^2 of deck surface has been milled.

Patches: Patched areas ranging in size from 3-in. diameter to 10-in. square are located in the vicinity of the pier (see photo 8). Reportedly these patches were to repair sand pockets identified at the time of construction. The sand pockets appear to have been caused by incomplete mixing of the silica fume.

Sand Pockets: Six sand pockets were identified that ranged in size from 1 in. to 2 in. and 1-in. to 1-½-in. deep (see Table 9). The sand pockets appear to be the result of inadequate mixing of the silica fume at the time of construction, due to the gray coloration (see photo 9). Core TNH-7 was sampled at a pocket 150 ft from Abutment 1 (see photo 10).

TABLE 9. Sand Pockets

Distance from Abutment 1 (ft)	Length (in.)	Width (in.)	Depth (in.)	Number
18	1-½	2	1-½	1
138	1	1-½	1	1
145	1-½	2	1	2
150	1-½	1-½	1-½	1
155	1	1	1	1

Inclusions: Three inclusions were identified on the surface of the deck (see Table 10). Generally, these inclusions consisted of debris including foam board similar to styrofoam (see photo 11).

TABLE 10. Inclusions

Distance from Abutment 1 (ft)	Length (in.)	Width (in.)	Depth (in.)	Number
13	1	1-½	¾	1
195	¾	¾	½	1
245	2	2	¾	1

Spall: One 3-in. diameter spall, ½-in. deep was identified 18-ft west of the pier along the south shoulder, approximately 157 ft from Abutment 1.

Embossed Areas: Approximately 4 ft off of each rail, a series of shallow, ½ in., embosses occur, repeating on 4-ft centers (see photo 12). The imprints are similar in size and shape, approximately 3-in. diameter, and appear to be due to a rolling screed at the time of construction.

MAXIMUM CRACK WIDTH

The maximum width of transverse cracks was measured to be 0.025 in., while the maximum width of map cracks was measured to be 0.010 in. The maximum width of diagonal cracks was measured to be 0.016 in.

GENERAL CONDITION OF THE DECK UNDERSIDE

The underside of the deck was not visible due to the presence of stay-in-place deck forms. However, the exterior cantilever portions of the deck were exposed, which exhibit no signs of distress. Photos 13, 14 and 15 show general views of the underside of the deck.

GENERAL CONDITION OF THE GIRDERS

The girders were inspected from the ground, without the aide of any access equipment. No signs of distress were noted on any of the girders. Girders 1 (6A2) and 2 (6B2) in Span 2 are instrumented with both electrical strain gauges and a mechanical string line (see photo 16).

CONCRETE CORE SAMPLES

Seven cores were retrieved from the deck, and their locations are illustrated in Figure 1. The locations were selected to distribute the samples along each shoulder of the bridge, since Tennessee DOT had requested that the coring and patching operation avoid the traveled lanes. The cores were labeled TNH-1 through TNH-7 (see Table 11) and transferred to FHWA on January 7, 2003, for further investigation.

TABLE 11. Core Dimensions

Sample	**TNH-1**	**TNH-2**	**TNH-3**	**TNH-4**	**TNH-5**	**TNH-6**	**TNH-7**
Diameter (in.)	4	4	4	4	4	4	4
Length (in.)	3-¼	3-¼	2-¾	2-¾	3-½	3-½	3

Preliminary Conclusions

Hickman Road Bridge was constructed in May, 2000. The retaining wall, abutments, bent, girders and deck were constructed with high performance concrete (HPC). The concrete mix used Type I cement (293 kg/m^3), Type C fly ash (91 kg/m^3), and silica fume

(30 kg/m^3). The total amount of cementitious materials was 414 kg/m^3. The water to cementitious materials ratio was 0.36. High range water reducer was used in the concrete mix. The deck was cured using water soaked burlap covered with plastic for seven days. Fogging with additional water through pressurized jets was also provided.

The bridge deck received a visual inspection in October, 2002. Defects in the top surface included transverse cracks, map cracks, diagonal corner cracks in the acute corners, patches, small sand pockets and an area of surface milling.

Transverse cracks were primarily along the centerline of the roadway. A total of 10 transverse cracks were identified on the deck. The crack widths ranged from 0.007 to 0.025 in. Map cracks were primarily along the centerline and Eastbound roadway near the pier. The crack widths ranged from 0.003 to 0.010 in. and the cracks were generally in a form of 8 in. by 8 in. network. Diagonal cracks were primarily in the acute corners, SE and NW, of the bridge deck. The widths of the 6 diagonal cracks ranged from 0.005 to 0.016 in.

In addition to the different types of cracking noted, a few isolated small defects were found. These defects included patches, sand pockets, inclusions, a small spall and a pattern of shallow embossing. Six sand pockets ranged in size from 1 in. to 2 in. and 1-in. to 1-½-in. deep. The sand pockets appear to be the result of inadequate mixing of the silica fume at the time of construction, due to the gray coloration. Three inclusions were identified on the surface of the deck. Generally, these inclusions consisted of debris including foam board similar to styrofoam. The embossed areas were due to a rolling screed at the time of construction.

Petrographic examination was performed on seven concrete core samples that were retrieved from the shoulders of the bridge. The identification on the cores was: TNH-1, TNH-2, TNH-3, TNH-4, TNH-5, TNH-6, and TNH-7. The diameter of the cores was 4 in., and the lengths varied from about 3 to 4 in. All seven cores appeared intact, and visual inspection revealed no defects.

The coarse aggregate in the concrete was crushed stone. It was primarily composed of limestone, with small amount of sandstone and dolomite. The aggregate particles were angular, and the maximum size was about 3/4 inch. Preferential orientation of coarse aggregate particles was not observed in this concrete, nor was segregation. The fine aggregate fraction as mainly composed of quartz, with a small portion of chert, sandstone, and quartzite. The fine aggregate was from natural sand and the particles appeared rounded to angular.

The cement was reasonably hydrated with respect to the age of the concrete. The cement paste contained some unhydrated cement. Fly ash particles were also present in the concrete.

The concrete was air entrained. Small, spherical air voids were present in the concrete. The entrained air voids were well dispersed in the concrete. No entrapped air voids were found in the concrete samples that were examined.

Sporadic and isolated micro-cracks exist in cement paste, as well as in the fine aggregate/paste interface. These cracks appeared to be isolated and short, and no cracking network was formed. Despite the defects in microscopical scale, the concrete appeared solid and sound.

Respectfully Submitted,
Professional Service Industries, Inc.
Structural Investigation & Petrography Department

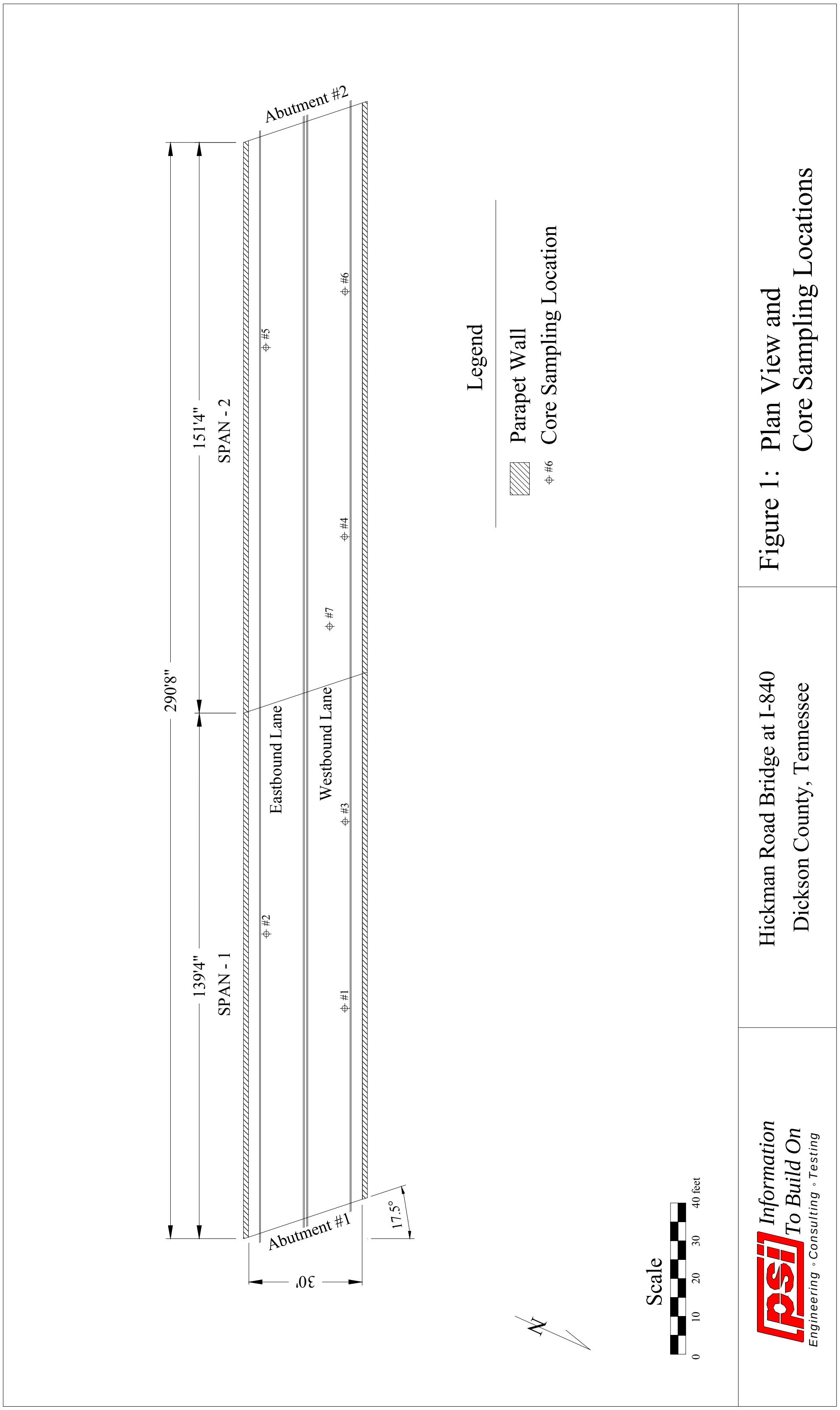


Figure 1: Plan View and Core Sampling Locations

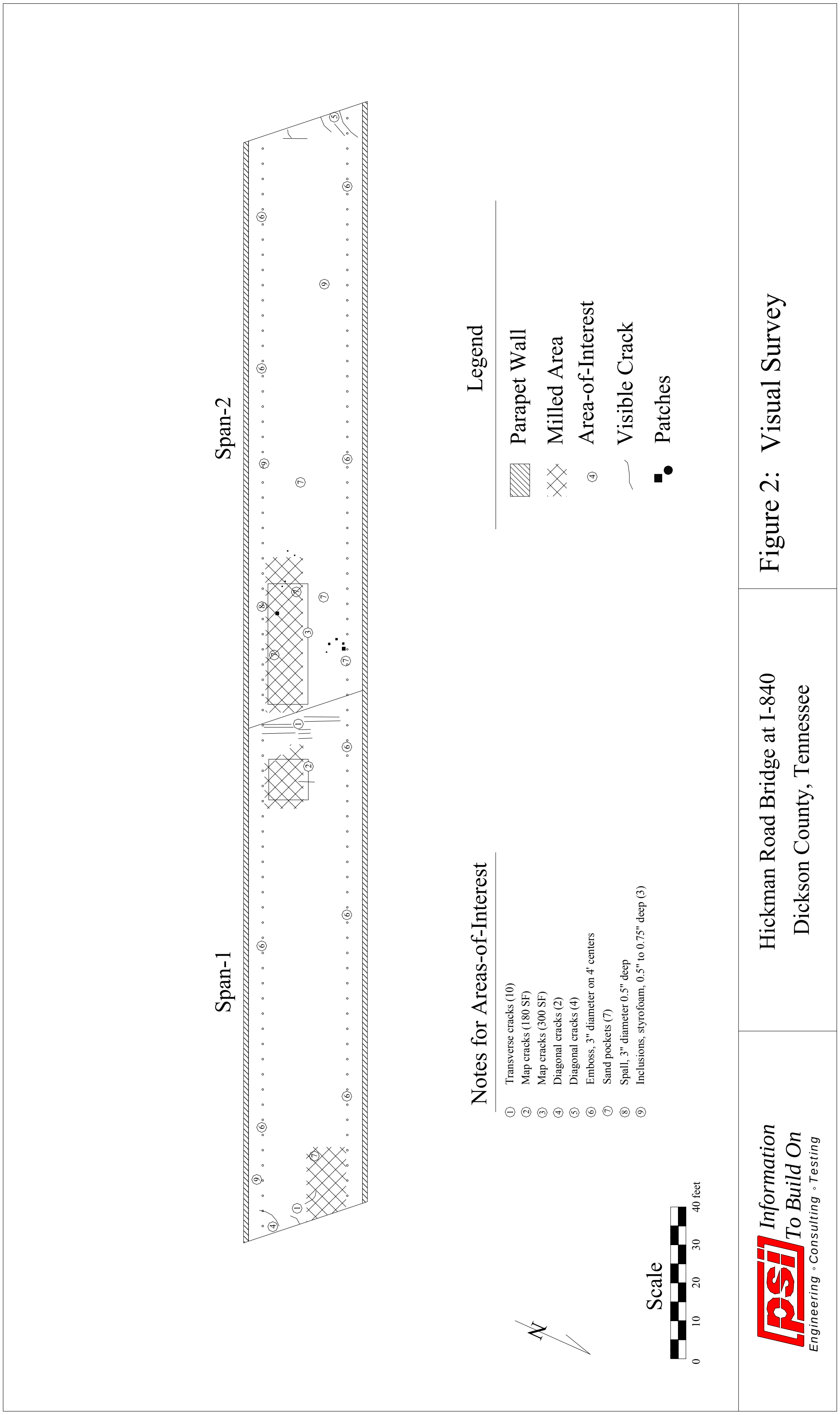


Figure 2: Visual Survey

Photographic Documentation

	Photo 1: General view of the north elevation.
	Photo 2: General view of the east approach.
	Photo 3: General view of the west approach.

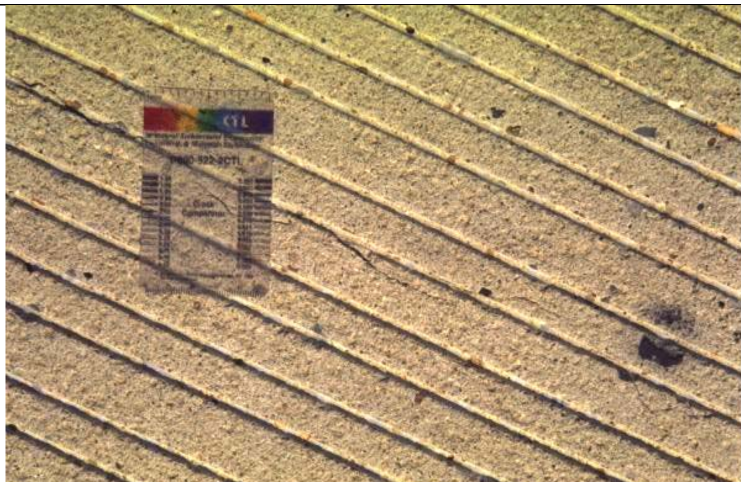


Photo 4: Typical transverse crack, 0.020 in. width.

Photo 5: General view of the typical map cracking.

Photo 6: Close-up view of the typical map cracking. Crack width 0.010 in.

	Photo 7: General view of the typical diagonal crack.
	Photo 8: Typical patched area.
	Photo 9: Typical sand pocket.

	Photo 10: Typical sand pocket at core sample TNH-7.
	Photo 11: Typical inclusion.
	Photo 12: Typical embossed area.

	Photo 13: Underside of the deck. Note the stay-in-place forms.
	Photo 14: Underside of the deck. Note the connections for instrumentation.
	Photo 15: Underside of the deck. Note the connections for instrumentation.

Photo 16: Underside of the deck. Note the instrumentation at abutment 2, girder 1 (exterior) left.

APPENDIX N – Supplement 1

Hickman Road, Tennessee
Petrographic Analysis

PETROGRAPHIC EXAMINATION OF SEVEN CONCRETE CORES FROM TENNESSEE (TNH)

A report submitted to the Structures Laboratory at TFHRC

Rongtang Liu
Petrographic Laboratory
FHWA TFHRC

March 29, 2005

1. Abstract

Seven concrete cores were received from the Structures Laboratory of the Turner-Fairbank Highway Research Center for petrographic examination. Reportedly, the concrete cores were collected from a concrete bridge in Tennessee.

Petrographic examination was performed on samples using optical microscopes. Thin sections, as well as polished slabs, were made from the concrete cores for microscopic examination.

The concrete appears sound and solid. Visual inspection of the concrete cores revealed no defects. The findings from microscopic examination indicate that the concrete has entrained air voids. The hydration of the cement was reasonable. The presence of unhydrated cement and fly ash particles was also observed in the cement paste. Sporadic and isolated microcracks were present in the paste, as well as in the paste/fine aggregate interfacial region.

2. Introduction

The Petrographic Laboratory of the FHWA Turner-Fairbank Highway Research Center was asked by the Structures Laboratory to examine a set of concrete cores retrieved from a bridge in Tennessee. Seven concrete cores of 4-in. diameter, 3- to 4-in. long were received by the Petrographic Laboratory. The identification on the cores was as follows: TNH-1, TNH-2, TNH-3, TNH-4, TNH-5, TNH-6, and TNH-7.

3. Laboratory Procedures

Petrographic examination of the concrete samples was performed in accordance with ASTM C 856, “Standard Practice for Petrographic Examination of Hardened Concrete.” Sections were polished and examined using a stereomicroscope at magnifications up to 350×. Small rectangular blocks were cut from concrete samples. One surface of each block was polished, dried, placed on petrographic slide with low−viscosity epoxy resin,

and reduced to a thickness of approximately 25 micrometers (0.001 in.). These thin sections were examined using a polarized–light microscope at magnification up to 400×, to determine aggregate mineralogy, paste characteristics, and microstructure.

Two ¾-inch thick slabs were sawn from the concrete cores. The slabs were ground and polished to obtain a smooth and plane surface. They were examined using a stereomicroscope at magnifications up to 200×.

4. Findings

Thirteen thin section samples taken from the received cores were examined using the polarized light microscope. Two polished sections were examined using the stereomicroscope. The findings are summarized as follows:

Aggregate

The coarse aggregate in the concrete is crushed stone, and the rocks are primarily limestone, with small amount of sandstone and dolomite. The shape of coarse aggregate particles is angular, and the maximum size is about 3/4 inch. Preferential orientation of coarse aggregate particles is not observed in this concrete, nor is segregation.

The fine aggregate fraction is mainly composed of quartz, with a small portion of chert, sandstone, and quartzite. The fine aggregate is from natural sand and the particles appear rounded to angular.

Cement Paste

The cement is reasonably hydrated and the paste contains some unhydrated cement particles as seen under the microscope (Figure N1-1). Fly ash particles are also present in the concrete, as shown in Figure N1-2.

Air Voids

Small, spherical air voids are observed in the concrete (Figure N1-3), hence the concrete was air entrained.

Cracks

Isolated cracks in cement paste are sporadically observed in the concrete. Figure N1-4 shows a crack connecting air voids, while the crack in Figure N1-5 spans the cement paste between two fine aggregate particles. Similar cracks also exist in the fine aggregate/paste interface (Figure N1-6). These cracks appear isolated and short, and no cracking network is formed.

Paste/Aggregate Interface

In general, the paste/aggregate interface is solid and dense, as shown in Figure N1-7 and Figure N1-8.

5. Summary

The concrete was air entrained, and the entrained air voids were well distributed in the concrete. Sporadic and isolated microcracks exist in cement paste, as well as in the fine aggregate/paste interface. Despite the defects in microscopical scale, the concrete appeared solid and sound.

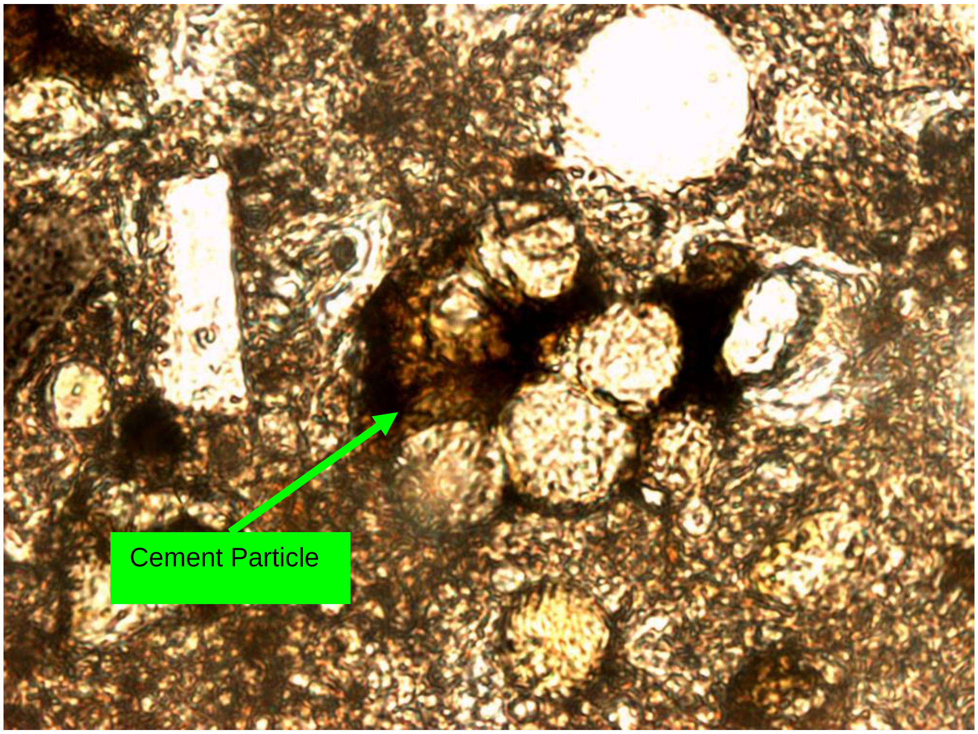


Figure N1-1: Unhydrated cement particles in paste. Width of field is 0.165 mm. Thin section image.

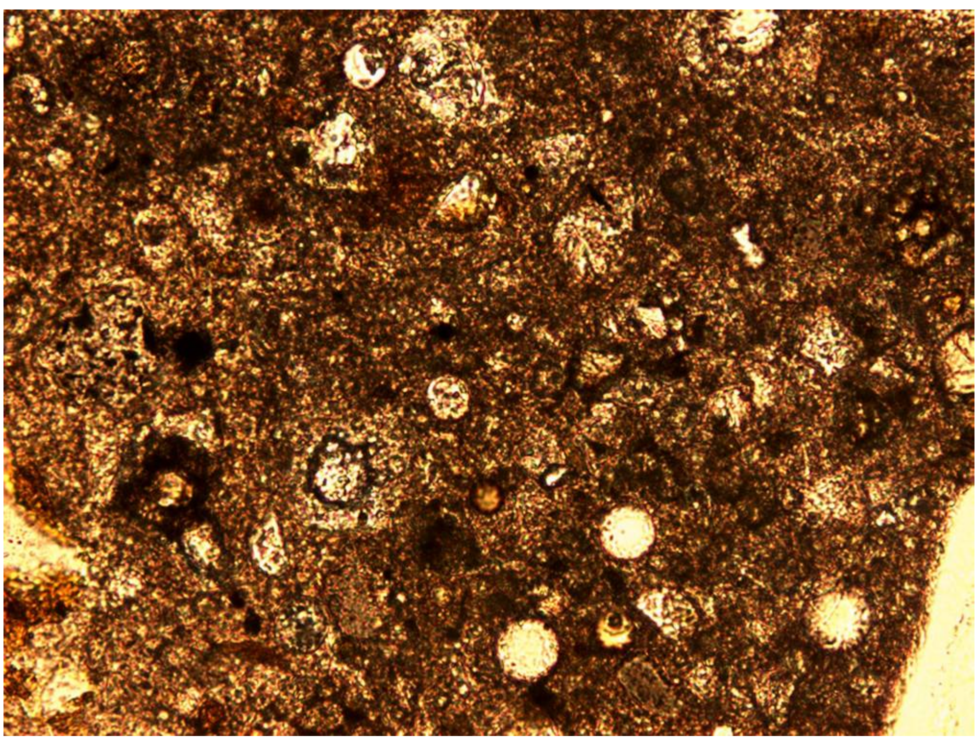


Figure N1-2: Fly ash particles in the concrete. Width of field is 0.33 mm. Thin section image.

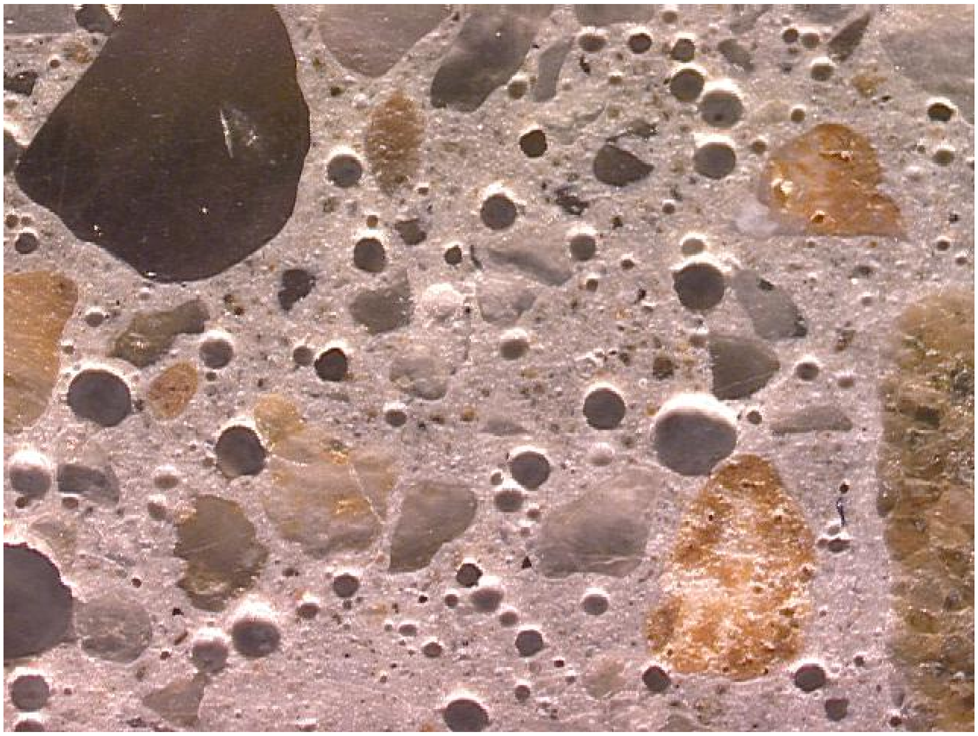


Figure N1-3: Air voids in the concrete. Width of field is 4.0 mm. Polished concrete surface image.

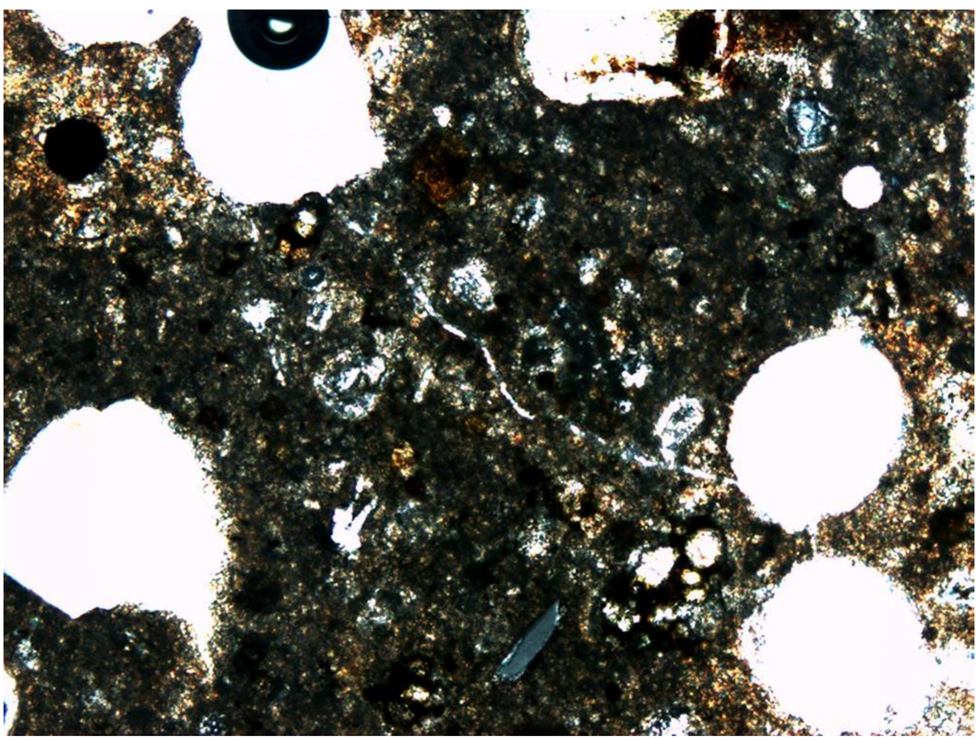


Figure N1-4: A crack connecting air voids. Width of field is 0.65 mm. Thin section image.

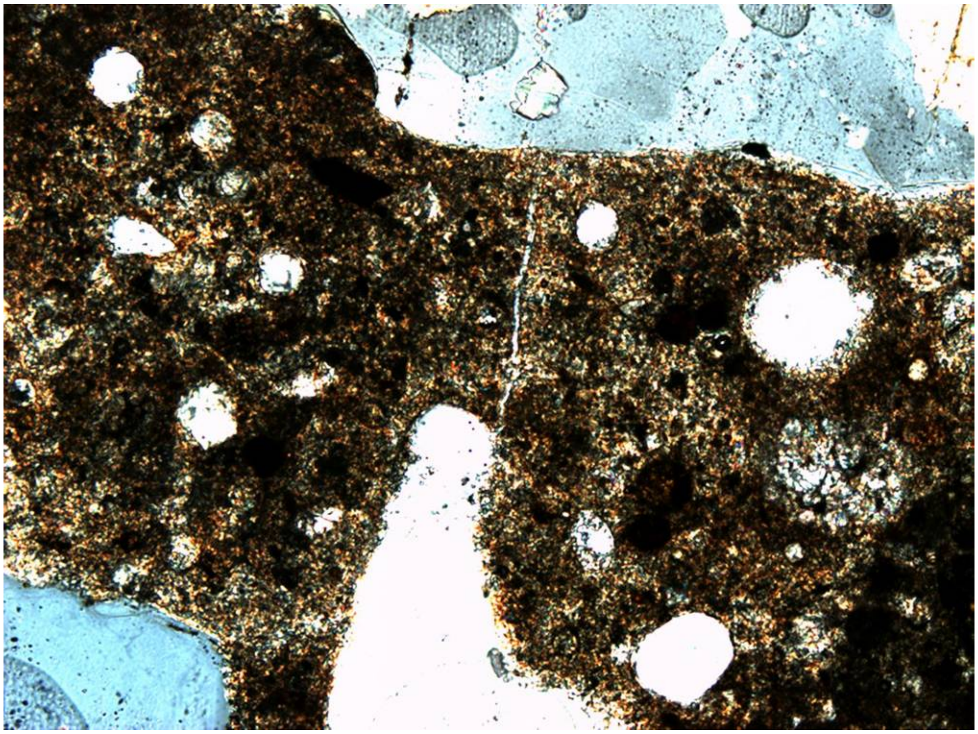


Figure N1-5: A crack connection two fine aggregate particles. Width of field is 0.65 mm. Thin section image.

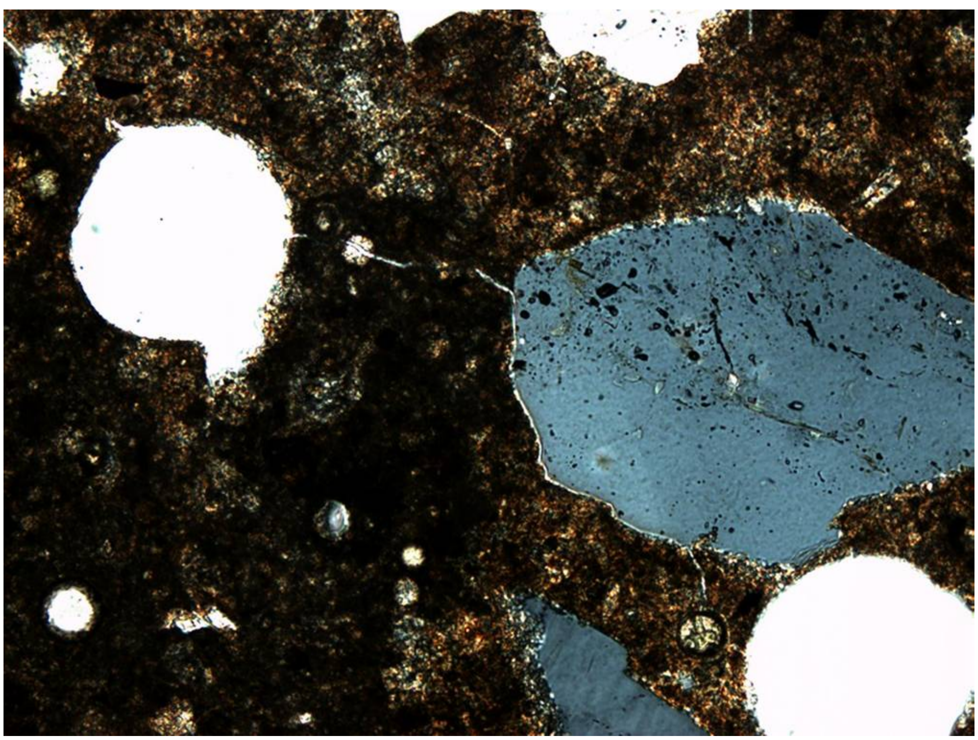


Figure N1-6: Cracking in the fine aggregate/paste interface. Width of field is 0.65 mm. Thin section image.

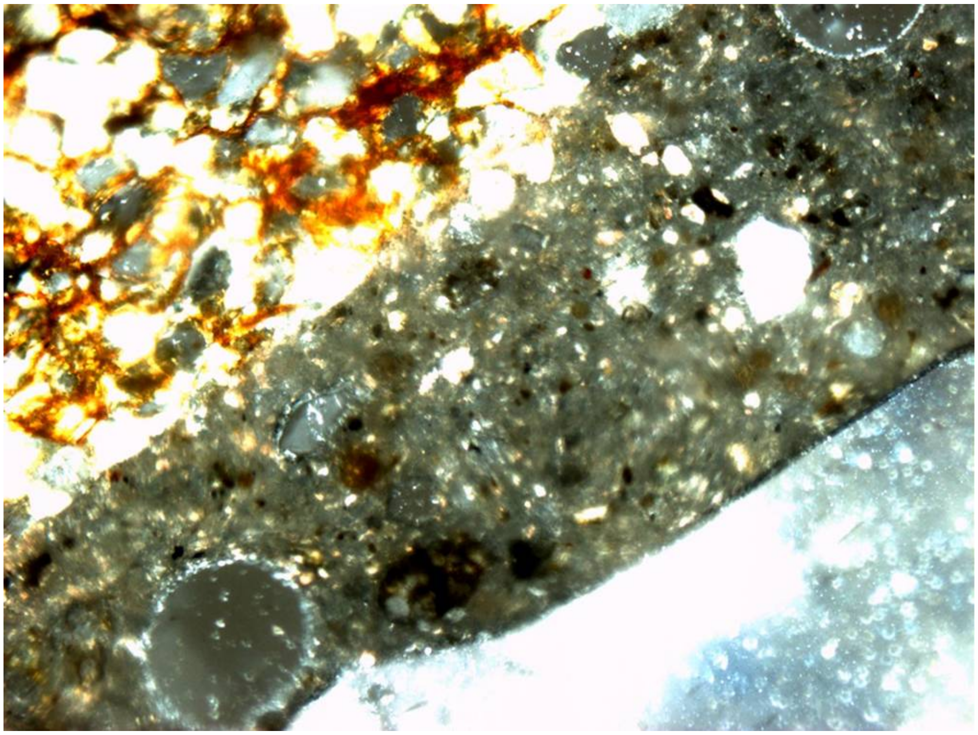


Figure N1-7: Coarse aggregate/paste interface. Width of field is 0.65 mm. Thin section image.

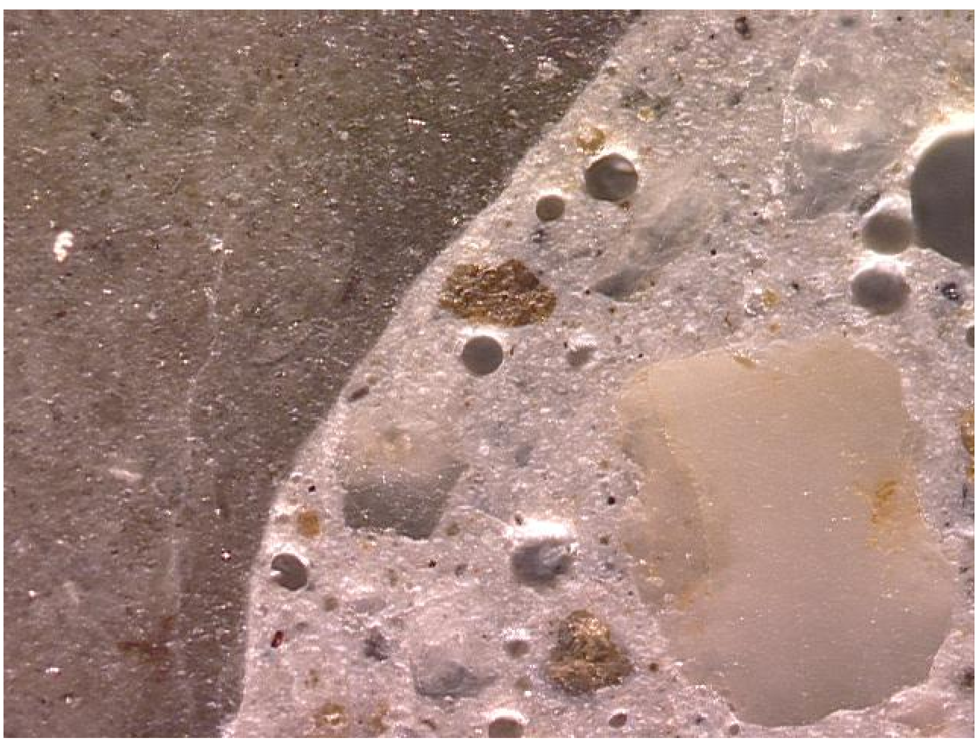


Figure N1-8: Another image of aggregate/paste interface. Width of field is 2.0 mm. Polished surface image.

APPENDIX N – Supplement 2

Hickman Road, Tennessee
Survey Checklist

Checklist

The following checklist is adapted from 201.1 R-2, and provided to facilitate a thorough survey. The definition of terms and associated photographs in 201.1 R-2, are utilized to standardize the reporting of the condition of the concrete in the structures.

1. Description of structure or pavement
   - 1.3 Name, location, type, and size
     Bridge 6, Hickman Road over I-840, Dickson Co.
     Two Span Continuous Prestressed Girder, 290 Ft
   - 1.2 Owner, project engineer, contractor, when built
     TNDOT, Built 2000
   - 1.3 Design
     - 1.3.1 Architect and/or engineer
     - 1.3.2 Intended use and history of use
     - 1.3.3 Special features
   - 1.4 Construction ________
     - 1.4.1 Contractor-general
     - 1.4.2 Subcontractors concrete placement
     - 1.4.3 Concrete supplier
     - 1.4.4 Agency responsible for testing
     - 1.4.5 Other subcontractors
   - 1.5 Photographs
     - 1.5.1 General view: Yes
     - 1.5.2 Detailed close up of condition of area: Yes
   - 1.19 Sketch map-orientation showing sunny and shady and well and poorly drained regions ________

2. Present condition of structure — Date of Evaluation: 10/2/02
   - 2.1 Overall alignment of structure: Good
     - 2.1.I Settlement: None
     - 2.1.2 Deflection: None
     - 2.1.3 Expansion: None
     - 2.1.4 Contraction: None
   - 2.2 Portions showing distress (beams, columns, pavement, walls, etc., subjected to strains and pressures): Parapets-Vert. hairline; Deck-Map hairline
   - 2.3 Surface condition of concrete
     - 2.3.1 General (good, satisfactory, poor, dusting, chalking, blisters): Good
     - 2.3.2 Cracks: Transverse/Map/Diag
       - 2.3.2.1 Location and frequency ________
       - 2.3.2.26 Type and size (see Definitions) ________
         - Longitudinal: N/A
         - Width (from Crack comparator): ____ in.

			Hairline	(Less than 1/32 in.)
			Fine	(1/32 in. - 1/16 in.)
			Medium	(1/16 – 1/8 in.)
			Wide	(Greater than 1/8 in.)
		Transverse		Along Roadway CL
		Width (from Crack comparator)		0.007-0.025 in.
			Hairline	(Less than 1/32 in.)
			Fine	(1/32 in. - 1/16 in.)
			Medium	(1/16 – 1/8 in.)
			Wide	(Greater than 1/8 in.)
		Craze		N/A
		Width (from Crack comparator)		in.
			Hairline	(Less than 1/32 in.)
			Fine	(1/32 in. - 1/16 in.)
			Medium	(1/16 – 1/8 in.)
			Wide	(Greater than 1/8 in.)
		Map		Travel Lanes
		Width (from Crack comparator)		0.003-0.010 in.
			Hairline	(Less than 1/32 in.)
			Fine	(1/32 in. - 1/16 in.)
			Medium	(1/16 – 1/8 in.)
			Wide	(Greater than 1/8 in.)
		D-Cracking		N/A
		Width (from Crack comparator)		in.
			Hairline	(Less than 1/32 in.)
			Fine	(1/32 in. - 1/16 in.)
			Medium	(1/16 – 1/8 in.)
			Wide	(Greater than 1/8 in.)
		Diagonal		At Skew Ends
		Width (from Crack comparator)		0.005-0.016 in.
			Hairline	(Less than 1/32 in.)
			Fine	(1/32 in. - 1/16 in.)
			Medium	(1/16 – 1/8 in.)
			Wide	(Greater than 1/8 in.)
	2.3.2.27	Leaching, stalactites		None/SIP Forms
2.3.3	Scaling			None
	2.3.3.1	Area, depth		n/a
	2.3.3.14	Type (see Definitions)		n/a
			Light	(Less than 1/8 in.)
			Medium	(1/8 in. – 3/8 in.)
			Severe	(3/8 in. – 3/4 in.)
			Very Severe	(Greater than 3/4 in.)
2.3.4	Spalls and popouts			Minor Spall
	2.3.4.1	Number, size, and depth		1) 3 in. diam. x ½ in.
	2.3.4.14	Type (see Definitions)		small
		Spalls		

Small (Less than 3/4 in. depth)
Large (Greater than 3/4 in. depth)
Popouts
Small (Less than 3/8 in. diameter)
Medium (3/8 in. – 2 in. diameter)
Large (Greater than 2 in. diameter)

2.3.5 Extent of corrosion or chemical attack, abrasion, impact, cavitation None
2.3.6 Stains, efflorescence None
2.3.7 Exposed reinforcement None
2.3.8 Curling and warping None
2.3.9 Previous patching or other repair Yes, 10) varying from 3 in. diam. to 10 in. square, located over pier
2.3.10 Surface coatings None
2.3.10.1 Type and thickness ______
2.3.10.2 Bond to concrete ______
2.3.10.3 Condition ______
2.3.11 Abrasion Surface Milled
2.3.12 Penetrating sealers None
2.3.12.1 Type ______
2.3.12.2 Effectiveness ______
2.3.12.15 Discoloration ______

2.4 Interior condition of concrete (in situ and samples) ______
2.4.1 Strength of cores
2.4.2 Density of cores
2.4.3 Moisture content
2.4.4 Evidence of alkali-aggregate or other reactions ______
2.4.5 Bond to aggregate, reinforcing steel, joints ______
2.4.6 Pulse velocity
2.4.7 Volume change
2.4.8 Air content and distribution
2.4.9 Chloride-ion content
2.4.10 Cover over reinforcing steel
2.4.11 Half-cell potential to reinforcing steel.
2.4.12 Evidence of reinforcement corrosion
2.4.13 Evidence of corrosion of dissimilar metals
2.4.26 Delaminations Small, Generally associated w/ inclusions
2.4.26.1 Location Span 2
2.4.26.2 Number, and size 3 in. diam.
2.4.15 Depth of carbonation
2.4.16 Freezing and thawing distress (frost damage)
2.4.17 Extent of deterioration
2.4.30 Aggregate proportioning, and distribution

3. Nature of loading and detrimental elements
3.1 Exposure

3.1.1	Environment (arid, subtropical, marine, freshwater, industrial, etc.)	agricultural / residential
3.1.2	Weather-(July and January mean temperatures,	77.8/35.1
	mean annual rainfall and	54 in.
	months in which 60 percent of it occurs)	Nov.-May (64%)
3.1.3	Freezing and thawing	Yes
3.1.4	Wetting and drying	Yes
3.1.11	Drying under dry atmosphere	No
3.1.6	Chemical attack-sulfates, acids, chloride	None
3.1.7	Abrasion, erosion, cavitation, impact	Milled
3.1.8	Electric currents	None
3.1.9	Deicing chemicals which contain chloride ions	Unknown
3.1.10	Heat from adjacent sources	None
3.2	Drainage	
3.2.1	Flashing	None
3.2.2	Weepholes	Good
3.2.3	Contour	Good
3.2.4	Elevation of drains	Good
3.3	Loading	No signs of distress
3.3.1	Dead	
3.3.2	Live	
3.3.3	Impact	
3.3.4	Vibration	
3.3.5	Traffic index	
3.3.6	Other	
3.4	Soils (foundation conditions)	n/a
3.4.1	Compressibility	
3.4.2	Expansive soil	
3.4.3	Settlement	
3.4.4	Resistivity	
3.4.5	Evidence of pumping	
3.4.6	Water table (level and fluctuations)	
4.	Original condition of structure	Good
4.1	Condition of formed and finished surfaces	Good
4.1.1	Smoothness	Fair
4.1.2	Air pockets ("bugholes")	Good
4.1.3	Sand streaks	Good
4.1.4	Honeycomb	Good
4.1.5	Soft areas (retarded hydration)	Good
4.1.6	Cold joints	Good
4.1.31	Staining	Good
4.1.32	Sand pockets	Fair
4.2	Defects	Cracking
4.2.1	Cracking	Fair
4.2.1.1	Plastic shrinkage	Fair

4.2.1.2	Thermal shrinkage	Fair
4.2.1.3	Drying shrinkage	Fair
4.2.14	Curling	None
5.	Materials of Construction	Good
6.	Construction Practices	Fair

APPENDIX O

SH 249 over Louetta Road, Houston, Texas

HIGH PERFORMANCE CONCRETE BRIDGE DECK INVESTIGATION

State Highway 249 (Tomball Parkway) over Louetta Road Houston, Texas

I. BACKGROUND

The Tomball Parkway (S.H. 249) Bridge over Louetta Road in Houston, Texas consists of two separate bridges, one carrying three lanes of the northbound traffic and the other carrying three lanes of the southbound traffic with an additional exit ramp (see photos 1 through 3). Both bridges consist of precast U-beam girders covered with precast concrete deck panels (3.5-inches thick × 8-feet long), which are in turn covered with 3.75-inches of cast-in-place concrete (see photo 4). The substructures consist of concrete columns and concrete abutments at each end.

The Tomball Parkway Bridge is a major structure carrying heavy traffic. It is 391-feet long and consists of three spans in each direction. Span 1, Span 2 and Span 3 have approximate lengths of 121.5 ft, 135.5 ft, and 134.0 ft, respectively. The width of the bridge is variable and varies from 160 ft at the ramp to 120 ft in the middle. The bridge has a skew of 33° to 39°. Each span in the northbound bridge consists of five Texas U54 beams, and each span in the southbound bridge consists of six Texas U54 beams. Beams are prestressed. The specified compressive strength of the girders at release of prestressing and 56 days ranged from 6,900 to 8,800 psi and 9,800 to 13,100 psi, respectively. At the interior bents, each beam is supported by a single post-tensioned pier.

The bridge was designed in accordance with the AASHTO Standard Specifications for Highway Bridges (1992) using HS 20-44 design live load and an E_c value of 6,000 ksi. Camber at release was specified to be in the range of 4.06 in. to 5.64 in. The concrete cover was specified to be 1 in. and would apply to the stirrups in the girders.

All beams, piers, and precast deck panels were fabricated using high performance / high strength concrete. For comparison purposes, the southbound main-lane bridge has a high performance / high strength cast-in-place deck, whereas the northbound main-lane bridge has a high performance / normal strength cast-in-place concrete deck. The precast deck panels were prestressed utilizing 3/8-in. diameter strands. The cast-in-place concrete overlay was reinforced with #5 bars at a spacing of 6 in. center-to-center in the transverse direction and #4 bars at a spacing of 12-in. center-to-center in the longitudinal direction. The rebar used in the cast-in-place deck was Grade 60 and uncoated. The concrete cover over #5 transverse reinforcing bars in the cast-in-place deck was specified as 2 in. The concrete cover below the 3/8-in. diameter strands in the precast deck panels was specified as 1-¾-in. The deck of each bridge was constructed simultaneously using similar construction techniques by the same personnel.

The Tomball Parkway (S.H. 249) Bridge over Louetta Road was the first of a series of demonstration projects for utilizing HPC in bridge structures, which were co-sponsored by the Federal Highway Administration (FHWA) and the Texas Department of Transportation (TxDOT). The construction of the bridge decks started in October 1996 and it was opened to traffic in both directions in June 1998.

II. SCOPE OF SERVICES

Professional Service Industries Inc. (PSI) is under contract with the Federal Highway Administration (FHWA) to conduct bridge deck inspections for HPC bridges. Our scope of services on this bridge included a series of tasks and sub-tasks, which are described as follows:

- Collect all available information relevant to the construction of each bridge, including;
    - Deck Concrete Properties
    - Specified Deck Concrete Construction Procedures
    - Approved Concrete Mixture Proportions
    - Measured Properties from QC Tests of Production Concrete
    - Measured Properties from Research Tests of Production Concrete
    - Actual Method of Deck Placement
    - Average Daily Traffic (ADT)
    - Average Daily Truck Traffic (ADTT)
    - Exposure Condition of the Bridge
    - Any Performed Maintenance
    - Any Inspection Reports
- Visually inspect the bridge, and obtain the following:
    - General condition of the deck top surface
    - Determination of the maximum crack width
    - General condition of the deck underside
    - General condition of the girders
    - Photograph areas of significant deterioration
    - Prepare drawings locating defects
    - Extract 8 concrete core samples

III. COMPILATION OF BRIDGE CONSTRUCTION INFORMATION

Information sources used for this report included bridge drawings, field inspection results, bridge summary data sheets, and technical information contained in FHWA's "Compilation and Evaluation of Results from High Performance Concrete Bridge Projects" version 3.

Deck Concrete Properties

Two HPC mixture designs were specified for use in the cast-in-place decks of the Louetta Road overpass bridge (Class K, and Class S Modified). The southbound cast-in-place deck utilized Class K (HPC) and the northbound cast-in-place deck utilized Class S (Modified). The Class K mixture design incorporated 32% fly ash by the weight of total cementitious materials and a high range water reducer (HRWR) for reduced water-to-cementitious material ratio (w/cm) and reduced permeability. The mixture had a specified maximum w/cm of 0.35. The Class K concrete may be classified as a high strength HPC. The Class S modified mixture design incorporated 28% fly ash by the weight of total cementitious materials without the use of a high or mid-range water reducer. A maximum w/cm of 0.43 was specified for this mixture. The Class S concrete may be classified as a normal strength HPC.

Table 1 lists the specified properties for concrete used in the cast-in-place decks of the two bridges. Limited information was available on the precast deck panels. For these panels, the slump of the mixture was specified as not to exceed 8 inches and the compressive strength at 28 days was specified as 8000 psi (Table 1)

TABLE 1: Specified Concrete Properties for Precast Deck Panels and Cast-in-Place Decks

Property	Precast Deck Panels	Northbound Cast-in-Place Deck (Class S HPC)	Southbound Cast-in-Place Deck (Class K HPC)
Max. Water/Cementitious Material Ratio:	---	0.43	0.35
Percentage of Fly Ash:	---	28%	32%
Slump:	≤ 8 in.	3 - 4 in.	8 - 9 in.
Air Content:	---	5%	0%
28 day Design Compressive Strength:	8000 psi	4000 psi	8000 psi
Chloride Permeability (AASHTO T 277):	Guideline of 1500 coulombs at 56 days		

Specified Deck Concrete Construction Procedures

The procedures specified for deck concrete construction were in conformance with the current TxDOT specifications for casting of bridge decks. It should be noted that HPC used in the construction of the Louetta Road overpass bridge incorporated a number of chemical and mineral admixtures. As a result, special attention was required during the concrete placement and curing, specifically for the HS/HPC mixture having a low water-to-cementitious material ratio (0.35). Since minimal bleeding water occurred from the mixture during concrete casting, plastic shrinkage cracking was a concern when the construction was done in conditions of high temperature and low relative humidity. Fogging of the deck while the concrete was still plastic was specified in the deck concrete construction procedures. Additionally, membranes such as curing compounds were specified after the surface finishing to help prevent moist loss.

For the cast-in-place deck a wet curing for 10 days was specified when fly ash was used and 8 days when no fly ash was used. Concrete cylinders 4 × 8 in. in dimensions and cured according to AASHTO T 23 were used for quality control testing.

Approved Concrete Mixture Proportions for Production Concrete

The Louetta Road overpass bridge has composite decks consisting of precast concrete deck panels (3.5-inches thick × 8-feet long), which are in turn covered with 3.75-inches of cast-in-place concrete. Table 2 provides the approved mixture proportions for the Class S normal strength HPC used in the northbound bridge decks and the Class K high strength HPC used in the southbound bridge decks.

TABLE 2: Approved Mixture Proportions for Precast Deck Panels and Cast-in-Place Decks

	Precast Deck Panels	Northbound Cast-in-Place Deck Class S (HPC)	Southbound Cast-in-Place Deck Class K (HPC)
Cement Brand:	Alamo	Capitol	Capitol
Cement Type:	III	I	I
Cement Quantity:	565 lb/yd^3	383 lb/yd^3	474 lb/yd^3
Fly Ash Type:	C	C	C
Fly Ash Quantity:	164 lb/yd^3	148 lb/yd^3	221 lb/yd^3
% Replacement by Weight:	22.5%	28%	32%
Fine Aggregate Type:	River Sand	River Sand	River Sand
Fine Aggregate FM:	2.60	2.54	2.54
Fine Aggregate Quantity:	1109 lb/yd^3	1243 lb/yd^3	1303 lb/yd^3
Coarse Aggregate, Max. Size:	¾ in.	1½ in.	1 in.
Coarse Aggregate Type:	No. 6 Crushed River Gravel	No. 4 Crushed Limestone	No. 5 Crushed Limestone
Coarse Aggregate Quantity:	1983 lb/yd^3	1856 lb/yd^3	1811 lb/yd^3
Water:	228 lb/yd^3	229 lb/yd^3	244 lb/yd^3
High-Range Water-Reducer Type:	F	--	F
High-Range Water-Reducer Quantity:	170 fl oz/yd^3	none	122 fl oz/yd^3
Retarder Type:	B and D	B and D	B and D
Retarder Quantity:	23 fl oz/yd^3	45 fl oz/yd^3	22 fl oz/yd^3
Air Entrainment Quantity:	None	2.1 fl oz/yd^3	None
Water/Cementitious Materials Ratio:	0.31	0.43	0.35

Measured properties of the approved concrete mixtures for precast deck panels and cast-in-place decks are summarized in Table 3.

TABLE 3: Measured Properties of Approved Mixture for Precast Deck Panels and Cast-in-Place Decks

Property	Precast Deck Panels	Northbound Cast-in-Place Deck	Southbound Cast-in-Place Deck
Slump, in	7 – 10	3 – 4	8 – 9½
Air Content, %	2.0%	5.0	0.9 – 1.4%
Unit Weight, lb/ft^3	149.9	143.2	150.2
Chloride Permeability (AASHTO T 277)	1430 coulombs @ 56 days	1730 coulombs @ 56 days	900 coulombs @ 56 days

The properties of the cement used in precast deck panels and cast-in-place decks are shown in Table 4.

TABLE 4: Properties of Cement used in the Construction of Precast Deck Panels and Cast-in-Place Decks

Property	Precast Deck Panels	Cast-in-Place Decks
Chemical, %		
SiO_2	18.95	20.24
Al_2O_2	6.50	5.66
Fe_2O_3	2.97	2.11
CaO	64.57	64.63
MgO	0.71	1.27
SO_3	3.79	3.16
Loss of Ignition	1.57	2.06
Insoluble Residue	0.24	0.19
Free Lime	1.70	N/A
C_3S	61.10	59.23
C_3A	1.80	11.43
Total Alkali	0.71	0.60
Specific Surface, cm^2/gm		
Blaine	6360	3430
Wagner	2933	1823
% Passing No. 325 Sieve	99.7	93.7
Compressive Strength, psi		
1 Day	4230	N/A
3 Day	5076	4085
7 Day	5930	5115
28 Day	N/A	N/A
Setting Time, min		
Vicat Initial	N/A	115
Final	N/A	180
Gilmore Initial	62	185
Final	118	350

Measured Properties from QC Tests of Production Concrete

Precast Deck Panel

For precast deck panels, limited amount of information was available on the measured properties from QC tests of production concrete. The typical properties of precast deck panels are documented later in this report under the research test results.

Cast-in-Place Deck

Table 5 summarizes the measured properties from QC tests of Class S and Class K production concrete used in the cast-in-place decks of both northbound and southbound bridges. In general, Class K concrete has better strength properties compared to Class S concrete for both standard and site cured specimens.

TABLE 5: Measured Properties of Production Concrete for Cast-in-Place Decks

Property	Age, days	Northbound Class S (HPC)		Southbound Class K (HPC)	
		Standard Cured	Site Cured	Standard Cured	Site Cured
Compressive Strength, psi	28 56	5600 5700	4890 5090	9630 9740	9220 9100
Modulus of Elasticity, ksi	28 56	4520 4870	4460 4010	5170 5750	4730 4990
Splitting Tensile Strength, psi	28 56	460 540	465 550	740 820	725 730
Chloride Permeability, coulombs (AASHTO T 277)	56	1730	2120	900	1300
Slump, in.		4		7	
Air Content, %		3.8		0	
Unit Weight, lb/ft^3		143		150	

Measured Properties from Research Tests of Production Concrete

Precast Deck Panel

The measured properties from research tests of production concrete used in the precast deck panels are shown in Table 6.

TABLE 6: Measured Properties from Research Tests of Production Concrete Used in the Precast Deck Panels

Property	Age, days	Standard Cured	Site Cured	Standard Cured	Site Cured	Standard Cured	Site Cured
Compressive Strength, psi (AASHTO T 22)	1	6010	-	-	-	-	-
	3	-	-	7380[(1)]	-	-	-
	7	6640	8260	8620	8870	-	-
	14	-	-	-	-	-	-
	28	8620	8440	9680	10050	-	-
	56	8810	9040	10330	10280	12370	11930
	90	-	-	-	-	12730	12290
Modulus of Elasticity, ksi (ASTM C 469)	1	5490	-	-	-	-	-
	3	-	-	4580[(1)]	-	-	-
	7	6000	5280	5530	5910	-	-
	14	-	-	-	-	-	-
	28	6040	5500	5900	5330	-	-
	56	6450	5640	6390	5650	6210	5890
	90	-	-	-	-	6520	6110
Splitting Tensile Strength, psi (ASTM C 496)	7	680	720	730	750	-	-
	28	800	780	760	810	-	-
	56	850	870	820	910	660	810
	90	-	-	-	-	860	880
RCPT, Coulombs (AASHTO T 277)	56	1420	1860	1460	2580	1260	1550

(1) Tested at 2 days.
NOTES: All tests were made using 4 × 8-in. cylinders.

The creep and shrinkage data for production concrete used in the precast deck panels is shown in Table 7. All 4 × 20-in. cylinders for the creep and shrinkage measurement were stored alongside the beams for 8 to 18 hours, stripped at approximately 24 hours after casting and loaded at an age of 2 days to 20 and 40 percent of the nominal design compressive strength of the mixture. Temperature and humidity were not controlled. Average relative humidity was 55 percent.

TABLE 7: Creep and Shrinkage Properties from Research Tests of Production Concrete Used in the Precast Deck Panels

Days after Loading	Creep Coefficient [(1)]	Specific Creep [(1)], millionths/psi	Shrinkage [(2)], millionths
7	0.54	0.085	80
28	0.77	0.120	198
56	0.92	0.143	233
180	1.16	0.180	268

(1) Reported creep values are the average values for specimens loaded to the 20 and 40 percent levels. Nine readings were taken on each specimen.
(2) Shrinkage values include adjustments for one day of drying before initial readings were taken and for length changes caused by variation in concrete temperatures.

Cast-in-Place Deck

The compressive strength, modulus of elasticity, splitting tensile strength, and chloride permeability of production concrete used in the cast-in-place decks are shown in Table 8.

TABLE 8: Measured Properties from Research Tests of Production Concrete Used in the Cast-In-Place Decks

Property	Age, days	Northbound Bridge Class S (HPC)		Southbound Bridge Class K (HPC)	
		Standard Cured	Site Cured	Standard Cured	Site Cured
Compressive Strength, psi	28 56	5600 5700	4890 5090	9630 9740	9220 9100
Modulus of Elasticity, ksi	28 56	4520 4870	4460 4010	5170 5750	4730 4990
Splitting Tensile Strength, psi	28 56	460 540	465 550	740 820	725 730
Chloride Permeability, coulombs	56	1730	2120	900	1300

Table 9 provides the creep and shrinkage data for production concrete used in the cast-in-place decks.

Table 9: Measured Creep and Shrinkage Properties of Production Concrete Used in the Cast-in-Place Decks

Days after Loading	Creep Coefficient [(1)]		Specific Creep [(1)], millionths/psi		Shrinkage [(2)], millionths	
	Northbound Class S	Southbound Class K	Northbound Class S	Southbound Class K	Northbound Class S	Southbound Class K
7	0.39	0.46	0.084	0.087	66	91
28	0.74	0.80	0.160	0.152	178	238
56	0.98	1.09	0.213	0.206	240	279
180	1.47	1.69	0.317	0.320	296	344

(1) Reported creep values are the average values for specimens loaded to the 20 and 40 percent levels. Nine readings were taken on each specimen.

(2) Shrinkage values include adjustments for one day of drying before initial readings were taken and for length changes caused by variation in concrete temperatures.

Compared to Class S HPC, the Class K HPC has higher strength and low chloride permeability; however, it also shows greater shrinkage value. It is noted that a greater shrinkage value may be responsible for the cracking to be discussed as follows.

Actual Method of Deck Placement

Construction of the Louetta Road overpass bridge decks occurred in October 1996, with the concrete for the decks pumped from a truck or placed using a concrete bucket. Prior to placement of the cast-in-place decks, the precast deck panels were saturated to prevent the loss of mixing water.

Concrete was distributed by a mechanical spreader. The concrete was compacted using internal vibrators and a rolling screed to provide proper consolidation and avoid internal segregation. A final troweled finish was applied followed by tining for enhanced skid resistance. Surface finishing consisted of motorized screed pan with a burlap drag.

Fogging of the concrete decks started when the concrete was still in plastic state. A curing compound was applied in addition to the continuous fogging. The wet mats were kept moist for 10 days after casting for the HPC decks with pozzolans. This procedure was aimed at avoiding the surface moisture evaporation and plastic shrinkage cracks.

For each bridge all three spans of the cast-in-place portion of the decks were placed in a single pour. Shortly after placement of concrete, tooled control joints were placed at each interior bent to control cracking.

Average Daily Traffic (ADT) and Average Daily Truck Traffic (ADTT)

Average daily traffic for northbound lanes was calculated based on a count of all vehicles crossing the bridge during a 15 minute period beginning at 1000 hrs on August 25, 2003. These vehicle counts gave at an ADT of 27,360 and an ADTT of 8,784.

Average daily traffic for southbound lanes was calculated based on a count of all vehicles crossing the bridge during a 15 minute period beginning at 0836 hrs on August 26, 2003. These vehicle counts gave an ADT of 81,120 and an ADTT of 3,360.

Exposure Condition of the Bridge

The Louetta Road overpass bridge is on State Highway 249 in northwest Houston, Texas. The bridge improved the State Highway 249 from a non-freeway facility to a freeway facility in northwest Harris County. The National Weather Service reports that the average high temperature is 93.1°F in July and the average low temperature is 38.4°F in February. The minimum temperature varies between 75.8°F in July and 33.7°F in February. The normal precipitation varies between 14.65 inches per month in October to 0.89 inches per month in February. Based on the National Weather Service record there is minimal annual exposure to wet/dry and freeze/thaw cycles. No deicing salt had been applied to the bridge decks by November 2003.

Performed Maintenance

No documents were found which would indicate any maintenance had been performed since bridge construction in 1994.

Inspection Reports

Several inspection trips to the Louetta Road overpass bridge had been made by the Construction Materials Research Group (CMRG) personnel from the University of Texas at Austin in June and July 1998. The inspections approximately 19 months after the decks were cast were to inspect and map crack patterns on the surface of the decks through visual observation. Further information is available through the Project report 7-3993 of the Center for Transportation Research at The University of Texas at Austin.

According to the report, similar crack patterns were noted for both the southbound and northbound bridge decks. However, the extent of longitudinal cracking was reported to be slightly higher on the southbound deck (constructed with Class K high strength HPC). The crack widths were measured to be less than 1/32 in. on the surface of the concrete decks. Crack widths were similar for mainspans of both the southbound and northbound bridges. Rectangular pattern cracking was observed on both the southbound and northbound bridges.

IV. BRIDGE DECK INSPECTION

PSI personnel performed a close visual inspection of the bridge deck during the week of August 25, 2003. The results of that inspection are summarized as follows.

General Condition of the Deck Top Surface

Figure 1 shows the general layout of the decks for the Louetta Road overpass bridge. Results of visual inspection of the decks of the two bridges are shown in Figure 2. No apparent sign of abrasion damage, freeze-thaw damage, D-cracking, pop-out, and alkali aggregate reaction (AAR) was observed. Surface defects observed and documented during visual inspection primarily included transverse cracks, longitudinal cracks, and diagonal cracks (see photos 5 and 6). Other defects observed and documented included small spalls at joints and cracks; exposed reinforcing steel at one location; and inclusion of wood pieces at one location. Small drilled holes (¾-in. diameter) and core locations with failing patches, which resulted from previous investigation by others, were also observed.

A total of 1,703 cracks (longitudinal, traverse, and diagonal) were recorded during visual survey of the bridge decks (see Figure 2). The sum of crack lengths was 11,098.4 ft over a bridge deck area of 66,636 ft^2. Crack density (total crack length / deck area) for the northbound and southbound bridges combined was calculated to be 0.167 ft/ft^2.

No significant difference was noted in the magnitude and pattern of cracking in the northbound and southbound bridges where two different classes of HPC were used (see Figure 2). Note that in the southbound bridge Class K high strength HPC was used and in the northbound bridge modified Class S normal strength HPC was used. Though the two bridges are exhibiting significant level of cracking, majority of the cracks are hairline cracks with a width of less than 1/32 in. A typical crack on the bridge decks is shown in photo 7. The number of cracks that were classified as hairline cracks totaled 1,671 with a combined total length of 10,810.2 ft. A relatively small number of cracks were classified as fine cracks with widths in the range of 1/32 to 1/16 in. The number of these cracks was 32 and their combined total length was 288.2 ft.

Figure 3 identifies the locations of the fine cracks along with other defects such as spalls, exposed rebar and wood inclusions. It can be noted that the fine cracks were limited at span ends along the skew. Small surface spalls, which either occurred due to breaking of tined edges or the crack edges, were observed at 23 locations (see Figure 3). Photo 8 illustrates typical spalling due to breaking of crack edges. A rebar was visible at one location in Span 1 of the southbound bridge (see Figure 3). Also inclusion of wood pieces was observed at one location in Span 3 of the southbound bridge (see Figure 3).

Figure 3 also illustrates the locations of drilled holes and cores, which resulted from previous investigation by others. Patching of most of the core holes was failing.

The number, length and density of cracks for each structure are shown in Tables 10 through 15, and described below according to the crack type.

Transverse Cracks: The transverse cracks in the northbound and southbound bridges were comparable. A total of 228 cracks were observed in three spans of the northbound bridge with a combined total length of 955 ft. A total number of 282 cracks with a combined total length of 1,456.7 ft were observed in the three spans of the southbound bridge (see Tables 10 and 11). The crack length per deck area for the northbound and southbound bridges was 0.03 ft/ft^2 and 0.04 ft/ft^2 respectively. Like longitudinal cracks, a number of transverse cracks were at the boundaries of the precast deck panels, creating semi-rectangular patterns.

Longitudinal Cracks: The number and length of longitudinal cracks are significantly greater than those of the transverse and diagonal cracks. The length per deck area in the southbound and northbound bridges was estimated to be 0.11 ft/ft^2 and 0.13 ft/ft^2, respectively (see Tables 12 and 13). The span length did not appear to have a distinct correlation with the magnitude of cracking. In the southbound bridge, Span 1, Span 2, and Span 3 with lengths of 121.5 ft, 135.5 ft, and 134 ft had crack length per deck area of 0.11 ft/ft^2, 0.14 ft/ft^2, and 0.10 ft/ft^2, respectively. In the northbound bridge, the corresponding crack length per deck area was 0.11 ft/ft^2, 0.13 ft/ft^2, and 0.16 ft/ft^2. Crack lengths as large as 131.5 ft and 109.4 ft were observed in the northbound and southbound bridges, respectively. Several of the longitudinal cracks were along the U beams and at the boundaries of the precast deck panels.

Diagonal Cracks: Diagonal cracks accounted for the least amount of total cracks and were comparable in the northbound and southbound bridge decks. These diagonal cracks were typically present in the acute corners and near the joints. The crack length per deck area was 0.009 ft/ft^2 in both the northbound and southbound bridges (see Tables 14 and 15).

TABLE 10: Measured Transverse Cracks on the Surface of Northbound Cast-in-Place Bridge Deck

Northbound Transverse Cracks	Count	Length Range (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span 1	66	0.8 to 17.3	275.6	10340	0.027
Span 2	76	1.0 to 13.5	302.9	10735	0.028
Span 3	86	1.2 to 18.9	376.5	9805	0.038

TABLE 11: Measured Transverse Cracks on the Surface of Southbound Cast-in-Place Bridge Deck

Southbound Transverse Cracks	Count	Length Range (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span 1	84	1.2 to 15.2	456.2	12348	0.037
Span 2	126	0.7 to 20.5	692.7	12530	0.055
Span 3	72	1.4 to 20.8	307.8	10879	0.028

TABLE 12: Measured Longitudinal Cracks on the Surface of Northbound Cast-in-Place Bridge Deck

Northbound Longitudinal Cracks	Count	Length Range (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span 1	113	0.8 to 120.4	1088.3	10340	0.105
Span 2	216	0.8 to 131.5	1369.9	10735	0.128
Span 3	247	0.9 to 137.2	1581.0	9805	0.161

TABLE 13: Measured Longitudinal Cracks on the Surface of Southbound Cast-in-Place Bridge Deck

Southbound Longitudinal Cracks	Count	Length Range (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span 1	205	0.8 to 105.5	1267	12348	0.103
Span 2	188	1.1 to 109.4	1719.9	12530	0.137
Span 3	125	1.1 to 55.9	1062	10879	0.098

TABLE 14: Measured Diagonal Cracks on the Surface of Northbound Cast-in-Place Bridge Deck

Northbound Diagonal Cracks	Count	Length Range (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span 1	10	1.8 to 16.3	79	10340	0.008
Span 2	24	1.6 to 13.3	164.7	10735	0.015
Span 3	7	1.4 to 12.3	31.2	9805	0.003

TABLE 15: Measured Diagonal Cracks on the Surface of Southbound Cast-in-Place Bridge Deck

Southbound Diagonal Cracks	Count	Length Range (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span 1	15	1.5 to 12.3	59.5	12348	0.005
Span 2	17	1.4 to 12.9	116.6	12530	0.009
Span 3	26	1.7 to 13.8	147.5	10879	0.014

Crack Widths

About 98% of the cracks on the two bridges were hairline cracks with a width of less than 1/32 in. The remaining 2% of the cracks were classified as fine cracks with widths in the range of 1/32 to 1/16 in. The fine width cracks were generally located at span ends along the skew and some exhibited spalling due to the breaking of the edges (see photo 8).

General Condition of the Deck Underside

The underside of the decks was inspected from the ground without the aide of any access equipment. The underside of the decks was generally in good condition with a few exceptions. Cracks were visible on the underside of a few precast deck panels at span ends (see photo 9). Efflorescence was visible at these crack locations.

General Condition of the Girders

The girders were inspected from the ground without the aide of any access equipment. Visible cracks were observed in only one girder. A series of fine cracks approximately 2 to 5 inches long were observed on the northeastern most girder of the northbound bridge near Abutment 2 (see photo 10).

Concrete Core Samples

Eight core samples were retrieved from the decks during the inspection. Core sample locations are shown on Figure 1. The cores were 3-¾-in. diameter and were labeled as 1 through 8. The cores were transferred to FHWA for further analysis.

Preliminary Conclusions

The construction of northbound and southbound State Highway 249 Bridges over Louetta Road was completed in 1997. Under a research project 7-3993, researchers from the University of Texas at Austin performed a visual inspection of the two bridges 19 months after the construction. It was reported that the two bridges exhibited longitudinal, transverse, and zigzag cracks at the time of this inspection and the crack widths were less than 1/32 in. The crack pattern in the two bridges was reported to be similar.

The visual inspection of the bridge decks as part of our study was performed about five years after the previous inspection. The northbound and southbound bridges are exhibiting comparable magnitude and pattern of cracking. A total of 1,703 longitudinal, transverse, and diagonal cracks were recorded on the two bridges with a combined total crack length of 11,098.4 ft over a bridge deck area of 66,636 ft^2. However, 98% of these cracks were hairline cracks with width less than 1/32 in. The remaining 2% of the cracks were classified as fine cracks with widths in the range of 1/32 to 1/16 in.

The cast-in-place decks of the northbound and southbound bridges were constructed with two different classes of HPC. Normal strength modified Class S HPC was used in the northbound bridge, and high strength Class K HPC was used in the southbound bridge. However, it appears that mixture proportions did not play a significant role in the cracking of the decks. Class K HPC used in the southbound bridge was reported to have a low w/cm of 0.35 and a fly ash content of 32% by weight of the cementitious material content. This Class K HPC mixture had a high shrinkage and cracking potential. However, the performance of this mixture was comparable to normal strength modified Class S HPC used in the northbound bridge, which was reported to have a w/cm of 0.43 and a fly ash content of 28% by weight of the total cementitious material content. This indicates that the curing procedures used during construction were probably effective.

Significant difference in the coefficient of thermal expansion of the precast deck panels and cast-in-place decks may partly be attributed to the shrinkage cracks observed in the two bridges. It was reported that the coefficient of thermal expansion of the cast-in-place deck was about 4.0 $\mu\varepsilon$ / ˚F. On the other hand the coefficient of thermal expansion of the precast deck panels was reported to be about 7.3 $\mu\varepsilon$ / ˚F.

It was also reported that the construction of all the spans of the northbound and southbound bridges was done as a single pour construction without properly locating the tooled control joints at the centerline of the skew. This single pour construction might have also contributed to the development of cracks observed at the northbound and southbound bridges.

At span ends along the skew, a number of fine width cracks (1/32 to 1/16 in.) were observed. Some of these cracks were exhibiting spalling due to breaking of the edges. The layout of the cast-in-place decks and precast deck panels at span ends may partly be attributed to the development and widening of these cracks. At span ends, the cast-in-place decks were skewed but precast deck panels had a straight geometry. Skewed deck panels at span ends might have helped control these cracks.

The structural system of the northbound and southbound Louetta Road overpass bridges is flexible compared to conventional bridges considering the wide beam spacing, large span, and relatively thin deck used on these bridges. This relatively flexible structural system combined with the heavy ADT and ADTT on these bridges might have contributed to the development and widening of some cracks on the bridges.

It is noted that for the longitudinal cracking, another factor that could contribute is shortening of the precast panels in the transverse direction. As the panels shorten because of creep and shrinkage, the cast-in-place portion of the deck has to accommodate the movement. This can lead to tensile stresses in the cast-in-place concrete. In addition, the Texas U-beam is stiffer than the same depth I-beam. This means that any transverse shortening of the deck is going to encounter more resistance with a U-beam than with an I-beam. This will also lead to higher tensile stresses in the deck with a U-beam and greater likelihood of longitudinal cracking.

Petrographic analysis was performed on the eight core samples that were retrieved from the decks of the bridge. Three cores (#1 to #3) were drilled from the northbound bridge, and five (#4 to # 8) were drilled from the southbound bridge, as shown in Figure 1. The collected cores represented the cast-in-place concrete. Cracks ran through cores #3 and #6. #3 core was a 2-inch thick disc, while #4 core was about 4-inch high. The examination of the broken cores (#3 and #6) revealed no evidence of material related deterioration. The major cracks that ran through the cores might be in existence in the concrete for a relatively long time, and the inner surface was covered with dust. The rest of the six concrete cores appeared sound, and visual inspection of the concrete cores revealed no further defects.

The coarse aggregate in the concrete was crushed limestone. Coarse aggregate particles were angular, and the maximum size was about 3/4 inch. Preferential orientation of coarse aggregate particles was not observed in the concrete core samples, nor was segregation. The fine aggregate fraction was mainly composed of quartz, with some quartzite and chert. The fine aggregate was from natural sand and the particles appeared rounded to angular.

The hydration of the cement was reasonably adequate in respect to the age of the concrete. The cement paste contained some unhydrated cement particles as seen under the microscope. Fly ash particles were also present in the concrete.

As mentioned previously in this report, the northbound cast-in-place concrete was air entrained. Microscopic examination revealed that small, spherical air voids were present in the concrete. The air content was estimated to be low.

The paste/aggregate interface generally appeared sound, and the bonding between aggregate and paste was strong. However, cracks in the interfacial region were sporadically observed under the microscope.

Cracks were also sporadically observed in the cement paste matrix. Most of these microscopic cracks were isolated cracks in cement paste. They might form connection with the cracks in the paste/aggregate interfacial region. Occasionally, there were cracks in coarse aggregate particles. These cracks could extend from coarse aggregate into cement paste and continued in the aggregate/paste interfacial region.

Small amount of ettringite crystals was found sporadically in air voids in the concrete. It was observed that some air voids were partially filled with ettringite, and the typical formation of ettringite was to line the internal wall of the void. Occasionally, air voids that were fully filled with ettringite were also observed under the microscope. There was no evidence of any damage or deterioration associated with the formation of the ettringite crystals in air voids. It was believed that the ettringite crystals in this concrete were harmless. It is common to see ettringite as secondary deposit in concrete, especially for the concrete that has been in service for some time.

Respectfully Submitted,
Professional Service Industries, Inc.
Structural Investigation and Petrography Department

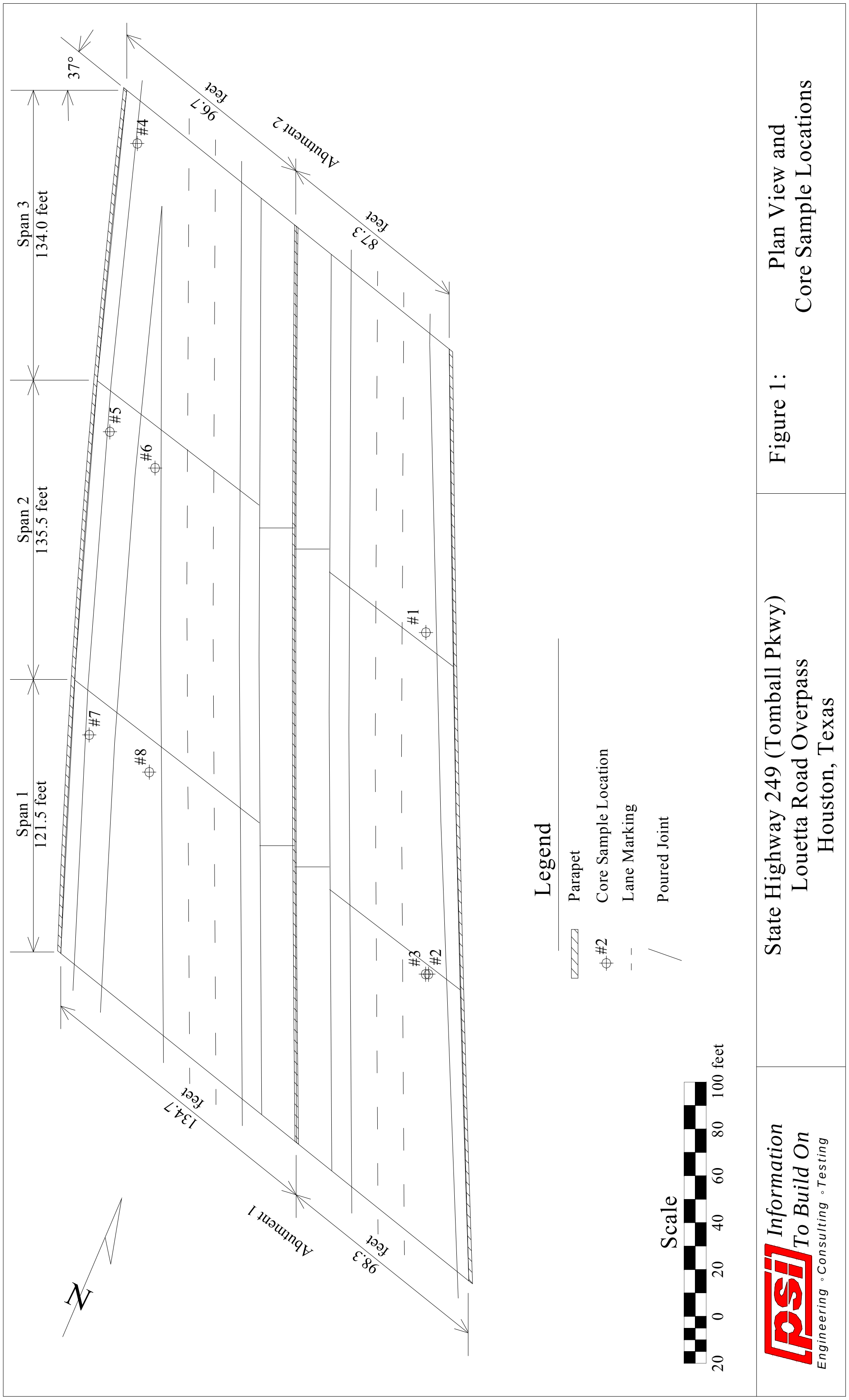
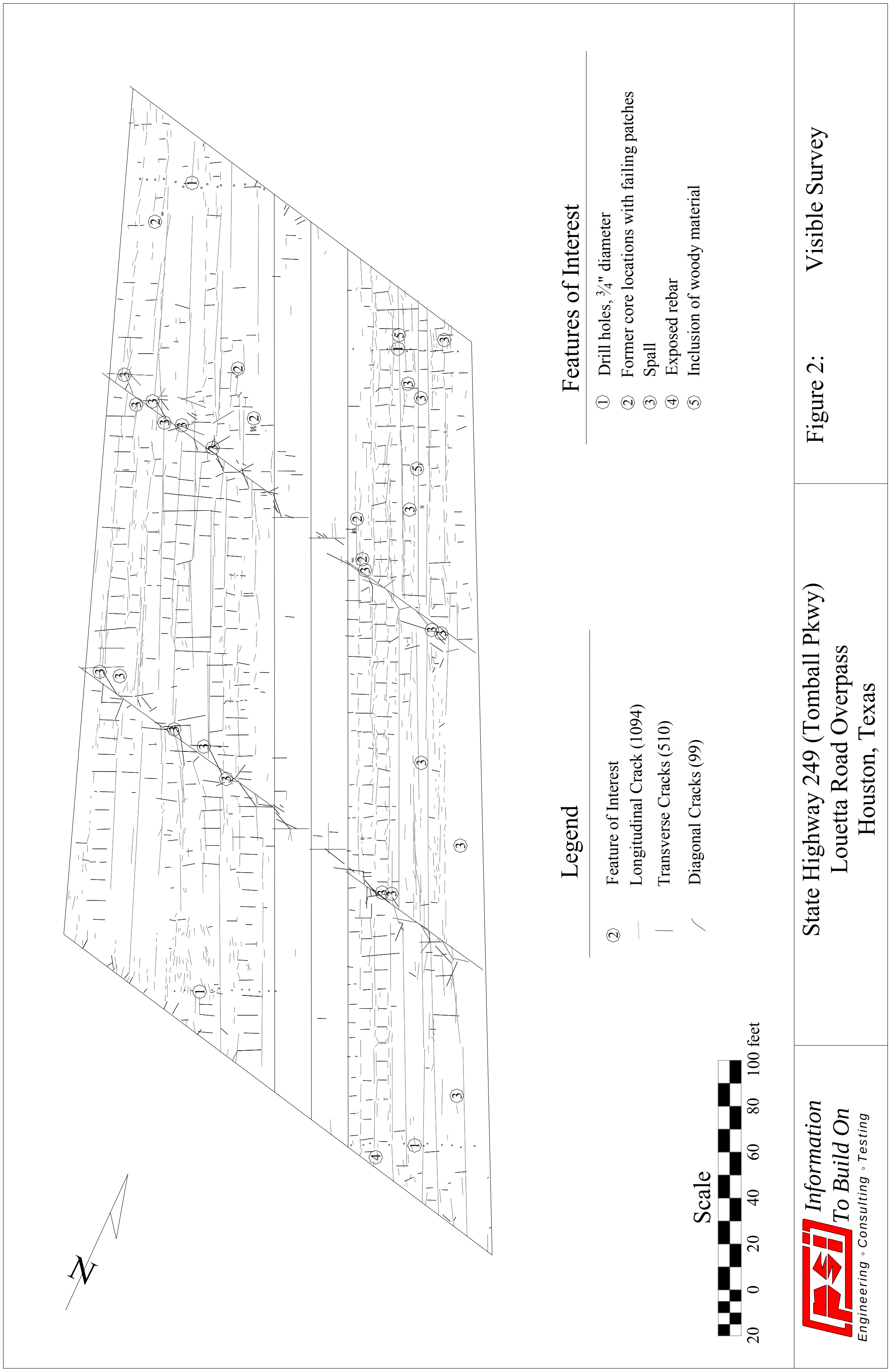


Figure 1: Plan View and Core Sample Locations


N
Scale
20 0 20 40 60 80 100 feet
Legend
② Feature of Interest
Longitudinal Crack (1094)
Transverse Cracks (510)
Diagonal Cracks (99)
Features of Interest
① Drill holes, $^3/_4$" diameter
② Former core locations with failing patches
③ Spall
④ Exposed rebar
⑤ Inclusion of woody material
PSI Information To Build On
Engineering • Consulting • Testing
State Highway 249 (Tomball Pkwy)
Louetta Road Overpass
Houston, Texas
Figure 2: Visible Survey

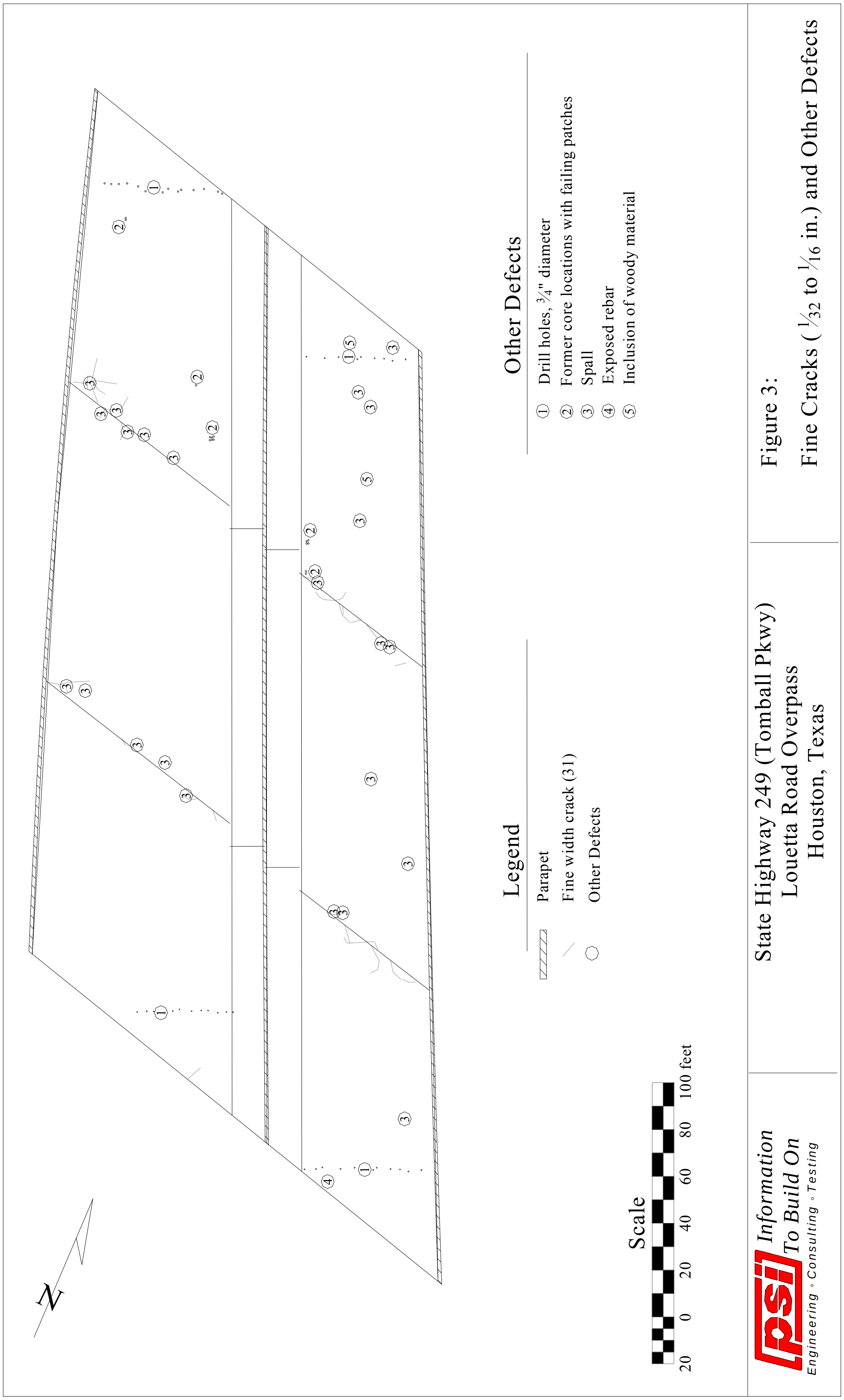


Figure 3:
Fine Cracks ($\frac{1}{32}$ to $\frac{1}{16}$ in.) and Other Defects

Photographic Documentation

Photo 1: View looking south at the northbound bridge.

Photo 2: View looking south at the southbound bridge.

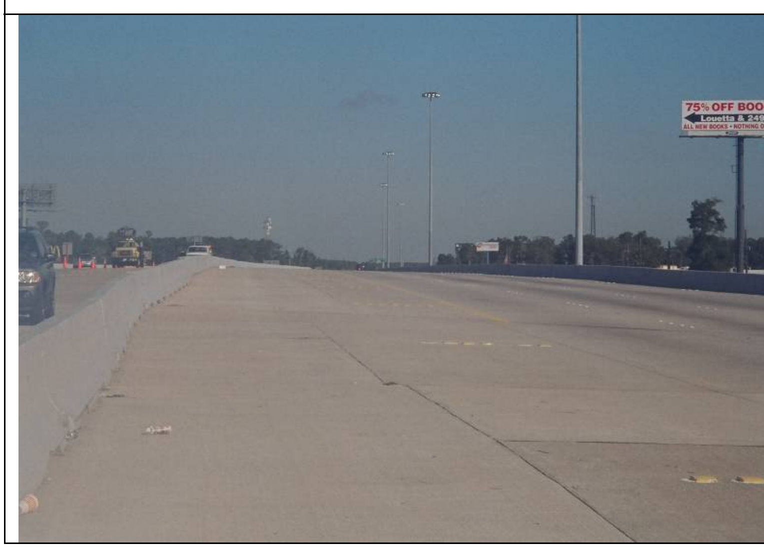


Photo 3: View looking north at the northbound bridge. The approach ramp is in foreground. Span 1 begins at mid-photo.

Photo 4: View showing bridge substructure consisting of U-beam girders and columns. Separation between northbound and southbound bridges is shown at the center.

Photo 5: View looking north at the left side of the northbound bridge.

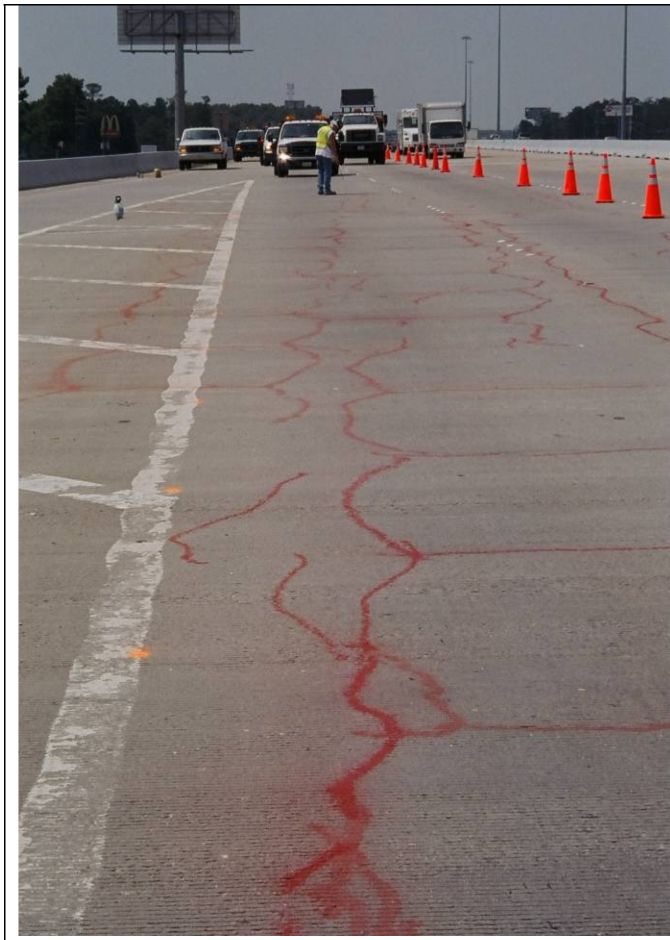
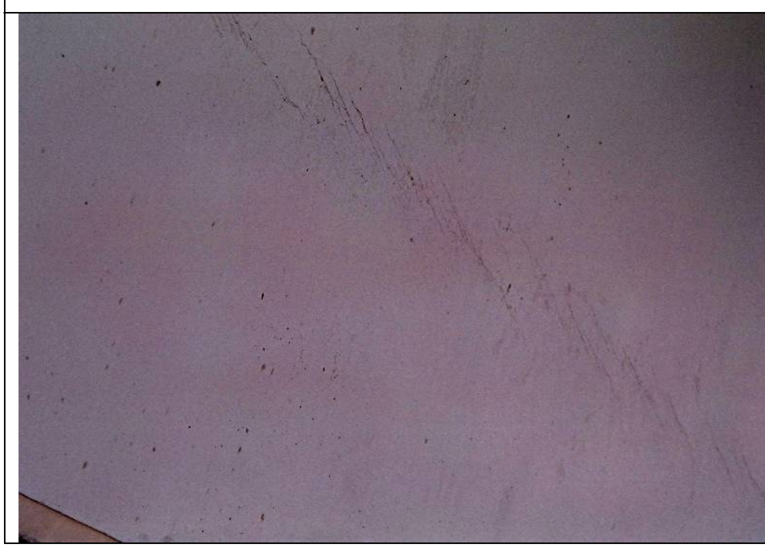


Photo 6: View looking north at southbound bridge.

Photo 7: View showing typical crack in the bridge deck. Crack width is less than 1/32 in. and is categorized as hairline.

	Photo 8: View showing spalling associated with cracks in the deck at span ends.
	Photo 9: View showing underside of bridge at span end.
	Photo 10: View showing close-up of fine cracks in the northeastern most girder of the northbound bridge.

APPENDIX O – Supplement 1

SH 249 over Louetta Road, Houston, Texas
Petrographic Report

PETROGRAPHIC EXAMINATION OF CONCRETE CORES FROM HOUSTON, TEXAS

A report submitted to the Structures Laboratory at TFHRC

Rongtang Liu
Petrographic Laboratory
FHWA TFHRC

March 29, 2005

Abstract

Eight concrete cores were received from the Structures Laboratory of the Turner-Fairbank Highway Research Center for petrographic examination. Reportedly, the concrete cores were collected from a concrete bridge on Louetta Road, Houston, Texas.

Petrographic examination was performed on samples using optical microscopes. Thin sections, as well as polished slabs, were made from the concrete cores for microscopic examination.

Among the eight 3-¾-in. diameter cores received, two were broken: #3 is a 2-inch disc, and #6 is about 4-inch high. The rest of the six concrete cores appeared sound, and visual inspection of the concrete cores revealed no further defects. The findings from microscopic examination indicate that the concrete has entrained air voids. The cement was reasonably hydrated with hard paste. The presence of low amount of unhydrated cement and fly ash particles was also observed in the concrete. Sporadic micro-cracks were present in the concrete. Ettringite as secondary deposit formed in some air voids.

Introduction

The Petrographic Laboratory of the FHWA Turner-Fairbank Highway Research Center was asked by the Structures Laboratory to examine a set of concrete cores identified as retrieved from a bridge called Louetta Road Bridge, Houston, Texas. Eight concrete cores of 3-3/4-in. diameter, 2- to 4-in. long were received by the Petrographic Laboratory. The identification on the cores was as follows: #1, #2, #3, #4, #5, #6, #7, and #8. The two broken cores (#3 and #6) were examined more extensively.

Laboratory Procedures

Petrographic examination of the concrete samples was performed in accordance with ASTM C 856, "Standard Practice for Petrographic Examination of Hardened Concrete."

Sections were polished and examined using a stereomicroscope at magnifications up to 350×. Small rectangular blocks were cut from concrete samples. One surface of each block was polished, dried, placed on a petrographic slide with low-viscosity epoxy resin, and reduced to a thickness of approximately 25 micrometers (0.001 in.). These thin sections were examined using a polarized–light microscope at magnification up to 400×, to determine aggregate mineralogy, paste characteristics, and microstructure.

Two ¾-inch thick slabs were sawn from the concrete cores. The slabs were ground and polished to obtain a smooth and plane surface. They were examined using a stereomicroscope at magnifications up to 200×.

Findings

Thirteen thin section samples taken from the received cores were examined using the polarized light microscope. Two polished sections were examined using the stereomicroscope. The findings are summarized as follows:

Aggregate

The coarse aggregate in the concrete is crushed limestone. Coarse aggregate particles are angular, and the maximum size is about 3/4 inch. Preferential orientation of coarse aggregate particles is not observed in this concrete, nor is segregation.

The fine aggregate fraction is mainly composed of quartz, with some quartzite and chert. The fine aggregate is from natural sand and the particles appear rounded to angular.

Cement Paste

The cement is reasonably hydrated and the paste contains some unhydrated cement particles as seen under the microscope (Figure O1-1 and Figure O1-2). Fly ash particles were also present in the concrete, as shown in Figure O1-3.

Air Voids

Small, spherical air voids are observed in the concrete (Figure O1-4 and Figure O1-5), hence the concrete was air entrained. The air content is low.

Cracks

Isolated cracks in cement paste are sporadically observed in the concrete, as shown in Figure O1-6. Cracked paste/aggregate interfacial region is also present in the concrete (Figure O1-7). Occasionally, cracks may extend from coarse aggregate into cement paste and continue in the aggregate/paste interfacial region, as shown in Figure O1-8.

Paste/Aggregate Interface

In general, the paste/aggregate interface appears solid and dense, as shown in Figure O1-9 and Figure O1-10. As mentioned above, cracks in the interfacial region were sporadically observed in the concrete.

Secondary Deposit

Small amount of ettringite was found sporadically in air voids in the concrete. Figure O1-11 shows the internal wall of a void partially lined with ettringite crystals, while Figure O1-12 shows a void fully filled with ettringite.

The examination of the broken cores (#3 and #6) revealed no evidence of material related deterioration. The major cracks have existed in the concrete for a relatively long time, and the inner surface was covered with dust. Although the exact cause of the crack is not clear, it is possible that they are due to mechanical break.

Summary

The concrete was air entrained (although at a low level of air content), and the entrained air voids were well distributed in the concrete. Sporadic micro-cracks exist in cement paste, in the aggregate/paste interfacial region, as well as in coarse aggregate. Despite the defects in microscopical scale, the concrete appears solid and sound.

Ettringite crystals form in some air voids. There was no evidence of deterioration related with the existence of the ettringite in the concrete. It is very common to see ettringite as secondary deposit in concrete.

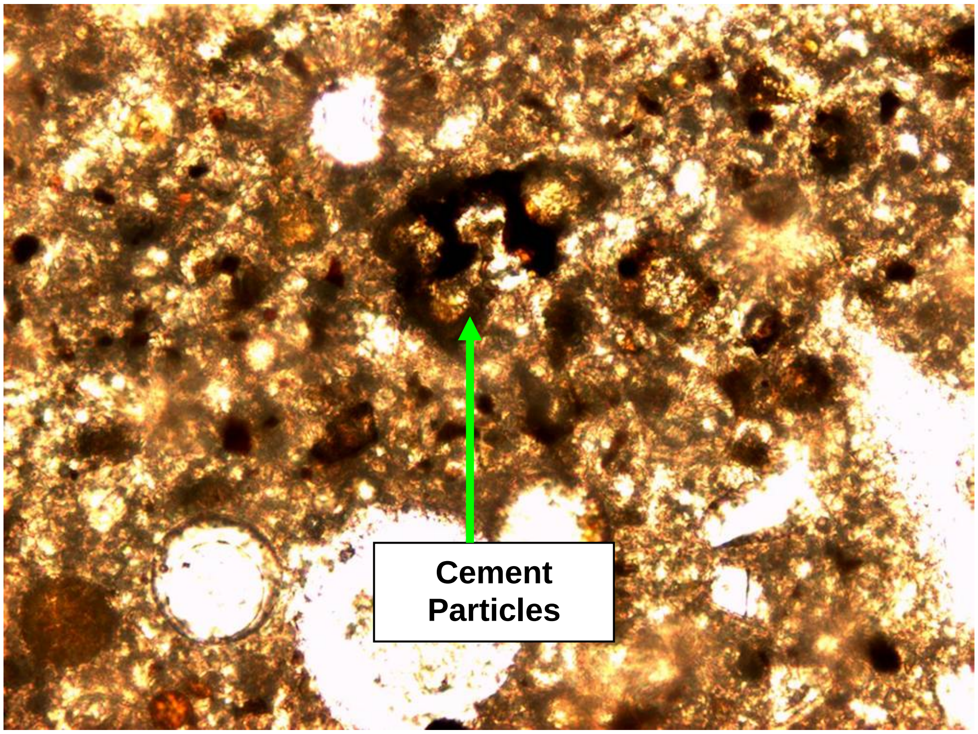


Figure O1-1: Unhydrated cement in paste. Width of field is 0.33 mm. Thin section image.

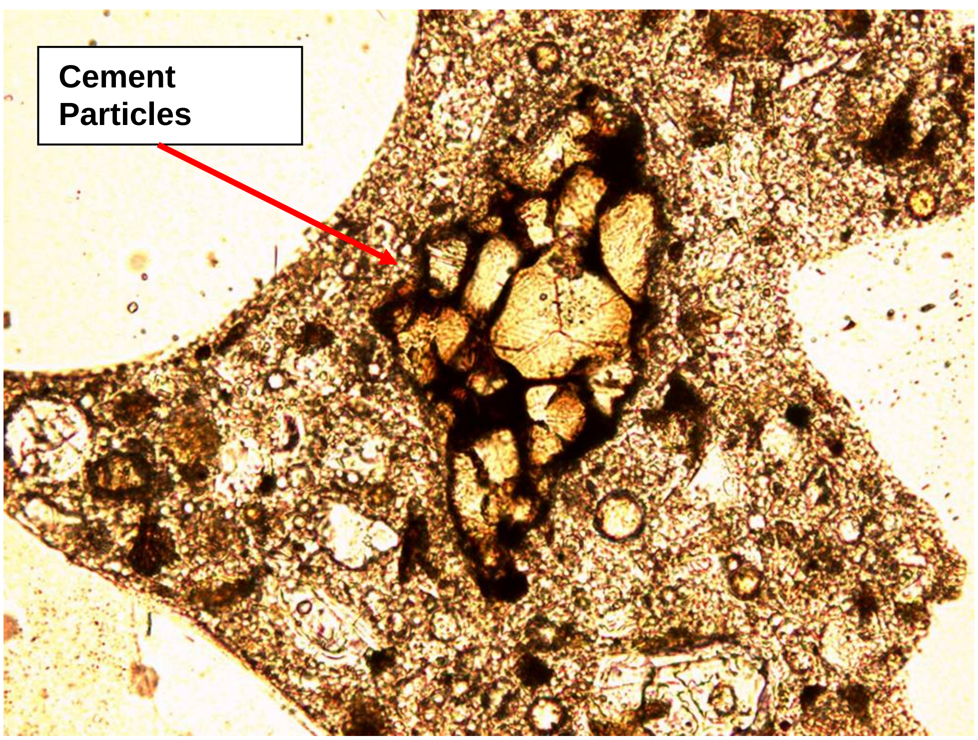


Figure O1-2: Another image of unhydrated cement. Width of field is 0.33 mm. Thin section image.

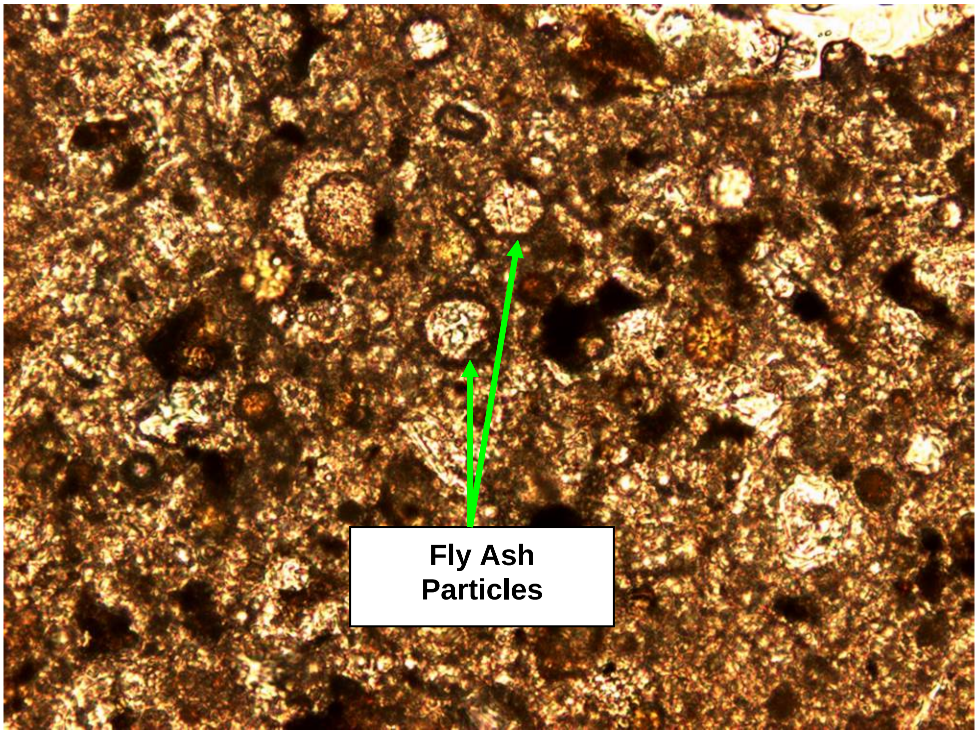


Figure O1-3: Fly ash particles in the concrete. Width of field is 0.33 mm. Thin section image.

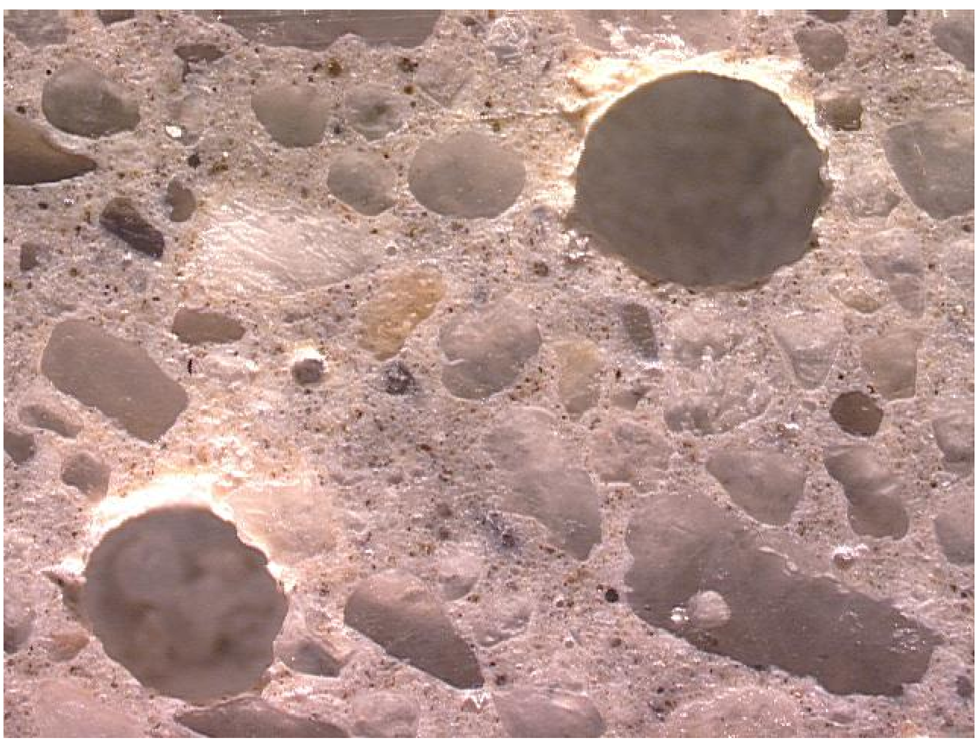


Figure O1-4: Air voids in the concrete. Width of field is 4.0 mm. Polished concrete surface image.

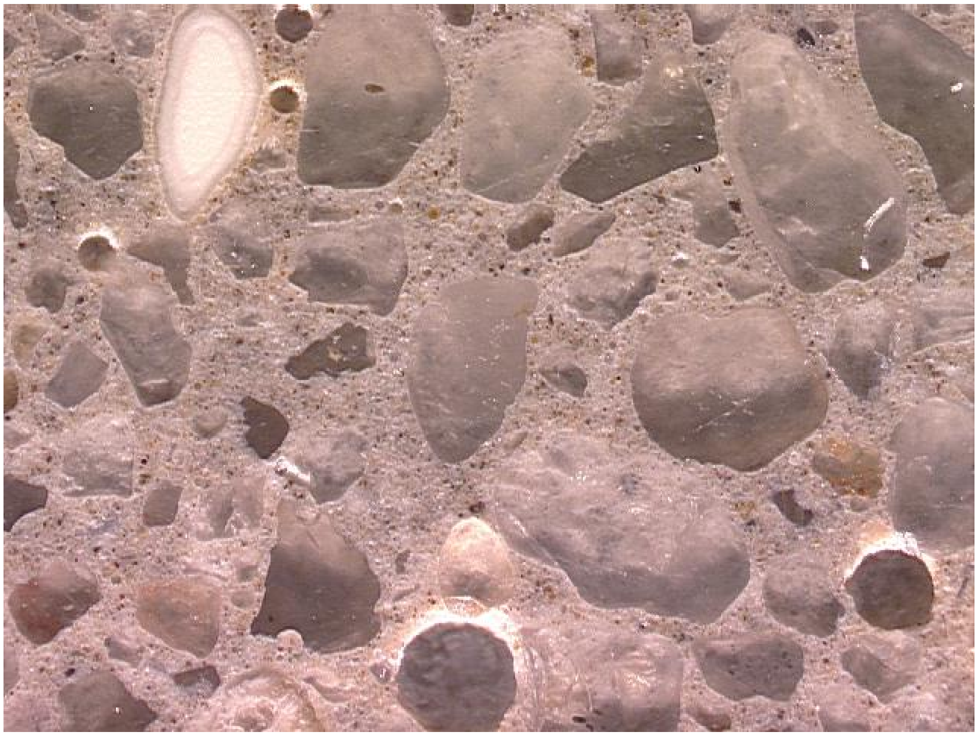


Figure O1-5: Another view of the air voids in the concrete. Width of field is 4.0 mm. Polished concrete surface image.

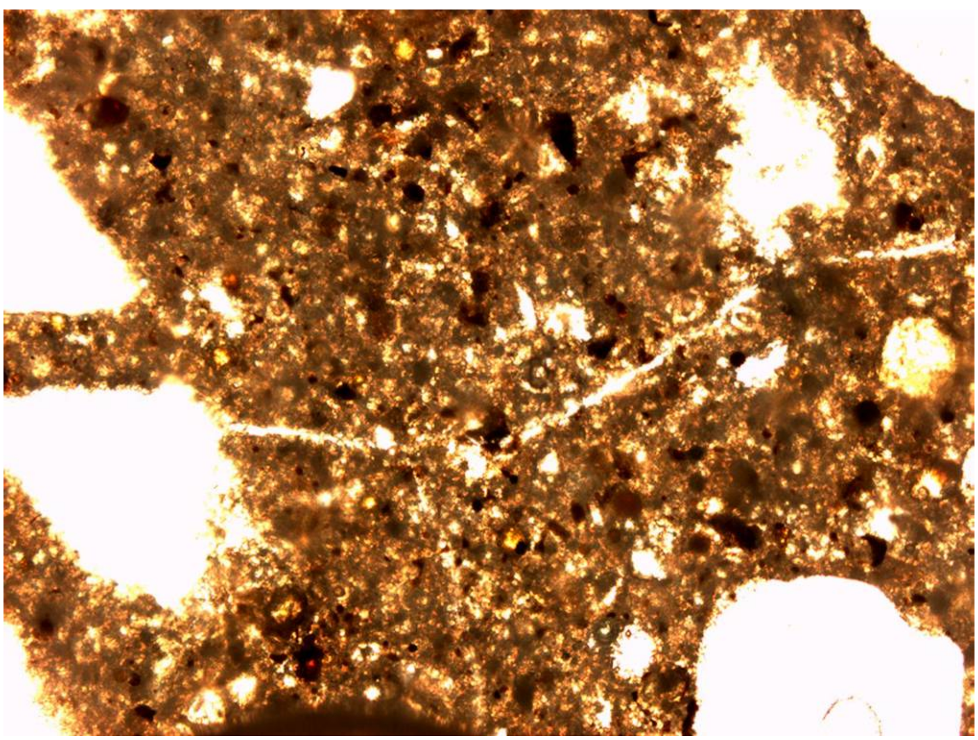


Figure O1-6: Micro-cracks in the concrete. Width of field is 0.65 mm. Thin section image.

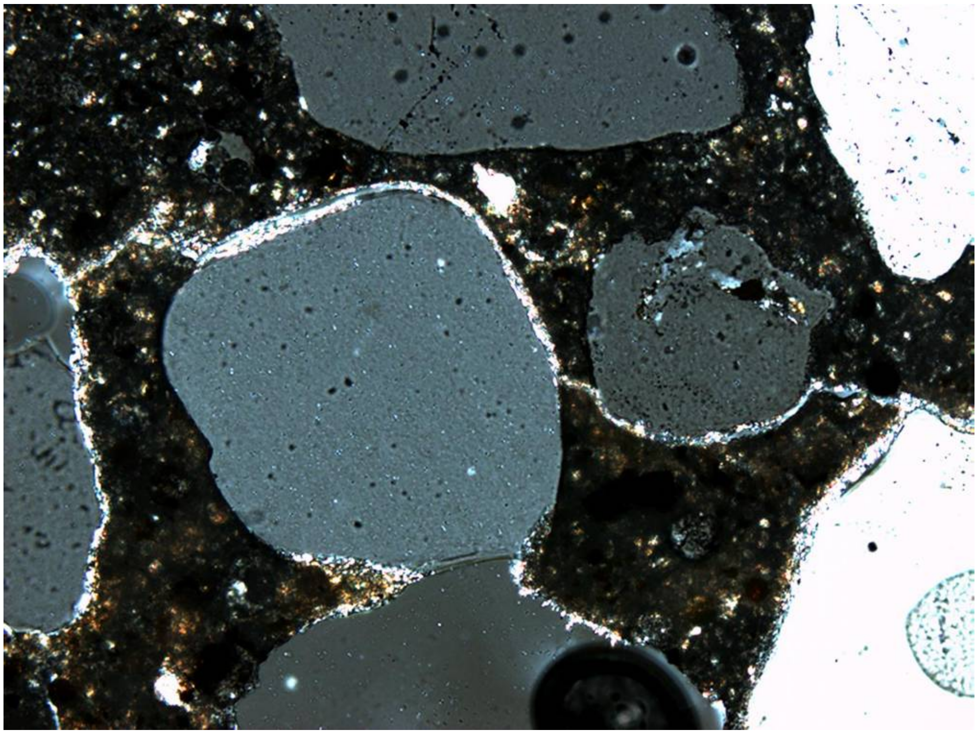


Figure O1-7 Cracks in the aggregate/paste interfacial region. Width of field is 0.65 mm. Thin section image.

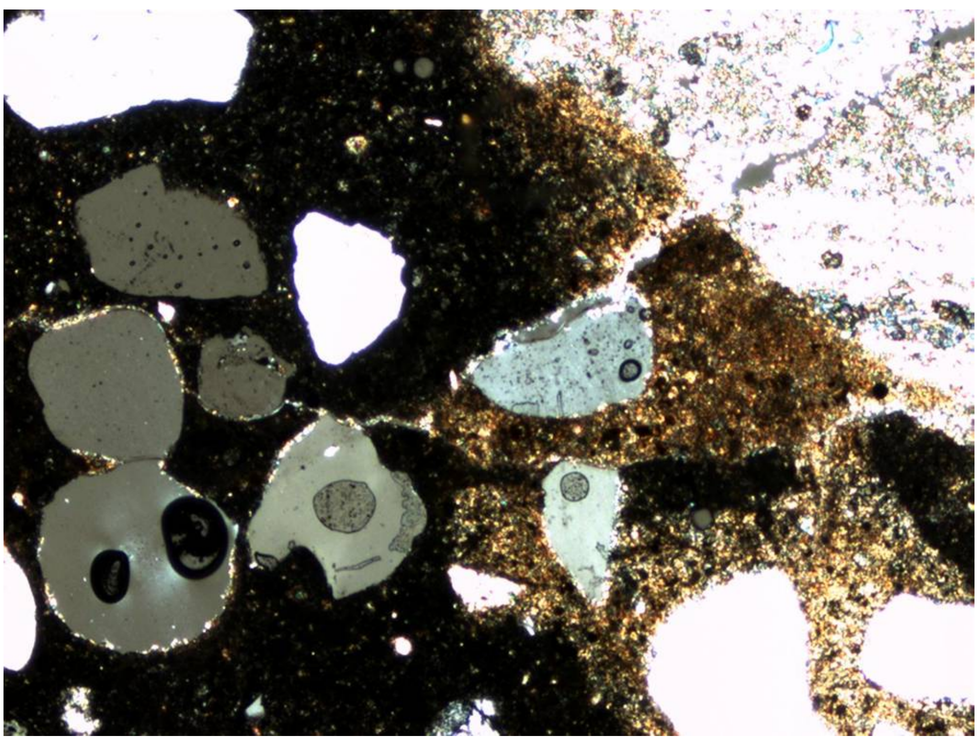


Figure O1-8: Another image showing cracking in the coarse aggregate, cement paste, and fine aggregate/paste interfacial region. Width of field is 1.6 mm. Thin section image.

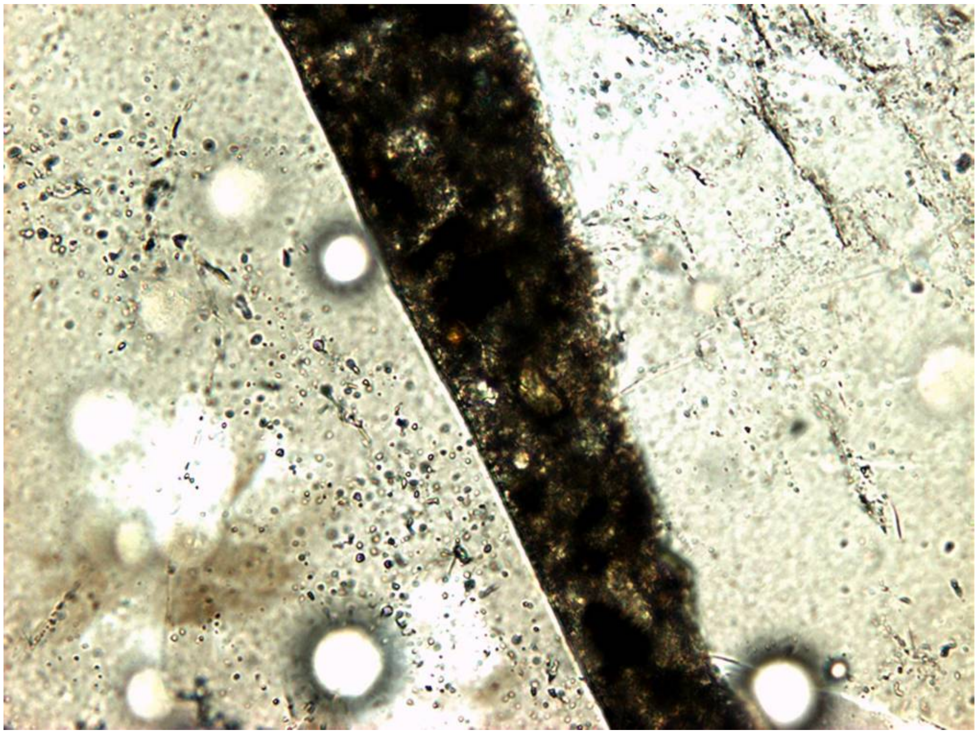


Figure O1-9: Paste/coarse aggregate interfacial region. Width of field is 0.33 mm. Polished surface image.

Figure O1-10: Paste/fine aggregate interfacial region. Width of field is 2.0 mm. Thin section image.

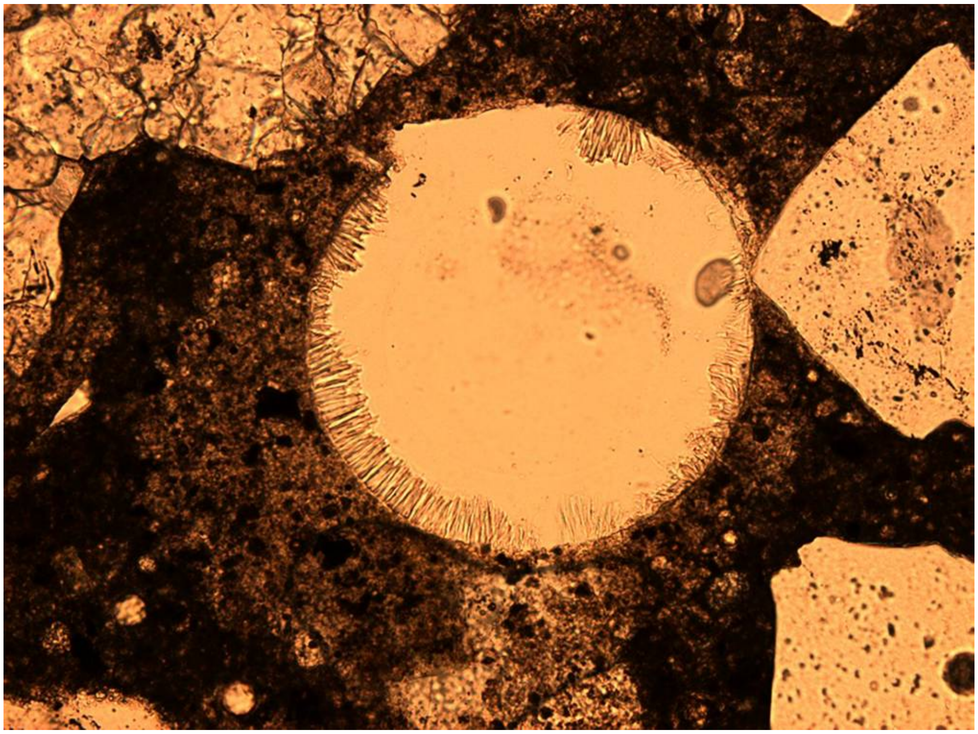


Figure O1-11: Ettringite crystals in an air void. Width of field is 0.65 mm. Thin section image.

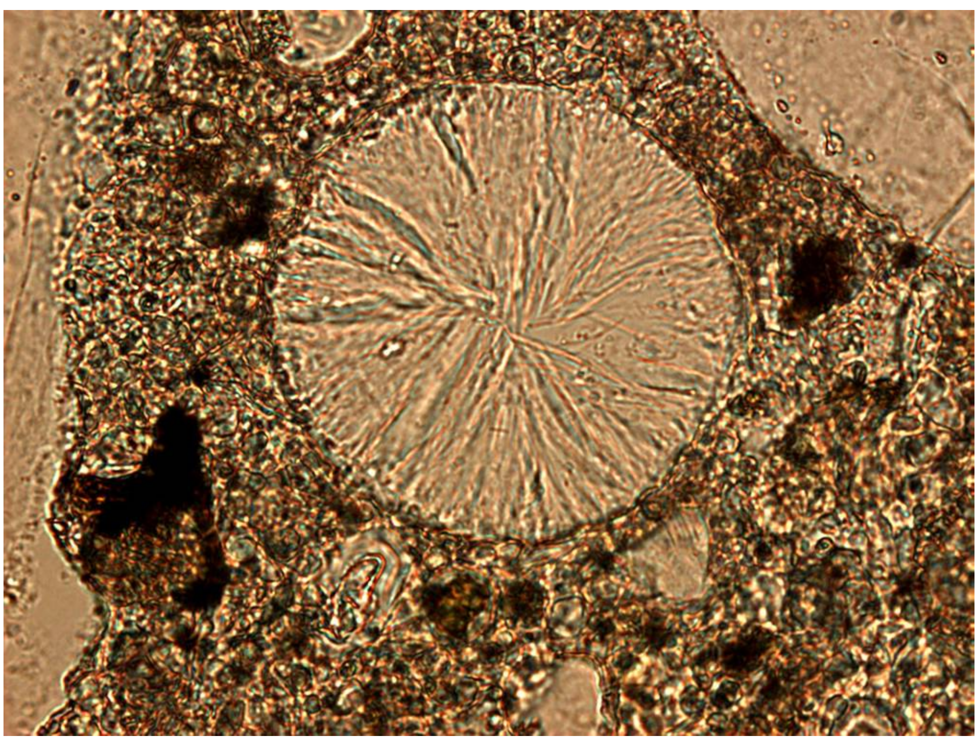


Figure O1-12: A void fully filled with ettringite. Width of field is 0.65mm. Thin section image.

APPENDIX O – Supplement 2

SH 249 over Louetta Road, Houston, Texas
Survey Checklist

Checklist

The following checklist is adapted from 201.1 R-92, and provided to facilitate a thorough survey. The definition of terms and associated photographs in 201.1 R-92, are utilized to standardize the reporting of the condition of the concrete in the structures.

1. Description of structure or pavement
    - 1.4 Name, location, type, and size
      The Tomball Parkway (S.H. 249) Bridge over Louetta Road in Houston, Texas consists of two separate structures. One carrying three lanes of northbound traffic and the other three lanes of southbound traffic with an additional exit ramp. The overpass bridge is 391-feet long and consists of three spans for each direction.
    - 1.2 Owner, project engineer, contractor, when built
      Owner-Texas Department of Transportation. This bridge is part of a demonstration project for HPC in bridge structures which were co-sponsored by the Federal Highway Administration (FHWA) and the Texas Department of Transportation (TxDOT). The bridge was constructed in 1996 and opened to traffic in June 1998. The contractor was William Brothers Inc. at Houston.
    - 1.3 Design
        - 1.3.1 Architect and/or engineer: The Texas Department of Transportation (TxDOT)
        - 1.3.2 Intended use and history of use: To carry traffic over the Louetta Road and improve the State Highway 249 from a non-freeway facility to a freeway facility. Opened to traffic in June 1998.
        - 1.3.3 Special features: Both structures consist of pre-cast Texas U54-beams. The structures are intended to be compared for relative durability and performance based on the extensive use of HS/HPC in the southbound structure and the use of normal strength HPC in the northbound structure.
    - 1.4 Construction
        - 1.4.1 Contractor-general William Brothers Inc. at Huston TX
        - 1.4.2 Subcontractors concrete placement: N/A
        - 1.4.3 Concrete supplier: Lopez-Gloria Construction Services of Houston
        - 1.4.4 Agency responsible for testing: N/A
        - 1.4.5 Other subcontractors: N/A
    - 1.5 Photographs
        - 1.5.1 General view Photos 1 through 4
        - 1.5.2 Detailed close up of condition of area Photos 5 through 10
    - 1.20 Sketch map-orientation showing sunny and shady and well and poorly drained regions N/A

2. Present condition of structure Date of Evaluation The week of August 25, 2003
    - 2.1 Overall alignment of structure No signs of misalignment

2.1.1 Settlement
2.1.2 Deflection
2.1.3 Expansion
2.1.4 Contraction
2.2 Portions showing distress (beams, columns, pavement, walls, etc., subjected to strains and pressures) N/A
2.3 Surface condition of concrete
2.3.1 General (good, satisfactory, poor, dusting, chalking, blisters) Good
2.3.2 Cracks Transverse, Diagonal, and Longitudinal
2.3.2.1 Location and frequency See Figure 2 and Figure 3
2.3.2.28 Type and size (see Definitions) See Figure 2 and Figure 3

Longitudinal Along the U-beam and panel boundaries
Width (from Crack comparator) Less than 0.03 in.
Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)

TransverseAt the U-beam diaphragm and panel boundaries
Width (from Crack comparator) Less than 0.03 in..
Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)

Craze N/A
Width (from Crack comparator) ____________
Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)

Map N/A
Width (from Crack comparator) _______________
Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)

D-Cracking N/A
Width (from Crack comparator) ____________
Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)

Diagonal In the acute corners and near the joints
Width (from Crack comparator) Less than 0.03 in.
Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)

Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)

2.3.2.29 Leaching, stalactites N/A

2.3.3 Scaling N/A
- 2.3.3.1 Area, depth ____
- 2.3.3.15 Type (see Definitions) ____
  - Light (Less than 1/8 in.)
  - Medium (1/8 in. – 3/8 in.)
  - Severe (3/8 in. – 3/4 in.)
  - Very Severe (Greater than 3/4 in.)

2.3.4 Spalls and popouts Minor, associated with wider cracks
- 2.3.4.1 Number, size, and depth ____
- 2.3.4.15 Type (see Definitions) ____
  - Spalls
    - Small (Less than 3/4 in. depth)
    - Large (Greater than 3/4 in. depth)
  - Popouts
    - Small (Less than 3/8 in. diameter)
    - Medium (3/8 in. – 2 in. diameter)
    - Large (Greater than 2 in. diameter)

2.3.5 Extent of corrosion or chemical attack, abrasion, impact, cavitation N/A

2.3.6 Stains, efflorescence Efflorescence along a few cracks on the underside of the deck panels

2.3.7 Exposed reinforcement One location, see Figure 3

2.3.8 Curling and warping N/A

2.3.9 Previous patching or other repair N/A

2.3.10 Surface coatings N/A
- 2.3.10.1 Type and thickness N/A
- 2.3.10.2 Bond to concrete N/A
- 2.3.10.3 Condition N/A

2.3.11 Abrasion N/A

2.3.12 Penetrating sealers ___
- 2.3.12.1 Type N/A
- 2.3.12.2 Effectiveness N/A
- 2.3.12.16 Discoloration N/A

2.4 Interior condition of concrete (in situ and samples) N/A
- 2.4.1 Strength of cores
- 2.4.2 Density of cores
- 2.4.3 Moisture content
- 2.4.4 Evidence of alkali-aggregate or other reactions N/A
- 2.4.5 Bond to aggregate, reinforcing steel, joints N/A
- 2.4.6 Pulse velocity
- 2.4.7 Volume change
- 2.4.8 Air content and distribution
- 2.4.9 Chloride-ion content

2.4.10 Cover over reinforcing steel
2.4.11 Half-cell potential to reinforcing steel.
2.4.12 Evidence of reinforcement corrosion
2.4.13 Evidence of corrosion of dissimilar metals
2.4.27 Delaminations ____________ N/A
2.4.27.1 Location ____________ N/A
2.4.27.2 Number, and size ____________ N/A
2.4.15 Depth of carbonation
2.4.16 Freezing and thawing distress (frost damage)
2.4.17 Extent of deterioration
2.4.31 Aggregate proportioning, and distribution

3. Nature of loading and detrimental elements
3.1 Exposure
3.1.1 Environment (arid, subtropical, marine, freshwater, industrial, etc.) N/A
3.1.2 Weather-(July and January mean temperatures, ___________
mean annual rainfall and -
months in which 60 percent of it occurs) ________
3.1.3 Freezing and thawing _______
3.1.4 Wetting and drying Minimal annual exposure
3.1.12 Drying under dry atmosphere ____________ N/A
3.1.6 Chemical attack-sulfates, acids, chloride ____________ N/A
3.1.7 Abrasion, erosion, cavitation, impact ____________ N/A
3.1.8 Electric currents ____________ N/A
3.1.9 Deicing chemicals which contain chloride ions ____________ N/A
3.1.10 Heat from adjacent sources ____________ N/A
3.2 Drainage ____________ N/A
3.2.1 Flashing ____________________
3.2.2 Weepholes ____________________
3.2.3 Contour ____________________
3.2.4 Elevation of drains ____________________
3.3 Loading Research Test Data Available in Compilation CD Version 3
3.3.1 Dead ____________________
3.3.2 Live ____________________
3.3.3 Impact ____________________
3.3.4 Vibration ____________________
3.3.5 Traffic index ____________________
3.3.6 Other ____________________
3.4 Soils (foundation conditions) ____________________
3.4.1 Compressibility
3.4.2 Expansive soil
3.4.3 Settlement
3.4.4 Resistivity
3.4.5 Evidence of pumping
3.4.6 Water table (level and fluctuations)

4.	Original condition of structure	Good
4.1	Condition of formed and finished surfaces	Good
4.1.1	Smoothness	
4.1.2	Air pockets ("bugholes")	
4.1.3	Sand streaks	
4.1.4	Honeycomb	
4.1.5	Soft areas (retarded hydration)	
4.1.6	Cold joints	
4.1.33	Staining	
4.1.34	Sand pockets	
4.2	Defects	N/A
4.2.1	Cracking	
4.2.1.1	Plastic shrinkage	
4.2.1.2	Thermal shrinkage	
4.2.1.3	Drying shrinkage	
4.2.15	Curling	
5.	Materials of Construction	See Table 2
6.	Construction Practices	See Report pg. 9

APPENDIX P

U.S. Route 67 Bridge, San Angelo, Texas

HIGH PERFORMANCE CONCRETE BRIDGE DECK INVESTIGATION

U.S. Route 67 Bridge San Angelo, Texas

I. BACKGROUND

The U.S. Route 67 Bridge in San Angelo, Texas is part of a high-speed expressway and carries traffic over the North Concho River, U.S. Route 87, and South Orient Railroad tracks. It was constructed in 1997 and opened to traffic in January 1998. The bridge consists of two separate structures, one carrying two lanes of eastbound traffic and the other two lanes of westbound traffic (see photos 1 and 2). Both structures consist of prestressed concrete I-beam girders covered with precast concrete deck panels (4-inches thick × 8-feet long), which in turn are covered with 3.5-inches of cast-in-place concrete. The substructures consist of concrete columns, concrete bent caps, and concrete abutments at each end.

The eastbound structure is 950-feet long and consists of eight spans (see photo 3). Spans 7 and 8 are skewed to accommodate the railroad tracks. The bridge decks at Spans 1 through 4 are 38-feet wide. The decks progressively widen in Spans 5, 6, 7, and 8 to accommodate an exit-ramp at the eastern end of the eastbound bridge. Except for the girders in Spans 6 through 8, high performance concrete (HPC) was used for all girders, deck panels, and cast-in-place concrete in the eastbound structure.

The westbound structure is 960-feet long and consists of nine spans (see photo 4). Spans 7, 8, and 9 are skewed to accommodate the railroad tracks. The bridge deck at Span 1 is 38-feet wide, and the decks progressively widen in Spans 2 through 9 to accommodate an on-ramp at the eastern end of the westbound bridge. At the time of the inspection, traffic lanes on the western half of the westbound bridge merged down to a single lane to accommodate original construction of the expressway west of the bridge. In the westbound structure, HPC was used only for the cast-in-place decks of Spans 1 through 5.

This bridge is part of a demonstration project for HPC in bridge structures which was co-sponsored by the Federal Highway Administration (FHWA) and the Texas Department of Transportation (TXDOT). It is evident that the structures were intended to be compared for relative durability and performance based on the extensive use of HPC in the eastbound structure and the limited use of HPC in the westbound structure. Span 1 of both structures is approximately of the same length and width, but the eastbound span uses 4 girders while the westbound span uses 7 girders. The eastbound structure has only 8 spans, while the westbound structure has 9 spans.

II. SCOPE OF SERVICES

Professional Service Industries Inc. (PSI) is under contract with the Federal Highway Administration (FHWA) to conduct bridge deck inspections for HPC bridges. Our scope of services on this bridge included a series of tasks and sub-tasks, which are described as follows:

- Collect all available information relevant to the construction of each bridge, including;
  - Deck Concrete Properties
  - Specified Deck Concrete Construction Procedures
  - Approved Concrete Mixture Proportions
  - Measured Properties from QC
  - Other Measured Properties
  - Actual Method of Deck Placement
  - Average Daily Traffic (ADT)
  - Exposure Condition of the Bridge
  - Any Performed Maintenance
  - Any Inspection Reports
- Visually inspect the bridge, and obtain the following:
  - General condition of the deck top surface
  - Determination of the maximum crack width
  - General condition of the deck underside
  - General condition of the girders
  - Photograph areas of significant deterioration
  - Prepare drawings locating defects
  - Extract 6 concrete core samples

III. COMPILATION OF BRIDGE CONSTRUCTION INFORMATION

Information sources used for this report include bridge drawings, inspection reports, bridge summary data sheets, and technical information contained in FHWA's "Compilation and Evaluation of Results from High Performance Concrete Bridge Projects" version 3.

Deck Concrete Properties

Three classes of HPC were specified for use in the decks of the San Angelo Bridge (Class H, Class K, and Class S). Class H (HPC) was used in the precast panels of the eastbound bridge, whereas Class K (HPC) was used in the cast-in-place deck of this bridge. As mentioned earlier, the use of HPC was limited in the westbound bridge. Class S (HPC) was used in the cast-in-place decks of Spans 1 through 5. The other elements of this bridge were constructed with normal concrete. Table 1 lists the classes of concrete used on the San Angelo Bridge. Table 2 lists the specified properties for each class of HPC.

Table 1: Classes of Concrete Used in the Construction of San Angelo Bridge

Direction	Span	Component		
		Precast Girders	Precast Deck Panels	CIP Deck
Eastbound	1-5	H (HPC)	H (HPC)	K (HPC)
	6-8	H	H (HPC)	K (HPC)
Westbound	1-5	H	H	S (HPC)
	6-9	H	H	S

TABLE 2: Specified Concrete Properties

Property	Class H (HPC)	Class K (HPC)	Class S (HPC)
Minimum Cementitious Materials Content:	564 lb/yd^3	611 lb/yd^3	611 lb/yd^3
Max. Water/Cementitious Materials Ratio:	0.49	0.44	0.44
Min. Percentage of Fly Ash:	20 with potentially reactive aggregates		
Max. Percentage of Fly Ash:	35, or 0 with Type IP and white portland cement		
Min. Percentage of GGBFS:	--	--	--
Max. Percentage of GGBFS:	50, or 0 with Type IP cement		
Maximum Aggregate Size:	1½ in.	1½ in.	¾ in.
Slump:	Not Specified	3-9 in.	3-4 in.
Air Content:	--	6	6
Design Compressive Strength:	6000 psi @ 28 days (1)	6000 psi @ 28 days (2)	4000 psi @ 28 days (3)
Chloride Permeability (AASHTO T 277):	Guideline of 1500 coulombs at 56 days		
ASR or DEF Prevention:	Min. fly ash content of 20% with potentially reactive aggregates.		

(1) Value applies to precast deck panels of the eastbound bridge.
(2) Value applies to cast-in-place deck of the eastbound bridge.
(3) Value applies to cast-in-place deck of the westbound bridge, Spans 1 to 5.

Specified Deck Concrete Construction Procedures

The decks of the San Angelo bridge are composite decks consisting of 4-in. precast prestressed deck panels and a topping layer of 3.5-in. thick cast-in-place reinforced concrete. A reinforcing bar with new deformation pattern was used in the cast-in-place decks of Spans 2 and 3 of both eastbound and westbound bridges. On the eastbound bridge, these special bars were uncoated, whereas on the westbound bridge, they were epoxy coated. The new reinforcing bar was developed by the University of Kansas and was reported to have improved bond characteristics. Regular uncoated reinforcing steel was used in Span 1 of both the eastbound and westbound bridges and Spans 2 through 8 of the eastbound bridge. Regular epoxy coated reinforcing steel was used in Spans 2 through 9 of the westbound bridge.

In the cast-in-place deck a wet curing for 10 days was specified when fly ash was used and 8 days when no fly ash was used. Concrete cylinder, 4 × 8 inch in dimensions and cured according to AASHTO T 23, were specified for quality control testing.

Approved Concrete Mixture Proportions

Precast Deck Panel

Class H (HPC) was used in the precast deck panels of the eastbound bridge. The precast panels of the westbound bridge were constructed with normal class H concrete. The approved proportions for these mixtures are shown in Table 3.

TABLE 3: Approved Mixture Proportions for Precast Panels

Mixture Parameters	Class H (HPC) Eastbound Deck Panels	Class H Westbound Deck Panels
Cement Brand:	Capitol	Capitol
Cement Type:	III	III
Cement Quantity:	658 lb/yd^3	564 lb/yd^3
Fine Aggregate Type:	River Sand	River Sand
Fine Aggregate FM:	2.63	2.63
Fine Aggregate SG:	Not available	Not available
Fine Aggregate Quantity:	1355 lb/yd^3	1457 lb/yd^3
Coarse Aggregate, Max. Size:	1-in.	1-in.
Coarse Aggregate Type:	No. 5 Crushed Limestone	No. 5 Crushed Limestone
Coarse Aggregate Quantity:	1844 lb/yd^3	1889 lb/yd^3
Water:	251 lb/yd^3	275 lb/yd^3
Water Reducer Type:	D	D
Water Reducer Quantity:	300 fl oz/yd^3	257 fl oz/yd^3
Retarder Type:	D	D
Retarder Quantity:	79 fl oz/yd^3	49 fl oz/yd^3
Air Entrainment Quantity:	None	None
Water/Cementitious Materials Ratio:	0.38	0.49

Measured properties of approved concrete mixtures for the precast deck panels are summarized in Table 4.

TABLE 4: Measured Properties of Approved Concrete Mixtures for Precast Deck Panels

Measured Concrete Properties	Class H (HPC) Eastbound Deck Panels	Class H Westbound Deck Panels
Slump:	5-6 in.	6-7 in.
Air Content:	1.5%	1.5%
Unit Weight	150.9 lb/ft^3	150.7 lb/ft^3
Rapid Chloride Permeability (AASHTO T 277):	1980 coulombs at 56 days	3230 coulombs at 56 days

The properties of the cement used in the deck panels are shown in Table 5.

TABLE 5: Properties of the Cement used in the Construction of Deck Panels

Chemical Composition, %		
SiO_2		19.66
Al_2O_2		5.38
Fe_2O_3		2.06
CaO		64.05
MgO		1.26
SO_3		4.09
Loss of Ignition		2.64
Insoluble Residue		0.27
Free Lime		N/A
C_3S		60.58
C_3A		10.77
Total Alkali		0.60
Specific Surface, cm^2/gm		
Blaine		5730
Wagner		2926
% Passing No. 325 Sieve		98.6
Compressive Strength, psi		
1 Day		4545
3 Day		5910
7 Day		6750
28 Day		N/A
Setting Time, min		
Vicat	Initial	75
	Final	120
Gilmore	Initial	135
	Final	255

Cast-in-Place Deck

Class K (HPC) was used in the cast-in-place deck of all the spans of the eastbound bridge. The HPC used in limited numbers of spans of the westbound bridge (Span 1 through 5) was of Class S. The remaining spans of the westbound bridge (Spans 6 through 9) utilized normal Class S concrete. The approved proportions for these mixtures are shown in Table 6.

TABLE 6: Approved Mixture Proportions for Cast-in-Place Deck

Mixture Parameters	Class K (HPC) Eastbound CIP Deck	Class S (HPC) Westbound CIP Deck (Spans 1-5)	Class S Westbound CIP Deck (Spans 6-9)
Cement Brand:	Lonestar	Lonestar	Lonestar
Cement Type:	II	II	II
Cement Quantity:	490 lb/yd^3	427 lb/yd^3	611 lb/yd^3
Fly Ash Type:	Class C	Class C	None
Fly Ash Quantity:	210 lb/yd^3	184 lb/yd^3	---
Fine Aggregate Type:	River Sand	River Sand	River Sand
Fine Aggregate FM:	2.70	2.70	2.70
Fine Aggregate Quantity:	1365 lb/yd^3	1240 lb/yd^3	1243 lb/yd^3
Coarse Aggregate, Max. Size:	1¼ in.	1¼ in.	1¼ in.
Coarse Aggregate Type:	No. 5 Crushed River Gravel	No. 5 Crushed River Gravel	No. 5 Crushed River Gravel
Coarse Aggregate Quantity:	1900 lb/yd^3	1856 lb/yd^3	1856 lb/yd^3
Water:	219 lb/yd^3	258 lb/yd^3	258 lb/yd^3
High Range Water Reducer Type:	F	---	---
HRWR Quantity:	156 fl oz/yd^3	---	---
Retarder Brand:	---	Plastocrete 161R	Plastocrete 161R
Retarder Type:	B and D	B and D	B and D
Retarder Quantity:	28 fl oz/yd^3	26 fl oz/yd^3	26 fl oz/yd^3
Air Entrainment Quantity:	3.1 fl oz/yd^3	3.1 fl oz/yd^3	3.1 fl oz/yd^3
Water/Cementitious Materials Ratio:	0.31	0.42	0.42

Measured properties of approved concrete mixtures for the cast-in-place decks are summarized in Table 7.

TABLE 7: Measured Properties of Approved Concrete Mixtures for Cast-in-Place Decks

Measured Concrete Properties	Class K (HPC) Eastbound CIP Deck	Class S (HPC) Westbound CIP Deck (Spans 1-5)	Class S Westbound CIP Deck (Spans 6-9)
Slump:	7-9 in.	3-4 in.	3-4 in.
Air Content:	6%	6%	6%
Un it Weight	149.4 lb/ft^3	145.3 lb/ft^3	145.6 lb/ft^3
Rapid Chloride Permeability (AASHTO T 277):	690 coulombs at 56 days	1380 coulombs at 56 days	2490 coulombs at 56 days

Measured Properties from QC Tests of Production Concrete

Precast Deck Panels

The measured properties from QC tests of class H (HPC) production concrete used in the precast deck panels of the eastbound bridge are shown in Table 8.

TABLE 8: Measured Properties from QC Tests of Class H (HPC) Production Concrete

Date Cast	Release		
	Slump (inches)	Age (hours)	Compressive Strength (psi)
8/16/96	8	14	4600
8/16/96	—	21	5590
8/19/96	4-1/2	19	5560
8/19/96	4	15-1/2	5080
8/20/96	3-1/2	20-1/4	5270
8/20/96	3-1/2	17	4960
8/20/96	5	16	4880
8/20/96	2-1/2	20	5450
8/22/96	7	20	5160
8/22/96	4	17	5560
8/23/96	4	20	4880
8/29/96	—	23	4790
9/4/96	4-1/4	24	4010
9/10/96	4	20	4570

NOTES: No information was available on the curing condition of the cylinders used in the above testing.

Cast-in-Place Deck

The measured properties from QC tests of Class K (HPC) production concrete used in the cast-in-place deck of the eastbound bridge are shown in Table 9.

TABLE 9: Measured Properties from QC Tests of Class K (HPC) Production Concrete

Date Cast	Eastbound Span No.	Slump (in.)	Concrete Temp. (°F)	Air Content (%)	Compressive Strength (psi)		
					5 days	7 days	28 days
6/12/97	1	5-1/2	77	7	5181 (1)	5373	6680
		5	78	6.2	6103 (1)	6991	7358
6/25/97	2	7-1/2	80	6.0	4755	5815	--
		8	75	6.2	5986	5657	--
7/9/97	3	8	80	6.2	6216	5792	--
		7	88	6.2	5976	6357	--
7/23/97	4	8	81	6.0	6481	6056	8180
		8	83	6.3	5779	6245	7454
7/26/97	5	7-3/4	84	6.8	5746 (1)	6100	7269
		7-1/2	83	6.4	5991 (1)	6128	--
8/19/97	6	8-1/4	80	6.6	5924 (2)	5972	--
		8-1/2	81	7.2	6386 (2)	5735	--
8/28/97	7	8	78	6.0	6253	6540	--
		8-1/4	82	6.0	5506	5804	--
9/4/97	8	7	80	6.0	4286 (1)	6247	7128
		6-1/4	82	5.2	5418 (1)	6049	--

(1) Tested at age 4 days.
(2) Tested at age 6 days.
NOTES: Concrete received a wet mat cure for 10 days. Test cylinders received AASHTO T 23 initial and standard curing. The unit weight of concrete was 149 lbs/ft^3.

Limited amount of information was available on measured properties from QC tests of Class S (HPC) production concrete used in Spans 1 through 5 of the westbound bridge. It was reported that the unit weight of the concrete was 145 lb/ft^3 and the concrete had a compressive strength of 6,120 psi at 28 days.

Measured Properties from Research Tests of Production Concrete

Precast Deck Panels

The compressive strength, modulus of elasticity, and coefficient of thermal expansion of class H (HPC) production concrete used in the eastbound bridge and Class H normal concrete used in the westbound bridge are shown in Table 10.

TABLE 10: Measured Properties from Research Tests of Production Concrete Used in the Precast Deck Panel

Bridge	Compressive Strength (1), psi		Modulus of Elasticity (1), ksi		Coefficient of Thermal Expansion (2), mill/ °F
	Release at 24 Hours	56 days HPC 28 days non-HPC	Release at 24 Hours	56 days HPC 28 days non-HPC	
Eastbound HPC	3140	10,100	2990	4620	4.7
Westbound non HPC	5310	8250	3990	4680	4.6

(1) Averaged values for all instrumented panels cast 2/5/97 for eastbound and 9/4/96 for westbound.
(2) Average of two increasing and two decreasing values between 40 and 20 °F at 60% relative humidity.
NOTES: All 4 × 8-in cylinders cured alongside the panels before and after release and tested using neoprene pads and steel caps.

The creep and shrinkage data for Class H (HPC) production concrete used in the eastbound bridge and Class H normal production concrete used in westbound bridge is shown in Table 11. All 4 × 20-in cylinders for the creep and shrinkage measurement were stored alongside the beams for 8 to 18 hours, stripped at approximately 24 hours after casting and loaded at an age of 2 days to 20 and 40% of the nominal design compressive strength of the mixture. Temperature and humidity were not controlled. Average relative humidity was 55 percent.

TABLE 11: Creep and Shrinkage Properties from Research Tests of Production Concrete Used in the Precast Deck

Days after Loading	Creep Coefficient (1)		Specific Creep (2) Millionths / psi		Shrinkage (2) millionths	
	Eastbound Class H (HPC)	Westbound Class H	Eastbound Class H (HPC)	Westbound Class H	Eastbound Class H (HPC)	Westbound Class H
7	0.58	0.74	0.133	0.168	135	249
28	1.12	1.07	0.257	0.244	330	360
56	1.41	1.37	0.324	0.310	404	387
180	1.95	1.97	0.445	0.444	528	428

(1) Reported creep values are the average values for specimens loaded to the 20 and 40 percent levels. Nine readings were taken on each specimen.
(2) Shrinkage values include adjustments for one day of drying before initial readings were taken and for length changes caused by variation in concrete temperatures.

Cast-in-Place Decks

The compressive strength, modulus of elasticity, and coefficient of thermal expansion of Class K (HPC) production concrete used in the eastbound cast-in-place deck, Class S (HPC) used in Spans 1 through 5 of the westbound bridge, and Class S normal concrete used in the Spans 6 and 7 of the westbound bridge are shown in Table 12.

TABLE 12: Measured Properties from Research Tests of Production Concrete Used in the Cast-In-Place Deck

	Compressive Strength [(1)], psi	Modulus of Elasticity, ksi	Coefficient of Thermal Expansion [(2)], mill/ °F
Eastbound - Class K (HPC)			
1	7290	5500	4.6
2	8420	5230	
3	9060	6060	
4	7550	5790	
5	8220	5010	
6	8680	5380	
7	7460	4920	
8	7770	5570	
Westbound - Class S (HPC)			
1	6400	5170	4.4
2	5160	4670	
3	4450	4310	
4	4700	4670	
5	4560	4640	
Westbound - Class S			
6 and 7	5340	4930	4.9

(1) At 56 days for HPC mixtures. At 28 days for non-HPC mixtures.

(2) Average of two increasing and two decreasing values between 40 and 20 °F at 60% relative humidity.

The creep and shrinkage data for Class K (HPC) production concrete used in the cast-in-place decks of the eastbound bridge, Class S (HPC) production concrete used in the cast-in-place decks of Spans 1 through 5 of the westbound bridge, and Class S normal concrete used in Spans 6 through 9 of the westbound bridge are shown in Table 13.

TABLE 13: Measured Creep and Shrinkage Properties from Research Tests of Production Concrete Used in the Cast-In-Place Deck

Days after Loading	Creep Coefficient [(1)]			Specific Creep [(1)], millionths/psi			Shrinkage [(2)], millionths		
	Eastbound Class K (HPC)	Westbound		Eastbound Class K (HPC)	Westbound		Eastbound Class K (HPC)	Westbound	
		Class S (HPC)	Class S		Class S (HPC)	Class S		Class S (HPC)	Class S
7	0.72	0.65	0.53	0.108	0.212	0.106	138	125	118
28	1.07	1.21	0.94	0.161	0.390	0.186	251	269	258
56	1.25	1.51	1.43	0.188	0.488	0.284	285	371	340
180	1.59	2.23	1.96	0.240	0.722	0.389	265	462	434

(1) Reported creep values are the average values for specimens loaded to the 20 and 40 percent levels. Nine readings were taken on each specimen.

(2) Shrinkage values include adjustments for one day of drying before initial readings were taken and for length changes caused by variation in concrete temperatures.

The chloride ion penetration data for Class K (HPC) production concrete used in the cast-in-place deck of the eastbound bridge, Class S (HPC) production concrete used in the cast-in-place deck of Spans 1 through 5 of the westbound bridge, and Class S normal concrete used in the Spans 6 through 9 of the westbound bridge are shown in Table 14.

TABLE 14: Chloride Ion Penetration (AASHTO T 259) Data for Production Concrete Used in the Cast-in-Place Deck

Concrete Class	K (HPC)	K (HPC)	S (HPC)	S
Casting Date	6/12/97	8/19/97	12/3/96	2/15/97
Depth, in				
0.25	0.201	0.156	0.269	0.368
0.75	0.000	0.000	0.000	0.058
1.25	0.000	0.000	0.000	0.000
Integral Chloride	0.56	0.44	0.85	1.17

NOTES: All specimens were moist cured. Values are chloride percentage by weight of concrete. Specimens were moist cured for 14 days followed by 42 days drying and ponding for 90 days.

For Class K (HPC) used in the cast-in-place decks of the eastbound bridge, a linear relationship was established between the splitting tensile strength and the square root of compressive strength (Equation 1) for both ASTM moist cured cylinder specimens and the site/ field test specimens.

$$f_{sp} = 8.80\sqrt{f_c'} \qquad \text{Eqn. (1)}$$

where f_c' is the compressive strength, and f_{sp} is the splitting tensile strength.

The Class K (HPC) used in the cast-in-place decks of the eastbound bridge exhibited very low to low Coulombs value, in the range of 250 Coulombs to 1,500 Coulombs. When tested for freeze-thaw resistance utilizing the ASTM C 666, the Class K (HPC) used in the cast-in-place decks of the eastbound bridge experienced a maximum weight loss of 1.30% after 300 freeze-thaw cycles. As a comparison, the Class S normal concrete used in the cast-in-place decks of Spans 6 through 9 of the westbound bridge experienced a maximum weight loss of 3.64% after 300 freeze-thaw cycles.

Actual Method of Deck Placement

Construction of the deck occurred in June 1997, with the concrete for the deck pumped from a truck or placed via a concrete bucket. Concrete was distributed by a mechanical spreader. A final troweled finish was applied followed by tining for enhanced skid resistance. Surface finishing consisted of motorized screed pan with a burlap drag. Prior to placement of the cast-in-place deck, the precast panels were saturated to prevent the loss of mixing water while the concrete was in plastic state. The concrete was internally vibrated to provide proper consolidation and avoid internal segregation. Fogging of the concrete deck started when the concrete was in the plastic state. A curing compound was applied in addition to the continuous fogging. This procedure avoided the surface moisture evaporation and plastic shrinkage cracks. The deck was cured using wet mat for

10 days when fly ash was used and 8 days when fly ash was not used. The wet mats were kept moist for 10 days after casting for the HPC decks with pozzolans.

Average Daily Traffic (ADT)

The district identifies this bridge as carrying 10,000 vehicles per day. This value appeared reasonable based on a cursory and subjective estimation of traffic flow made while the PSI inspection crew was on-site.

Exposure Condition of the Bridge

The area surrounding the bridge is developed with mixed residential and commercial land use. The National Weather Service reports that the normal maximum temperature varies between 94.4°F in July and 57.9°F in January. The normal minimum temperature varies between 70.4°F in July and 31.8°F in January. The normal precipitation varies between 3.09 inches per month in May to 0.81 inch per month in January with between 4 to 7 days per month where precipitation is greater than 0.01 inches. There are normally 51 days per year when the temperature drops below 32°F. Based on this information, the bridge has minimal annual exposure to wet/dry and freeze/thaw cycles. There was no reported application of deicing salt on this bridge.

Performed Maintenance

No documents were found which would indicate any maintenance had been performed since the bridge was constructed in 1997.

Inspection Reports

A “Bridge Inspection Record”, a “Bridge Inventory Record”, and a “Bridge Appraisal Worksheet” dated November 2002 were obtained from TXDOT. The Bridge Inspection Report mentions “minor cracks in the deck” parallel to Bent 9 at 18 to 24 inches from the control joint in the westbound bridge. These cracks were reported to relate to the deck discontinuity. The inspection report also indicates “the deck overhangs have minor transverse cracks with efflorescence, at the bottom side” of the westbound bridge.

IV. BRIDGE DECK INSPECTION

PSI personnel performed a visual inspection of the bridge decks during the week of July 14, 2003. The results of the inspection are summarized as follows.

General Condition of the Deck Top Surface

Figure 1 shows the general layout of the decks of the eastbound and westbound bridges. Results of our visual inspection of the decks of the two bridges are shown in Figure 2. Surface defects observed and documented during our visual inspection primarily included transverse cracks, longitudinal cracks, and diagonal cracks. All these cracks were hairline cracks with respective maximum widths of 0.014 in., 0.012 in., and 0.010 in. (see photos

5 and 6). The number and length of different type of cracks is summarized in Tables 15 through 20. Signs of abrasion were visible on the decks, particularly in the wheel paths (see Photo 3). However, apparent signs of other serious damages, such as freeze-thaw, D-cracking, alkali-silica section, and alkali-aggregate reaction, were not observed.

A distinct difference was noted in the number and length of cracks on the decks of the two bridges. The westbound bridge had significantly more cracks than the eastbound bridge (see photos 7 and 8). A comparison of the magnitude of cracking on the decks of the two bridges is presented below by crack type.

Transverse Cracks: Spans 1 through 4 of the eastbound bridge had relatively small number of transverse cracks. A total of 50 cracks were observed in these spans with a combined total length of 243.9 ft. Compared to this a total number of 56 cracks with a combined total length of 776.4 ft were observed in spans 1 through 4 of the westbound bridge (see Tables 15 and 16). Since the number, length and width of the spans of the two bridges are variable, a comparison of crack length per deck area is more useful. For Spans 1 through 4, the crack length per deck area for the eastbound and westbound bridges is 0.11 ft/ft^2 and 0.37 ft/ft^2, respectively.

The eastbound bridge deck exhibited a relatively large number of short-length cracks in Spans 5 through 8. The number of cracks in Spans 5 through 9 of the westbound bridge was less but the cracks were of larger length. The crack length per deck area in Spans 5 through 8 of the eastbound bridge was 0.54 ft/ft^2. Compared to this, the crack length per deck area in Spans 5 through 9 of the westbound bridge was 0.62 ft/ft^2. In particular, Span 5 of both of the eastbound and westbound bridges has exhibited large crack densities, 0.615 ft/ft^2 and 0.752 ft/ft^2 respectively.

Considering the total length of the two bridges, the crack length per deck area for the eastbound and westbound bridges was estimated to be 0.31 ft/ft^2 and 0.50 ft/ft^2, respectively.

A pattern which was clearly present in the westbound bridge and absent in the eastbound bridge was that in the westbound bridge deck the cracks appeared to be at the boundaries of the precast deck panels, creating rectangular patterns (see photo 8). Texas DOT reported that this pattern of cracking was observed along Bents 7 and 9 of the westbound bridge soon after the construction was completed. The cracking was attributed to a single pour construction of Spans 6 through 9 of the westbound bridge. A single pour construction was not reported at other locations of the westbound or eastbound bridges.

Longitudinal Cracks: The number and length of longitudinal cracks was significantly less than that of the transverse cracks. However, the westbound bridge decks again had more cracks compared to the eastbound bridge decks. The length per deck area of longitudinal cracks in the eastbound and westbound bridges was estimated to be 0.15 ft/ft^2 and 0.24 ft/ft^2, respectively (see Tables 17 and 18).

Diagonal Cracks: Relatively small number of short-length diagonal cracks was observed in both the eastbound and westbound bridges. These cracks were typically present in the acute corners and near the joints. The crack length per deck area was comparable in the two bridges, 0.009 ft/ft^2 on the eastbound bridge and 0.01 ft/ft^2 on the westbound bridge (see Tables 19 and 20).

TABLE 15: Measured Transverse Cracks on the Surface of Eastbound Bridge Decks

Eastbound Transverse Cracks	Count	Length Range (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span 1	8	2.2 to 6.3	31.9	497	0.064
Span 2	4	3.0 to 11.6	26.1	595	0.044
Span 3	21	1.2 to 12.7	93.3	570	0.164
Span 4	17	1.9 to 25.5	92.6	567	0.163
Span 5	54	1.1 to 36.6	330.4	538	0.615
Span 6	33	1.7 to 19.2	193.7	393	0.493
Span 7	58	1.9 to 30.3	460.3	480	0.958
Span 8	9	2.6 to 6.2	33.3	478	0.070

NOTES: Transverse cracks include cracks oriented parallel to skewed joints.

TABLE 16: Measured Transverse Cracks on the Surface of Westbound Bridge Decks

Westbound Transverse Cracks	Count	Length Range (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span 1	17	1.7 to 33.5	209.8	516	0.407
Span 2	15	2.9 to 39.8	201.8	510	0.396
Span 3	10	3.1 to 35.1	180.4	525	0.344
Span 4	14	2.7 to 41.8	184.4	558	0.331
Span 5	33	2.8 to 29.6	349.6	465	0.752
Span 6	24	3.5 to 39.3	363.8	686	0.530
Span 7	2	6.1 to 10.2	16.2	126	0.129
Span 8	15	1.3 to 13.3	87.4	321	0.273
Span 9	49	2.4 to 39.4	521.4	560	0.932

NOTES: Transverse cracks include cracks oriented parallel to the skewed joints.

TABLE 17: Measured Longitudinal Cracks on the Surface of Eastbound Bridge Decks

Eastbound Longitudinal Cracks	Count	Length Range (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span 1	13	1.3 to 7.8	56.9	497	0.115
Span 2	17	1.1 to 75.6	134.6	595	0.226
Span 3	13	2.2 to 8.6	51.3	570	0.090
Span 4	7	4.4 to 7.1	37.5	567	0.066
Span 5	21	1.9 to 10.3	102.9	538	0.191
Span 6	2	3.8 to 6.6	10.4	393	0.026
Span 7	28	2.2 to 41.5	163.9	480	0.341
Span 8	14	1.9 to 7.8	68.1	478	0.142

NOTES: Longitudinal cracks include cracks that are oriented perpendicular to the skewed joints.

TABLE 18: Measured Longitudinal Cracks on the Surface of Westbound Bridge Decks

Westbound Longitudinal Cracks	Count	Length Range (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span 1	12	2.4 to 5.9	45.4	516	0.088
Span 2	16	1.7 to 37.6	127.6	510	0.250
Span 3	21	1.6 to 14.4	82.3	525	0.157
Span 4	58	0.6 to 25.9	191.6	558	0.343
Span 5	31	1.4 to 8.9	114.9	465	0.247
Span 6	10	1.9 to 55.2	82.0	686	0.120
Span 7	8	2.8 to 6.7	35.7	126	0.283
Span 8	9	1.9 to 31.7	87.8	321	0.274
Span 9	12	2.9 to 109.0	236.9	560	0.423

NOTES: Longitudinal cracks include cracks that are oriented perpendicular to the skewed joints

TABLE 19: Measured Diagonal Cracks on the Surface of Eastbound Bridge Decks

Eastbound Diagonal Cracks	Count	Length Range (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span 1	0	-	-	497	-
Span 2	2	2.7 to 3.7	6.4	595	0.011
Span 3	1	2.1 to 2.1	2.1	570	0.004
Span 4	3	1.7 to 5.0	9.5	567	0.017
Span 5	5	2.3 to 6.6	18.2	538	0.034
Span 6	0	-	-	393	-
Span 7	0	-	-	480	-
Span 8	0	-	-	478	-

TABLE 20: Measured Diagonal Cracks on the Surface of Westbound Bridge Decks

Westbound Diagonal Cracks	Count	Length Range (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span 1	0	-	0.0	516	-
Span 2	3	2.7 to 4.6	10.7	510	0.021
Span 3	4	2.4 to 5.2	14.6	525	0.028
Span 4	2	4.1 to 5.3	9.4	558	0.017
Span 5	0	-	0.0	465	-
Span 6	3	2.9 to 3.4	9.7	686	0.014
Span 7	0	-	0.0	126	-
Span 8	0	-	0.0	321	-
Span 9	0	-	0.0	560	-

Maximum Crack Width

The maximum width of longitudinal cracks and transverse cracks was measured to be 0.012 in. and 0.014 in., respectively. The maximum width of diagonal cracks was measured to be 0.010 in. According to ACI 201.R-92, these crack widths are classified as hairline cracks.

General Condition of the Deck Underside

The underside of the deck, inspected from ground without the aid of any access equipment, exhibited no visible signs of significant distress. However, a few minor transverse cracks along with efflorescence on the bottom side of the deck overhang of the westbound bridge, as noted in earlier inspection reports, were observed (see photo 10).

General Condition of the Girders

The girders were inspected from the ground, without the aide of any access equipment. No visible signs of any significant distress were noted.

Concrete Core Samples

Six cores, approximately 3.5-inches long and 4 inches in diameter, were retrieved from the decks. The core sample locations are shown in Figure 1. The locations were evenly distributed along shoulders of the bridges. The cores were labeled as TX-1 through TX-6 and transferred to FHWA for further analysis.

TABLE 21: Core Dimensions

Sample	TXS-1	TXS-2	TXS-3	TXS-4	TXS-5	TXS-6
Diameter (in.)	3¾	3¾	3¾	3¾	3¾	3¾
Length (in.)	5¼	4¾	5 ½	4¾	5	5

Preliminary Conclusions

Table 22 shows the various combinations of concretes in each span of the eastbound and westbound bridges and corresponding crack density for transverse cracks, longitudinal cracks, and diagonal cracks.

TABLE 22: Summary of Crack Density for Different Bridge Spans

Westbound Bridge

	Span 1	Span 2	Span 3	Span 4	Span 5	Span 6	Span 7	Span 8	Span 9
Precast Girders	Class H								
Precast Deck Panels	Class H								
Cast-in-Place Decks	Class S (HPC)					Class S			
Transverse Crack Density (ft./sq.ft.)	0.407	0.369	0.343	0.331	0.752	0.530	0.128	0.273	0.932
Longitudinal Crack Density (ft./sq.ft.)	0.088	0.250	0.157	0.343	0.247	0.120	0.283	0.274	0.423
Diagonal Crack Density (ft./sq.ft.)	0	0.021	0.028	0.017	0	0.014	0	0	0

Eastbound Bridge

	Span 1	Span 2	Span 3	Span 4	Span 5	Span 6	Span 7	Span 8
Precast Girders	Class H (HPC)					Class H		
Precast Deck Panels	Class H (HPC)							
Cast-in-Place Decks	Class K (HPC)							
Transverse Crack Density (ft./sq.ft.)	0.064	0.044	0.164	0.163	0.615	0.493	0.958	0.070
Longitudinal Crack Density (ft./sq.ft.)	0.115	0.226	0.090	0.066	0.191	0.026	0.341	0.142
Diagonal Crack Density (ft./sq.ft.)	0	0.011	0.004	0.017	0.034	0	0	0

The eastbound bridge where both the precast deck panels and the cast-in-place decks were constructed utilizing HPC has exhibited better performance compared to the westbound bridge where HPC was used only in the cast-in-place decks of a limited number of spans (1 through 5). Comparing cast-in-place deck of Spans 1 through 5 of the westbound bridge with that of the eastbound bridge, the performance of the eastbound cast-in-place deck is found to be superior. This could be attributed to a better quality HPC used in the cast-in-place deck of the eastbound bridge. The Class K (HPC) used in the cast-in-place deck of the eastbound bridge was reported to have a lower water-to-cementitious material ratio compared to the Class S (HPC) used in the cast-in-place deck of the westbound bridge. The water-to-cementitious material ratio of Class K (HPC) was specified as 0.31 compared to 0.42 of Class S (HPC).

It is noted that relatively large number of short-length transverse cracks were observed in Spans 5 through 8 of the eastbound bridge. The consistent cracking pattern appeared to be related to two factors. First, the cracks may be caused by an inadequate moist curing less than that used in Spans 1 through 4. Since the HPC used had a very low water-to-cementitious material ratio and contained 30% fly ash by weight of the total cementitious material content, this concrete needed a very careful attention to moist curing. Second, the eastbound bridge along the southern edge is also the side where the beams have a longer span and larger skew angle. This type of structural configuration makes the southern edge of the eastbound bridge relatively flexible. Combined with the heavy ADT and ADTT, this structural configuration might have contributed to a large number of short-length transverse cracks.

The rectangular pattern cracking particularly observed in Spans 8 and 9 of the westbound bridge may be attributed to a combination of factors. These may include single pour construction of a number of spans, as indicated by Texas DOT, and the higher shrinkage of non-HPC mixture at these locations. The Class S concrete used in the cast-in-place decks of Spans 6 through 9 of the westbound bridge had a cement content of 6.5 sacks/yd^3, without any pozzolans, and had a water-to-cement ratio of 0.42.

Petrographic analysis was performed on the six core samples that were retrieved from the decks of the bridge. Three cores (TXS-1, TXS-2, and TXS-3) were drilled from the eastbound structure, while the other three (TXS-4, TXS-5, and TXS-6) were from the westbound structure. The drilling locations of the cores were illustrated in Figure 1. The collected cores represented the cast-in-place concrete of the bridge decks. The length of the cores ranged from 2 to 3-¾ inches. A crack ran through core TXS-4 in the axial direction, splitting the core equally into two pieces. The rest of the five cores appeared intact, and visual inspection of the concrete cores revealed no further defects.

The coarse aggregate in the concrete was gravel. Coarse aggregate particles were rounded to sub-rounded, and the maximum size measured from the examined cores was about ¾ inch. Preferential orientation of aggregate particles was not observed, nor was segregation. The rock type of this gravel coarse aggregate included limestone, quartz, chert, and quartzite. The fine aggregate was mainly composed of quartz, chert, and limestone, with a small portion of sandstone and quartzite. The fine aggregate was natural sand and the particles appeared rounded to angular.

The cement was reasonably hydrated with respect to the age of the concrete. The cement paste contained some unhydrated cement particles. Fly ash particles were present in the cement paste matrix.

The concrete was air entrained. Small, spherical air voids were observed in the concrete. The air voids were well dispersed in the concrete. No entrapped air voids were observed in the examined concrete core samples. The concrete was well consolidated.

Frequently micro-cracks were observed in cement paste matrix as well as in the paste/aggregate interfacial region. Typically, these cracks meandered along the aggregate peripherals and some extended into the paste. Gap between coarse aggregate particles and paste, although rare, were occasionally found in the concrete.

Occasionally ettringite crystals were observed in some voids in the concrete. Ettringite crystals when present normally filled up a portion of a void. There was no evidence of deterioration associated with the existence of the ettringite in the concrete. Ettringite is commonly seen as a secondary deposit in concrete in service.

Respectfully Submitted,
Professional Service Industries, Inc.
Structural Investigation and Petrography Department

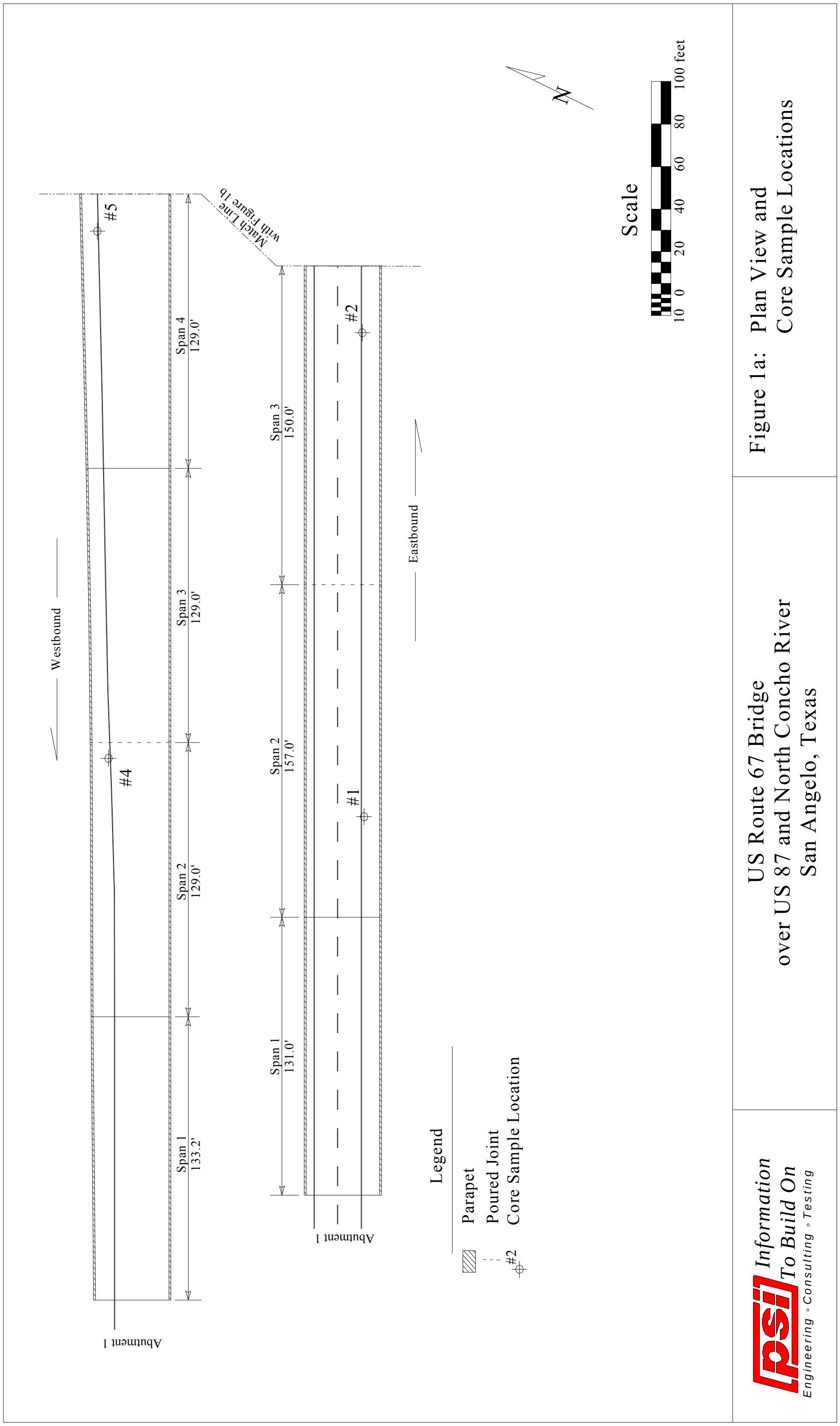
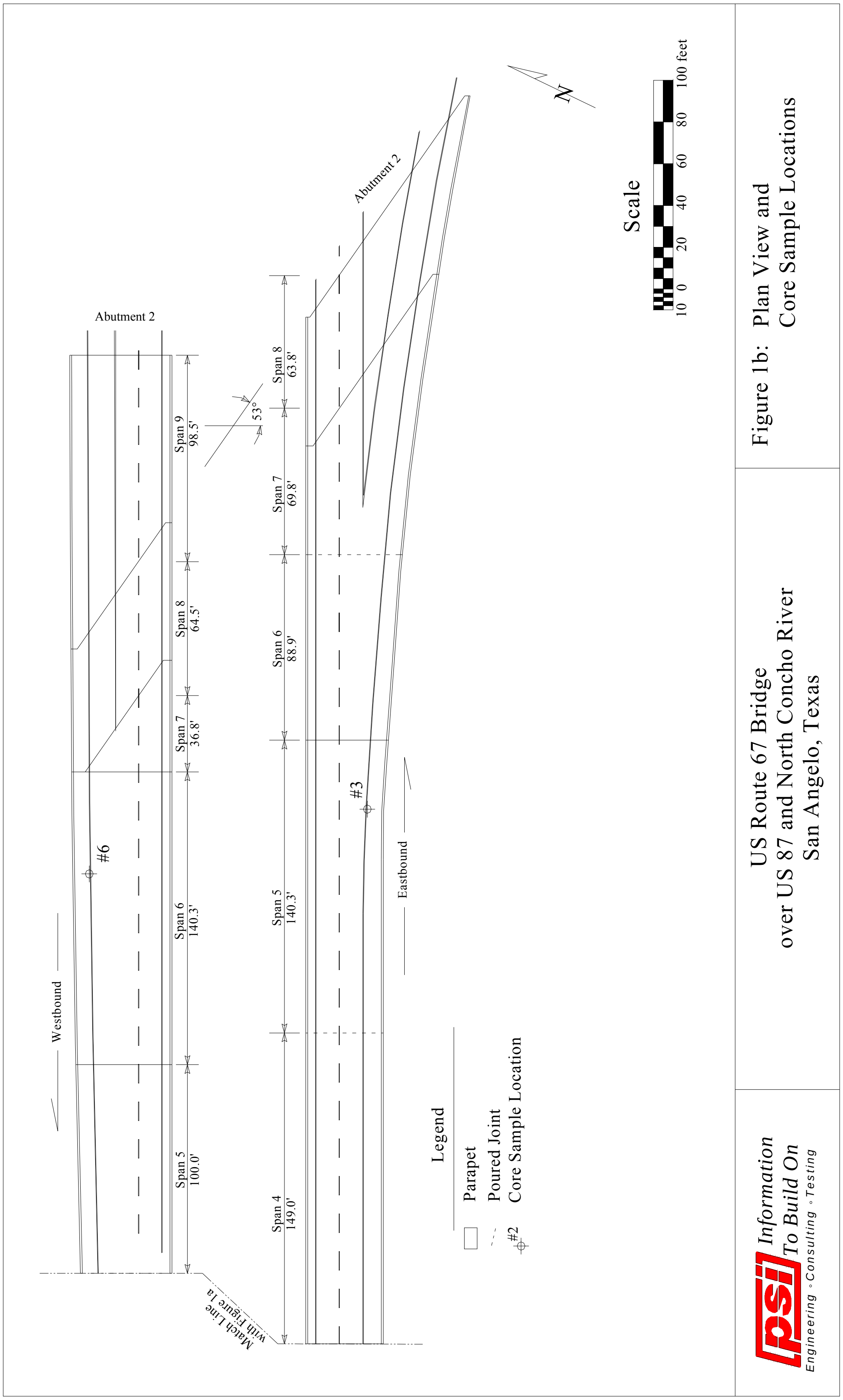


Figure 1a: Plan View and Core Sample Locations



Abutment 2
Westbound
Span 5
100.0'
Span 6
140.3'
Span 7
36.8'
Span 8
64.5'
Span 9
98.5'
#6
53°
Match Line
with Figure 1a
Span 4
149.0'
Span 5
140.3'
Span 6
88.9'
Span 7
69.8'
Span 8
63.8'
#3
Eastbound
Abutment 2
N
Scale
10 0 20 40 60 80 100 feet
Legend
Parapet
Poured Joint
#2
Core Sample Location
Information
To Build On
Engineering • Consulting • Testing
US Route 67 Bridge
over US 87 and North Concho River
San Angelo, Texas
Figure 1b: Plan View and
Core Sample Locations

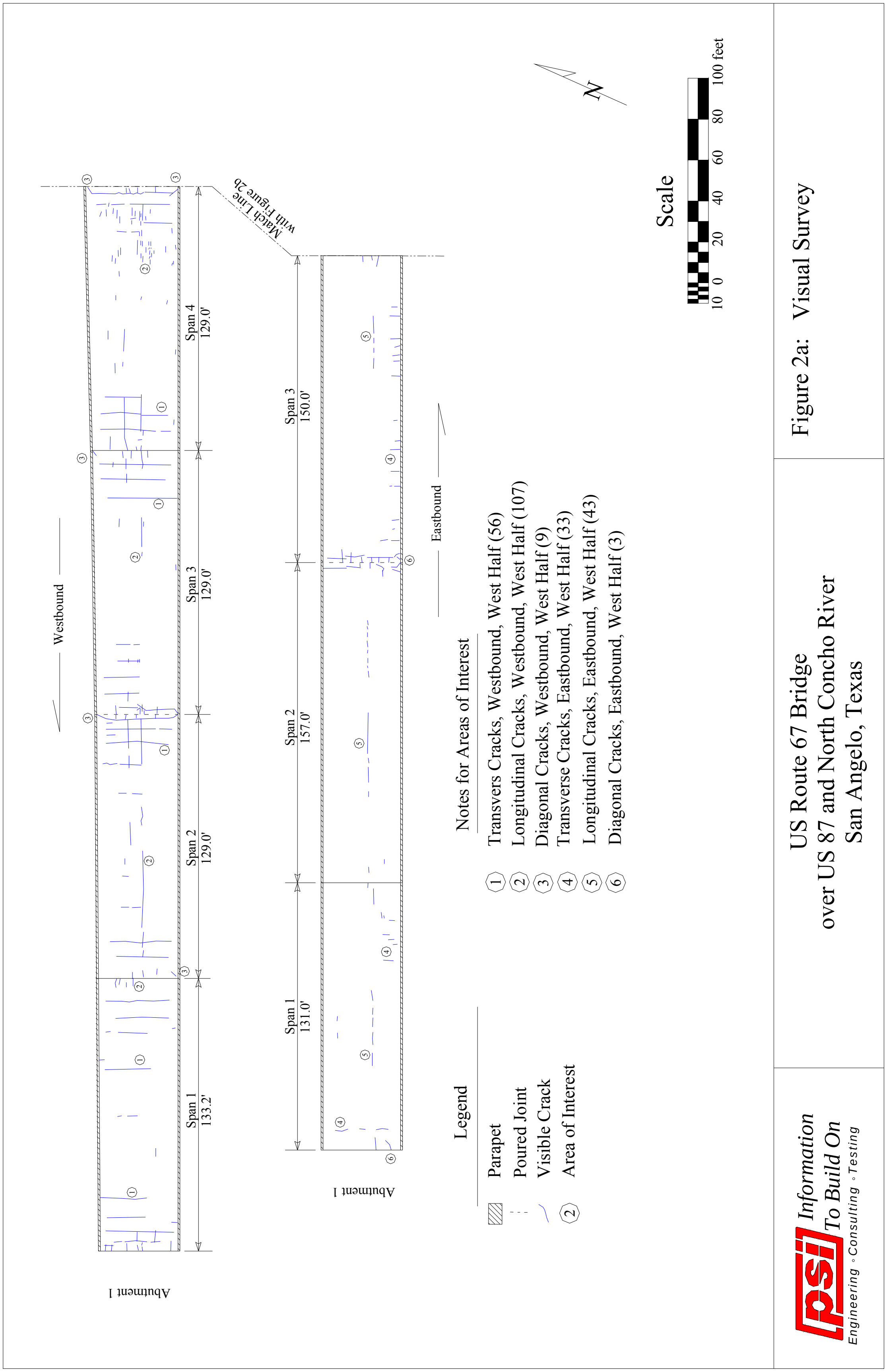


Figure 2a: Visual Survey

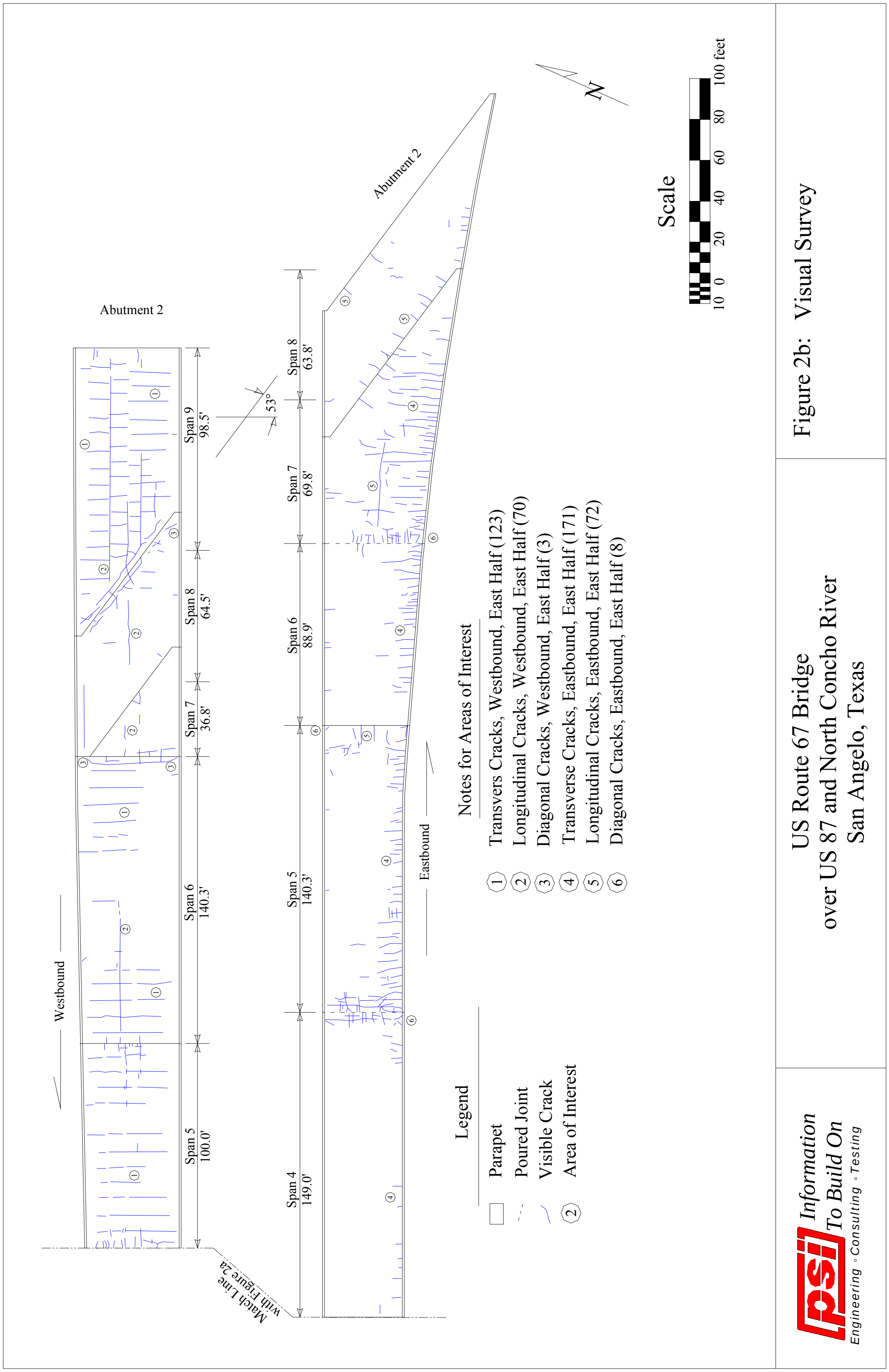


Figure 2b: Visual Survey

Photographic Documentation

Photo 1: View looking northeast. The eastbound bridge is in foreground, and the westbound bridge is in the background.

Photo 2: View looking northwest at the eastern end of the eastbound bridge.

Photo 3: View looking west from the eastern end of the eastbound bridge.

	Photo 4: View looking west from the eastern end of the westbound bridge.
	Photo 5: Closeup view of a transverse crack.
	Photo 6: Closeup view of a longitudinal crack.

	Photo 7: View of the eastbound bridge, the western end of Span 5, showing cracking pattern at the edge of span.
	Photo 8: View of the westbound bridge, the western end of Span 4 showing the rectangular cracking pattern.
	Photo 9: Typical condition of the underside of the deck.

	Photo 10: Cracks with efflorescence along the overhang on the westbound bridge.

APPENDIX P – Supplement 1

U.S. Route 67 Bridge, San Angelo, Texas
Petrographic Report

PETROGRAPHIC EXAMINATION OF CONCRETE CORES FROM A TEXAS BRIDGE (TXS)

A report submitted to the Structures Laboratory at TFHRC

Rongtang Liu
Petrographic Laboratory
FHWA TFHRC

March 29, 2005

Abstract

Six concrete cores were received from the Structures Laboratory of the Turner-Fairbank Highway Research Center for petrographic examination. Reportedly, the concrete cores were collected from a concrete bridge in San Angelo, Texas.

Petrographic examination was performed on samples using optical microscopes. Thin sections, as well as polished slabs, were made from the concrete cores for microscopic examination.

The findings indicate that the concrete has entrained air voids. The concrete was generally well consolidated and strong. There were micro-cracks in cement paste and in the cement paste/aggregate interfacial region. The presence of unhydrated cement particles, fly ash, and slag particles were also observed in the cement paste.

Introduction

The Petrographic Laboratory of the FHWA Turner-Fairbank Highway Research Center was asked by the Structures Laboratory to examine a set of concrete cores retrieved from a bridge. Six concrete cores of 3-1/2-in. diameter, 2- to 3-3/4-in. long were received by the Petrographic Laboratory. The identification on the cores was as follows: TXS-1, TXS-2, TXS-3, TXS-4, TXS-5, and TXS-6. Core TXS-4 had a conspicuous axial crack, splitting the core equally into two pieces.

Laboratory Procedures

Petrographic examination of the concrete samples was performed in accordance with ASTM C 856, "Standard Practice for Petrographic Examination of Hardened Concrete." Sections were polished and examined using a stereomicroscope at magnifications up to 350×. Small rectangular blocks were cut from concrete samples. One surface of each block was polished, dried, placed on petrographic slide with low−viscosity epoxy resin,

and reduced to a thickness of approximately 25 micrometers (0.001 in.). These thin sections were examined using a polarized–light microscope at magnification up to 400×, to determine aggregate mineralogy, paste characteristics, and microstructure.

Two ¾-inch thick slabs were sawn from the concrete cores. The slabs were ground and polished to obtain a smooth and plane surface. They were examined using a stereomicroscope at a magnification of 100×.

Findings

Six thin section samples taken from the received cores were examined using the polarized light microscope. Two polished sections were examined using the stereomicroscope. The findings are summarized as follows:

Aggregate

The coarse aggregate in the concrete is gravel. Coarse aggregate particles are rounded to sub-rounded, and the maximum size is about 3/4 inch. Preferential orientation of aggregate particles is not observed, nor is segregation. The rock type of this gravel coarse aggregate includes chert, limestone, quartz, and quartzite.

The fine aggregate is mainly composed of quartz, chert, and limestone, with a small portion of sandstone and quartzite. The fine aggregate is natural sand and the particles appear rounded to angular.

Cement Paste

The cement is reasonably hydrated and the cement paste contains some unhydrated cement particles as seen under the microscope (Figure P1-1). Fly ash is present in the concrete, shown in Figure P1-2. Slag particles are also present in the concrete as part of the paste (Figure P1-3).

Air Voids

Small, spherical air voids are observed in the concrete (Figure P1-4), hence the concrete was air entrained.

Cracks

Core TXS-4 has a visible axial crack, which is the only conspicuous crack among the six cores. The crack splits the core equally into two pieces.

Micro-cracks in cement paste were frequently observed in the concrete, as shown in Figures P1-5, P1-6, and P1-7. Cracks were also found in the paste/aggregate interfacial region (Figure P1-8 and Figure P1-9). Typically, these cracks meander along the aggregate peripherals and extend into the paste.

Gap between coarse aggregate particles and paste, although rare, were occasionally found in the concrete. Figure P1-10 shows such a gap in the paste/coarse aggregate interfacial region.

Secondary Deposit

Occasionally ettringite was observed in some voids in the concrete. Ettringite crystals when present normally filled up a portion of a void, as shown in Figure P1-11. There was no evidence of deterioration associated with the existence of the ettringite in the concrete. And ettringite is commonly seen as a secondary deposit in concrete in service.

Summary

The concrete appeared solid and sound. Except for the major crack in core TXS-4, there was no visible deterioration in the concrete. Microscopical examination revealed that the concrete was air entrained. Micro-cracks were frequently found in the cement paste, as well as in the aggregate/paste interfacial region. Occasionally, gaps between coarse aggregate and cement paste were observed in the concrete.

Voids partially filled by ettringite crystals were sporadically present in the concrete. There was no evidence of deterioration associated with the existence of the ettringite in the concrete. It is very common to see ettringite as secondary deposit in concrete.

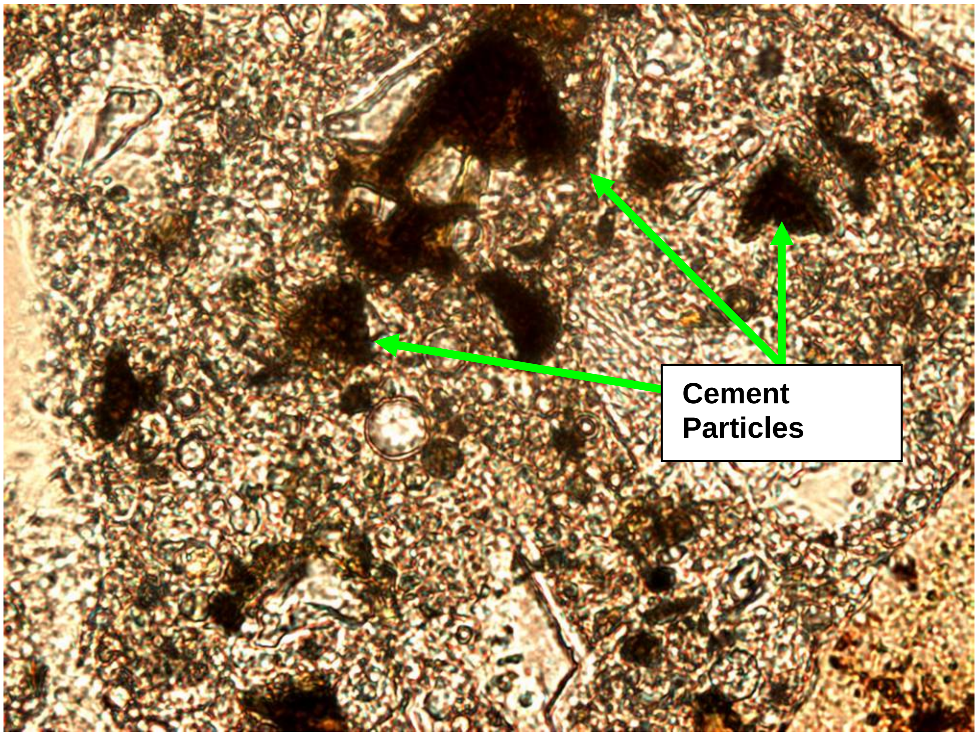


Figure P1-1. Unhydrated cement particles in paste. Width of field is 0.165 mm. Thin section image.

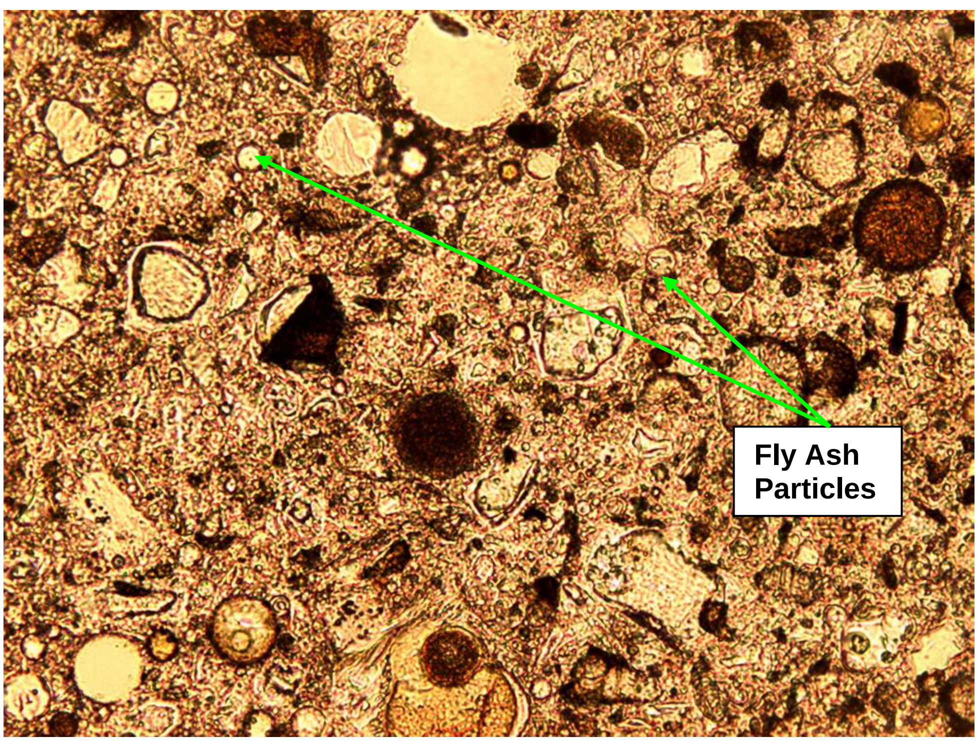


Figure P1-2. Fly ash in the concrete. Width of field is 0.33 mm. Thin section image.

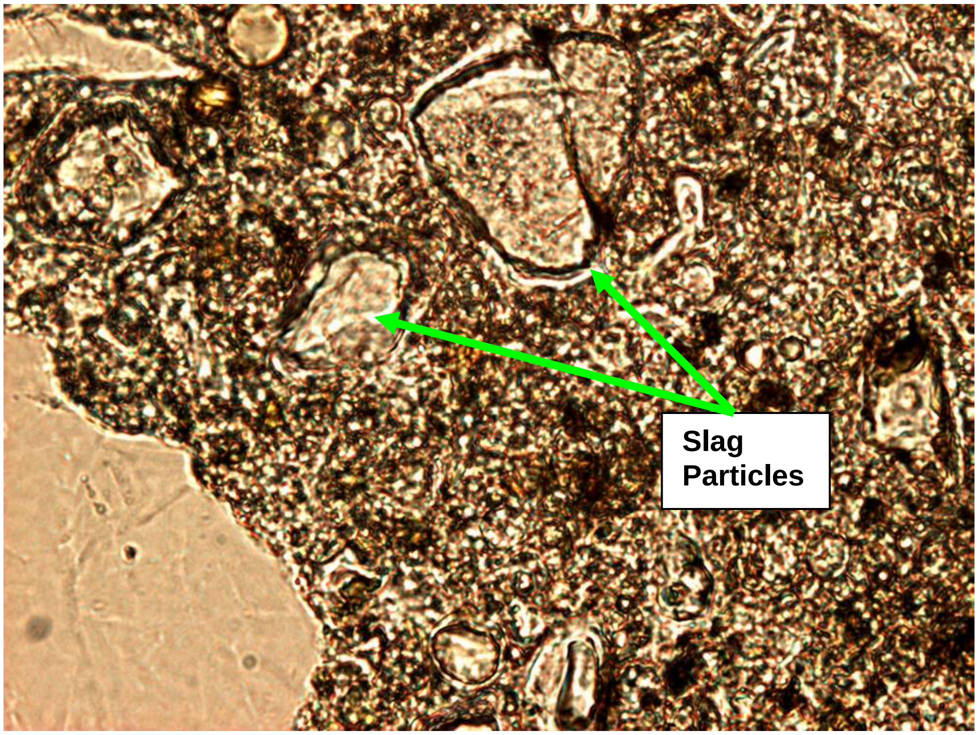


Figure P1-3. Slag particles in the concrete. Width of field is 0.165 mm. Thin section image.

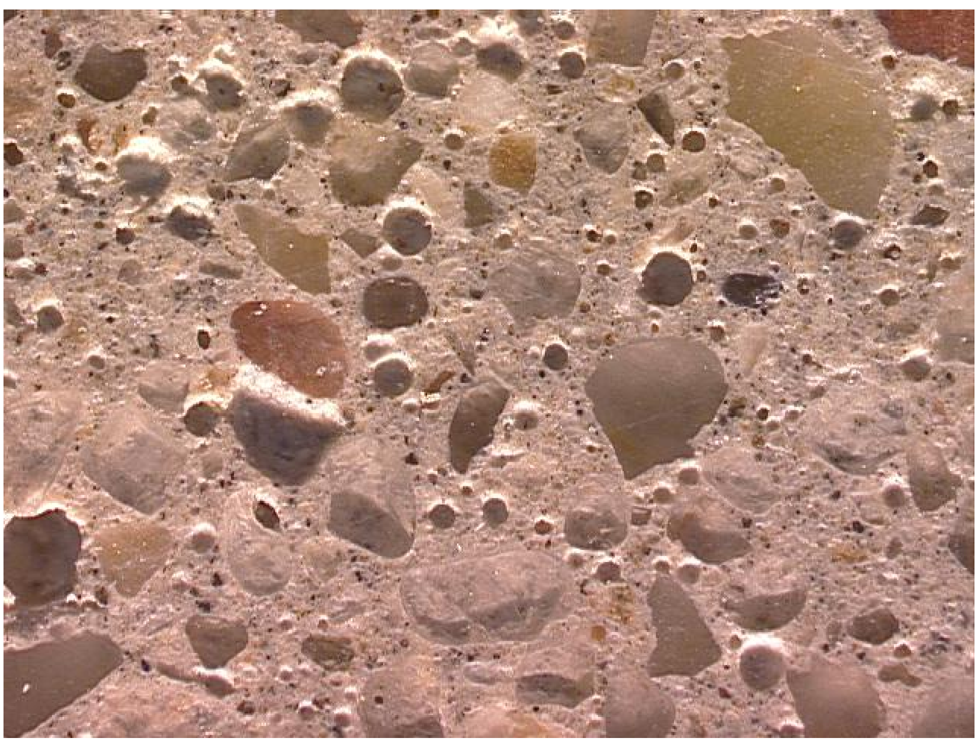


Figure P1-4. Small, spherical entrained air voids in the concrete. Width of field is 4.0 mm. Polished surface image.

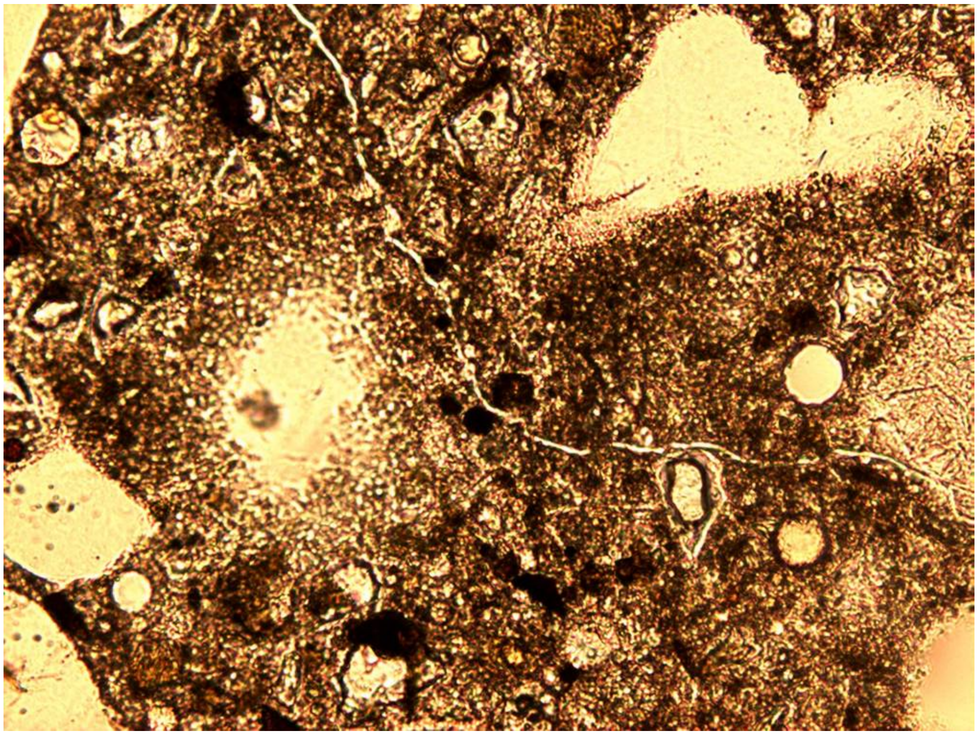


Figure P1-5. Crack in cement paste. Width of field is 0.33 mm. Thin section image.

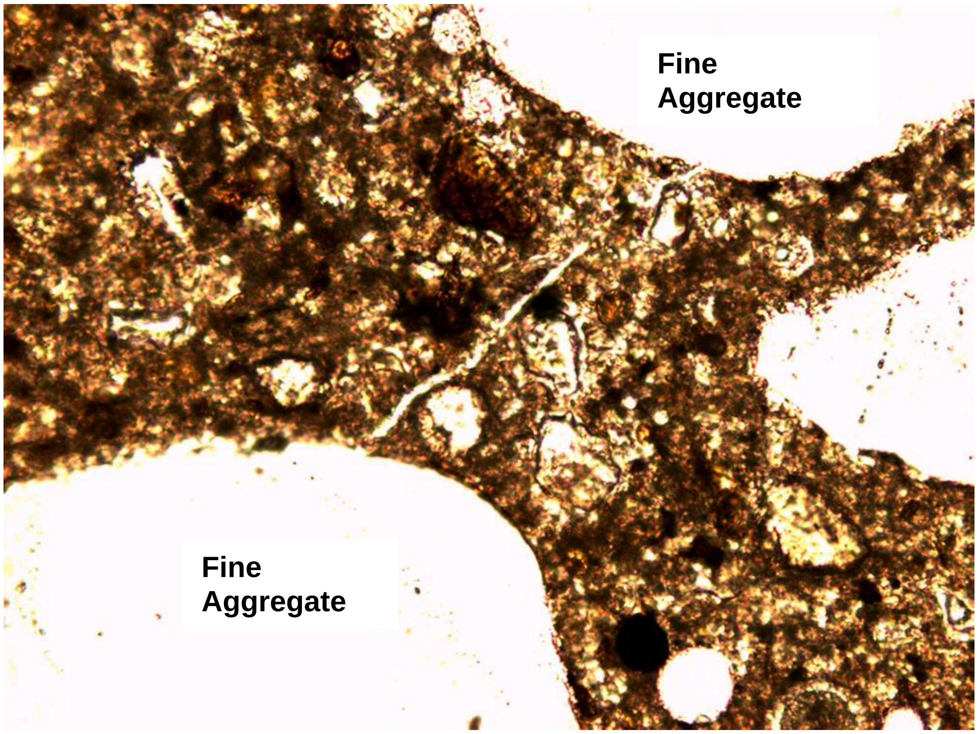


Figure P1-6. Crack connecting two fine aggregate particles. Width of field is 0.33 mm. Thin section image.

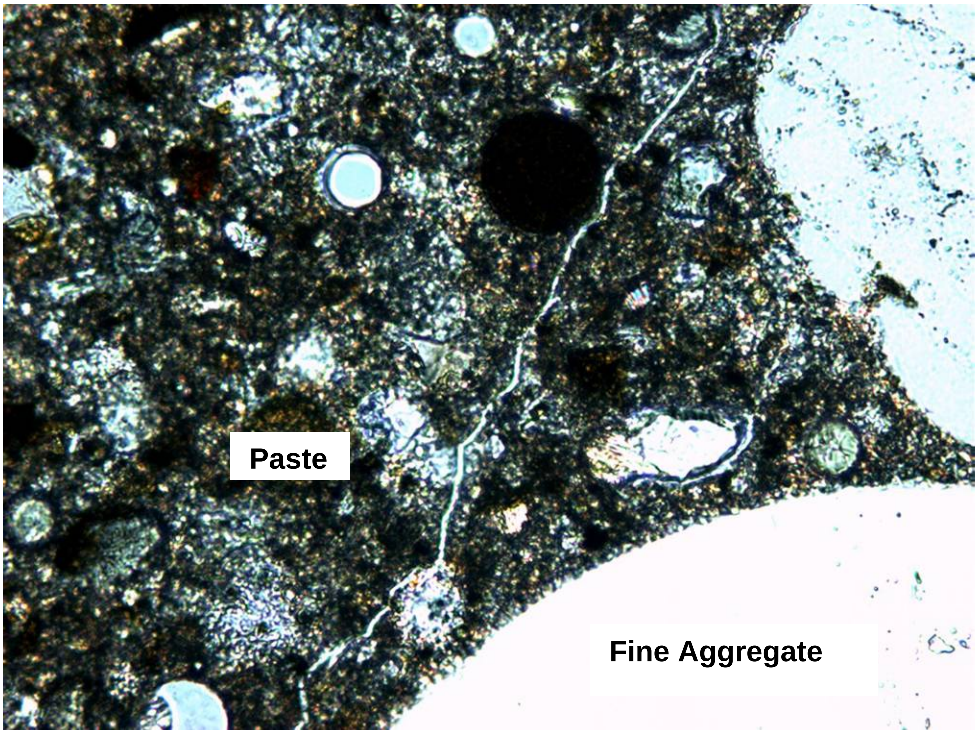


Figure P1-7. Crack in the cement paste and close to the aggregate/paste interface. Width of field is 0.33 mm. Thin section image.

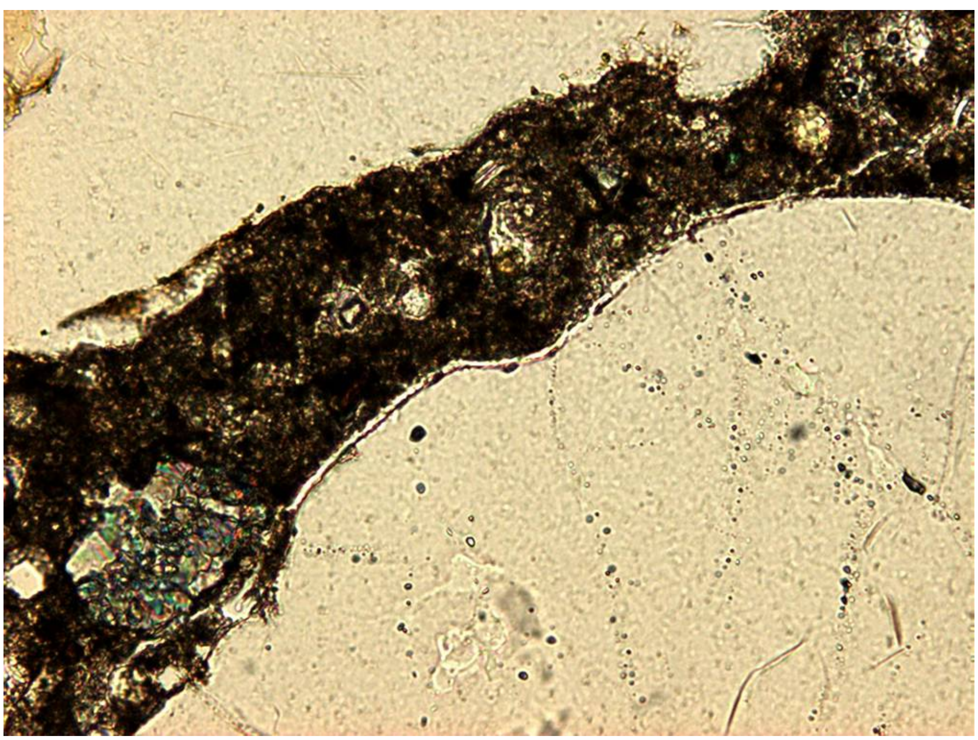


Figure P1-8. Crack in the paste/fine aggregate interfacial region. Width of field is 0.33 mm. Thin section image.

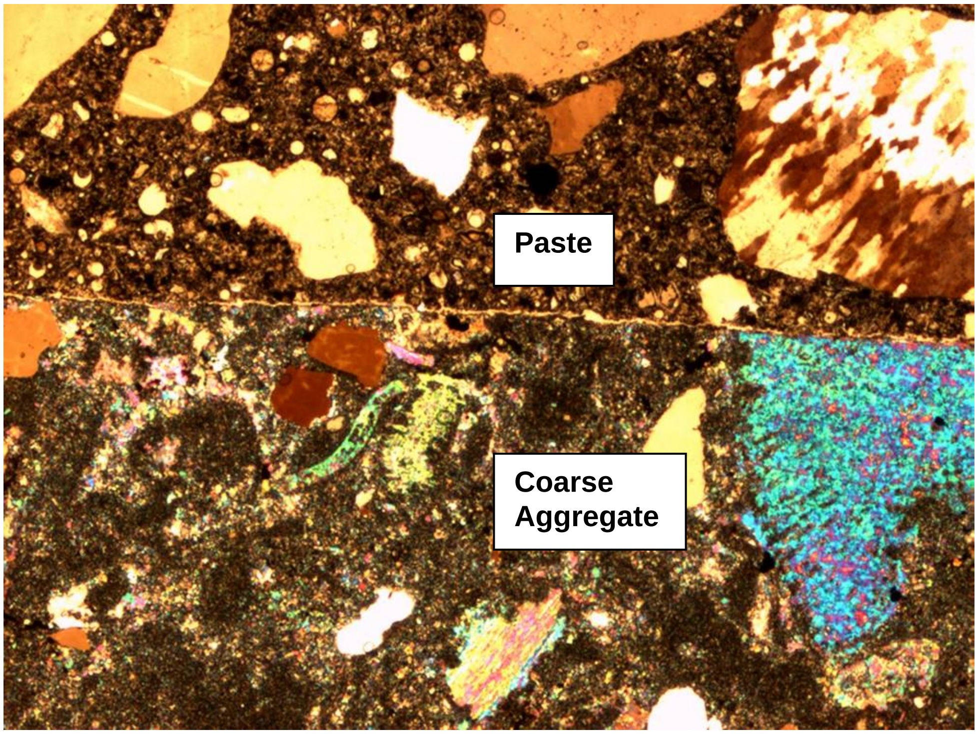


Figure P1-9. Crack in the paste/coarse aggregate interfacial region. Width of field is 1.60 mm. Thin section image.

Figure P1-10. Gap between the coarse aggregate and the paste. Width of field is 4.0 mm. Polished surface image.

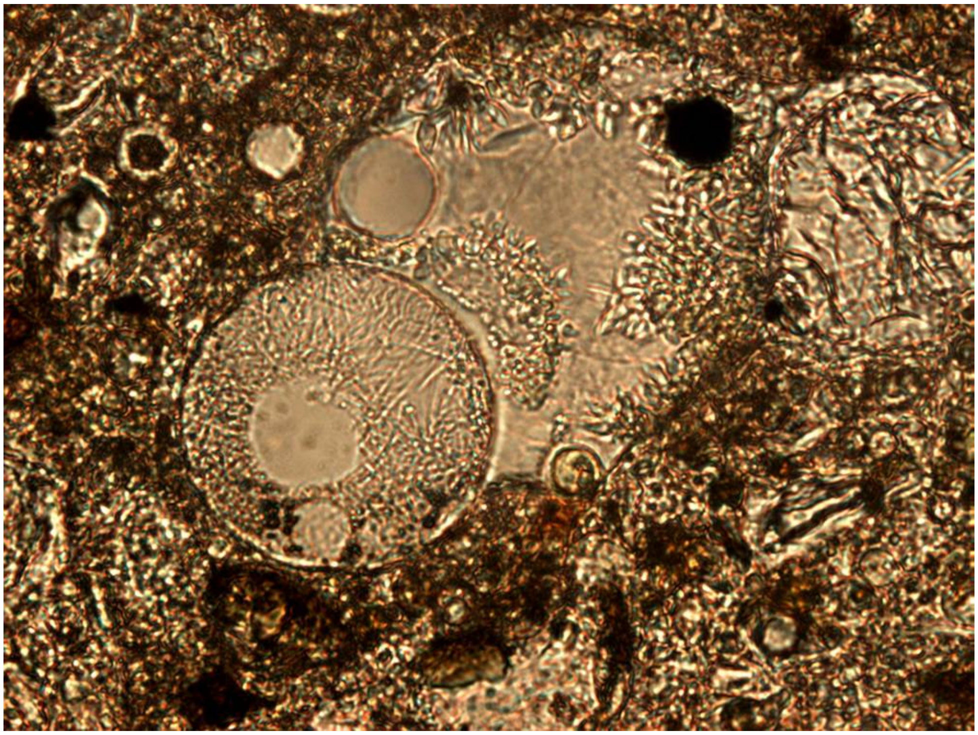


Figure P1-11. Needle-shaped ettringite crystals form in voids. Width of field is 0.165 mm. Thin section image.

APPENDIX P – Supplement 2

U.S. Route 67 Bridge, San Angelo, Texas
Survey Checklist

Checklist

The following checklist is adapted from 201.1 R-92, and provided to facilitate a thorough survey. The definition of terms and associated photographs in 201.1 R-92, are utilized to standardize the reporting of the condition of the concrete in the structures.

1. Description of structure or pavement
   - 1.1 Name, location, type, and size
     The U.S. Route 67 Bridge in San Angelo, Texas is part of a high-speed expressway and carries traffic over the North Concho River, U.S. Route 87, and South Orient Railroad tracks. The bridge consists of two separate structures, one carrying two lanes of eastbound traffic and the other two lanes of westbound traffic. The eastbound structure is 950-feet long and consists of eight spans. The westbound structure is 958-feet long and consists of nine spans.
   - 1.2 Owner, project engineer, contractor, when built
     Owner-Texas Department of Transportation. This bridge is part of a demonstration project for HPC in bridge structures which was co-sponsored by the Federal Highway Administration (FHWA) and the Texas Department of Transportation (TXDOT). The bridge was constructed in 1997 and opened to traffic in January 1998.
   - 1.3 Design
     - 1.3.1 Architect and/or engineer: The Texas Department of Transportation (TXDOT)
     - 1.3.2 Intended use and history of use: To carry traffic over the North Concho River, U.S. Route 87, and South Orient Railroad tracks. Opened to traffic in January 1998, No documents were found which would indicate any maintenance had been performed since bridge construction in 1997.
     - 1.3.3 Special features: Certain spans are skewed to accommodate the railroad tracks, the exit-ramp at the eastern end of the eastbound bridge, and the on-ramp at the eastern end of the westbound bridge. The structures are intended to be compared for relative durability and performance based on the extensive use of HPC in the eastbound structure and the limited use of HPC in the westbound structure.
   - 1.4 Construction
     - 1.4.1 Contractor-general Jascon Inc. of Uvalde TX and Reece Albert Inc. of San Angelo TX
     - 1.4.2 Subcontractors concrete placement: N/A
     - 1.4.3 Concrete supplier: Cast-in-place concrete provided by Concho Concrete Inc. of San Angelo TX
     - 1.4.4 Agency responsible for testing: N/A
     - 1.4.5 Other subcontractors: N/A
   - 1.5 Photographs
     - 1.5.1 General view Photos 1 through 2

1.5.2 Detailed close up of condition of area Photos 3 through 6

1.21 Sketch map-orientation showing sunny and shady and well and poorly drained regions: N/A

2. Present condition of structure Date of Evaluation The week of July 14, 2003

2.1 Overall alignment of structure No signs of misalignment

2.1.1 Settlement

2.1.2 Deflection

2.1.3 Expansion

2.1.4 Contraction

2.2 Portions showing distress (beams, columns, pavement, walls, etc., subjected to strains and pressures) N/A

2.3 Surface condition of concrete

2.3.1 General (good, satisfactory, poor, dusting, chalking, blisters) Good

2.3.2 Cracks Transverse, Diagonal, and longitudinal

2.3.2.1 Location and frequency See Figure 2

2.3.2.30 Type and size (see Definitions) See Figure 2

Longitudinal Over each of the girder lines

Width (from Crack comparator) Less than 0.012 in.

Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)

Transverse Throughout the length

Width (from Crack comparator) Less than 0.014 in..

Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)

Craze N/A

Width (from Crack comparator) ____________

Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)

Map N/A

Width (from Crack comparator) ______________

Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)

D-Cracking N/A

Width (from Crack comparator) ____________

Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)
Diagonal At Skew Ends and acute corners
Width (from Crack comparator) Less than 0.010 in.
Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)

2.3.2.31 Leaching, stalactites N/A

2.3.3 Scaling N/A

2.3.3.1 Area, depth ______

2.3.3.16 Type (see Definitions) ______
Light (Less than 1/8 in.)
Medium (1/8 in. – 3/8 in.)
Severe (3/8 in. – 3/4 in.)
Very Severe (Greater than 3/4 in.)

2.3.4 Spalls and popouts Very minor, insignificant at the base of few columns

2.3.4.1 Number, size, and depth ______

2.3.4.16 Type (see Definitions) ______
Spalls
Small (Less than 3/4 in. depth)
Large (Greater than 3/4 in. depth)
Popouts
Small (Less than 3/8 in. diameter)
Medium (3/8 in. – 2 in. diameter)
Large (Greater than 2 in. diameter)

2.3.5 Extent of corrosion or chemical attack, abrasion, impact, cavitation N/A

2.3.6 Stains, efflorescence Minor, around the Deck overhangs at the bottom side of the westbound bridge

2.3.7 Exposed reinforcement N/A

2.3.8 Curling and warping N/A

2.3.9 Previous patching or other repair N/A

2.3.10 Surface coatings N/A

2.3.10.1 Type and thickness N/A

2.3.10.2 Bond to concrete N/A

2.3.10.3 Condition N/A

2.3.11 Abrasion N/A

2.3.12 Penetrating sealers N/A

2.3.12.1 Type N/A

2.3.12.2 Effectiveness N/A

2.3.12.17 Discoloration N/A

2.4 Interior condition of concrete (in situ and samples) N/A
- 2.4.1 Strength of cores
- 2.4.2 Density of cores
- 2.4.3 Moisture content
- 2.4.4 Evidence of alkali-aggregate or other reactions N/A
- 2.4.5 Bond to aggregate, reinforcing steel, joints N/A
- 2.4.6 Pulse velocity
- 2.4.7 Volume change
- 2.4.8 Air content and distribution
- 2.4.9 Chloride-ion content
- 2.4.10 Cover over reinforcing steel
- 2.4.11 Half-cell potential to reinforcing steel.
- 2.4.12 Evidence of reinforcement corrosion
- 2.4.13 Evidence of corrosion of dissimilar metals
- 2.4.28 Delaminations N/A
  - 2.4.28.1 Location N/A
  - 2.4.28.2 Number, and size N/A
- 2.4.15 Depth of carbonation
- 2.4.16 Freezing and thawing distress (frost damage)
- 2.4.17 Extent of deterioration
- 2.4.32 Aggregate proportioning, and distribution

3. Nature of loading and detrimental elements

3.1 Exposure
- 3.1.1 Environment (arid, subtropical, marine, freshwater, industrial, etc.) ____________
- 3.1.2 Weather-(July and January mean temperatures, 82˚F/45˚F
  mean annual rainfall and 20.9 in.
  months in which 60 percent of it occurs) May- Oct. 68%
- 3.1.3 Freezing and thawing 51 days per year below 32˚F, Minimal annual exposure to F-T
- 3.1.4 Wetting and drying Minimal annual exposure
- 3.1.13 Drying under dry atmosphere N/A
- 3.1.6 Chemical attack-sulfates, acids, chloride N/A
- 3.1.7 Abrasion, erosion, cavitation, impact N/A
- 3.1.8 Electric currents N/A
- 3.1.9 Deicing chemicals which contain chloride ions N/A
- 3.1.10 Heat from adjacent sources N/A

3.2 Drainage N/A
- 3.2.1 Flashing ____________
- 3.2.2 Weepholes ____________
- 3.2.3 Contour ____________
- 3.2.4 Elevation of drains ____________

3.3 Loading Research Test Data Available in Compilation CD Version 3
- 3.3.1 Dead ____________
- 3.3.2 Live ____________

3.3.3 Impact	
3.3.4 Vibration	
3.3.5 Traffic index	
3.3.6 Other	
3.4 Soils (foundation conditions)	
3.4.1 Compressibility	
3.4.2 Expansive soil	
3.4.3 Settlement	
3.4.4 Resistivity	
3.4.5 Evidence of pumping	
3.4.6 Water table (level and fluctuations)	
4. Original condition of structure	Good
4.1 Condition of formed and finished surfaces	Good
4.1.1 Smoothness	
4.1.2 Air pockets ("bugholes")	
4.1.3 Sand streaks	
4.1.4 Honeycomb	
4.1.5 Soft areas (retarded hydration)	
4.1.6 Cold joints	
4.1.35 Staining	
4.1.36 Sand pockets	
4.2 Defects	
4.2.1 Cracking	
4.2.1.1 Plastic shrinkage	
4.2.1.2 Thermal shrinkage	
4.2.1.3 Drying shrinkage	
4.2.16 Curling	
5. Materials of Construction	See Tables 1, 2, 4, 5
6. Construction Practices	See Report pg. 11-12

APPENDIX Q

Route 40 Bridge, Brookneal, Virginia

HIGH PERFORMANCE CONCRETE BRIDGE DECK INVESTIGATION

Route 40 Bridge over Falling River Brookneal, Virginia

I. BACKGROUND

The Route 40 Bridge over Falling River in Campbell County was constructed during the winter of 1995 - 1996. The structure is 320-ft long and 44-ft wide (see photos 1 through 3, and Figure 1). It carries one eastbound lane and one westbound lane of Virginia Route 40. The structure consists of 8-½-in. thick concrete deck with stay-in-place forms on four 80-ft long simple span concrete prestressed superstructure, on three concrete piers and two concrete abutments. The structure was built with a 20^{o} skew at both abutments and all three piers. Five precast AASHTO Type IV girders, on 10-ft centers support each span, girder lines are identified as A through E from north to south. The concrete stub abutments are separated from Falling River with loose riprap slope protection. The concrete piers are comprised of cast-in-place concrete hammerhead caps on cast-in-place pier stems. Girders D and E, under the eastbound lane, have an 8 in. sewer line suspended below them

The abutments, piers, girders and deck were constructed with high performance concrete (HPC). The factors that led to the use of HPC in this bridge included the use of fewer girders and a more durable structure. If HPC was not used, two more lines of girders would have been required. The cost of HPC in this bridge was \$49.32 per ft^2, whereas the average Federal-Aid cost for bridges constructed that year was \$58 per ft^2. This represents a potential saving of \$122,000.

II. SCOPE OF SERVICES

Professional Service Industries Inc. (PSI) is under contract with the Federal Highway Administration (FHWA) to conduct bridge deck inspections for HPC bridges. Our scope of services on this bridge included a series of tasks and sub-tasks, which are described as follows:

- Collect all available information relevant to the construction of each bridge, including;
    - Deck Concrete Properties
    - Specified Deck Concrete Construction Procedures
    - Approved Concrete Mixture Proportions
    - Measured Properties from QC
    - Other Measured Properties
    - Actual Method of Deck Placement
    - Average Daily Traffic (ADT)
    - Exposure Condition of the Bridge
    - Any Performed Maintenance
    - Any Inspection Reports

- Visually inspect the bridge, and obtain the following:
  - General condition of the deck top surface
  - Determination of the maximum crack width
  - General condition of the deck underside
  - General condition of the girders
  - Photograph areas of significant deterioration
  - Prepare drawings locating defects
  - Extract 6 concrete core samples

III. COMPILATION OF BRIDGE CONSTRUCTION INFORMATION

Information sources used for this report include bridge drawings, inspection reports, bridge summary data sheets, and technical information contained in FHWA's "Compilation and Evaluation of Results from High Performance Concrete Bridge Projects" version 3.

Deck Concrete Properties

Table 1 lists the specified properties for the HPC used for the bridge deck construction.

TABLE 1: Specified Concrete Properties

Property	Deck
Minimum Cementitious Materials Content:	635 lb/yd^3
Max. Water/Cementitious Materials Ratio:	0.45
Min. Percentage of Fly Ash:	20
Max. Percentage of Fly Ash:	25
Min. Percentage of Silica Fume:	7
Max. Percentage of Silica Fume:	10
Min. Percentage of GGBFS:	35
Max. Percentage of GGBFS:	50
Maximum Aggregate Size:	1 in.
Slump:	2-4 in. (1)
Air Content:	6.5 ± 1.5% (2)
Compressive Strength	4000 psi at 28 days
Chloride Permeability (AASHTO T 277):	2500 coulombs at 28 days (3)
ASR or DEF Prevention:	ASR (4)
Freeze-Thaw Resistance:	Not specified
Deicer Scaling:	Not specified
Abrasion Resistance:	Not specified

NOTES: Minimum and maximum percentages of mineral admixtures only apply if the materials are used.
(1) Maximum of 7 in. when a high-range water-reducing admixture (HRWR) is used.
(2) Target air content is increased by 1 percent when a HRWR is used.
(3) Curing procedure of one week at 73°F and three weeks at 100°F with AASHTO T 277 test.
(4) Cement shall be Type II with a maximum alkali content of 0.40% or Type I-P, unless otherwise specified in the contract. Fly ash or granulated iron blast-furnace slag shall not be added to concrete when Type I-P cement is used. Fly ash, granulated iron blast-furnace slag, silica fume, or other VDOT approved mineral admixtures shall be used with Types I, II (if above 0.40% alkali content), or III cements.

Specified Deck Concrete Construction Procedures

For the cast-in-place concrete deck a moist curing for 7 days was specified. Wet burlap was placed on the newly cast concrete, and a plastic sheeting was put on top of the wet burlap. Curing compound was applied after removal of plastic sheeting and burlap. Air content, slump, and concrete temperature measurements were required for QA/QC. Concrete cylinders, 4 x 8 in. in dimensions and moist cured, were prepared for quality control testing.

Approved Concrete Mixture Proportions

The approved proportions for the concrete mixture are shown in Table 2.

TABLE 2: Approved Mixture Proportions

Mixture Parameters	Deck
Cement Brand:	Not available
Cement Type:	II
Cement Composition:	Not available
Cement Fineness:	Not available
Cement Quantity:	329 lb/yd^3
GGBFS Brand:	Not available
GGBFS Quantity:	329 lb/yd^3
Fly Ash Brand:	—
Fly Ash Type:	—
Fly Ash Quantity:	—
Silica Fume Brand:	—
Silica Fume Quantity:	—
Fine Aggregate Type:	Natural sand
Fine Aggregate FM:	2.80
Fine Aggregate SG:	2.63
Fine Aggregate Quantity:	1173 lb/yd^3
Coarse Aggregate, Max. Size:	1 in
Coarse Aggregate Type:	No. 57 arch marble
Coarse Aggregate SG:	2.73
Coarse Aggregate Quantity:	1773 lb/yd^3
Water:	263 lb/yd^3
Water Reducer Brand:	Polyhead 997
Water Reducer Type:	A and F
Water Reducer Quantity:	66 fl oz/yd^3
High-Range Water-Reducer Brand:	Rheobuild 1000
High-Range Water-Reducer Type:	A and F
High-Range Water-Reducer Quantity:	13 to 20 fl oz/yd^3

continued

TABLE 2 (continued): Approved Mixture Proportions

Mixture Parameters	Deck
Retarder Brand:	—
Retarder Type:	—
Retarder Quantity:	—
Corrosion Inhibitor Brand:	—
Corrosion Inhibitor Type:	—
Corrosion Inhibitor Quantity:	—
Air Entrainment Brand:	Micro-Air
Air Entrainment Type:	Synthetic surfactant mixture
Air Entrainment Quantity:	8.5 fl oz/yd^3
Water/Cementitious Materials Ratio:	0.40

Measured properties of approved concrete mixtures for the concrete deck are summarized in Table 3.

TABLE 3: Measured Properties of Approved Concrete Mixtures and QC Tests of Production Concrete

Measured Concrete Properties	Deck
Approved Concrete Mixture	
Slump:	3.5 in.
Air Content:	5.5%
Compressive Strength	6430 psi at 28 days
Rapid Chloride Permeability (AASHTO T 277):	1109 coulombs at 28 days
QC Tests from Production Concrete	
Slump:	5.7 in.
Air Content:	7.0%
Compressive Strength	6600 psi at 28 days

The properties of the cement used in the deck were not available.

Measured Properties from Research Tests of Production Concrete

The values listed in Table 4 were obtained from research tests of production concrete for the deck.

TABLE 4: Measured Properties of Deck Concrete Mixtures

Test	Specimen			Deck Batch No.			
	Size	Age	No.	1	2	3	4
Air, %				—	6.4	3.4	6
Slump, in				5.5	4.8	4.3	4.3
Concrete Temp. at Time of Placement, °F				53	53	61	61
Air Temp., °F				56	56	67	67
Compressive Strength [1], psi	4x8 in.	1 d	—	590	420	1730 [2]	1660 [2]
		7 d	3	5820	5440	5400	4890
		28 d	3	8400	8100	9050	9290
		1 yr	3	9510	9280	10,680	10,810
Modulus of Elasticity [3], ksi	4x8 in.	1 yr	3	5590	5470	6320	6120
Flexural Strength [4], psi	3x3x 11-1/4 in.	28 d	3	870	830	1040	1000
Splitting Tensile Strength [5], psi	4x8 in.	28 d	3	765	685	750	750
Chloride Permeability [6], coulombs	2x4 in.	28 d [7]	2	696	773	743	898
		28 d	—	1428	1405	1256	1677
		1 yr	—	705	674	602	782
Shrinkage [8], %	3x3x 11-1/4 in.	64 wk	3	0.059	0.057	0.045	0.048

(1) AASHTO T 22 with neoprene pads in steel end caps.
(2) At 3 days.
(3) ASTM C 469.
(4) AASHTO T 97 (ASTM C 78).
(5) AASHTO T 198 (ASTM C 496).
(6) AASHTO T 277 (ASTM C 1202).
(7) Cured one week at 73°F and three weeks at 100°F.
(8) AASHTO T 160 (ASTM C 157). Moist cured for 28 days, and then air dried.

Actual Method of Deck Placement

Construction of the deck occurred in December 1995, with the concrete for the deck delivered by truck and pumped to the deck surface. Surface finishing consisted of vibratory screed followed by a roller screed. The edges were consolidated with immersion type vibrators and hand floated. The deck was cured using water soaked burlap covered with white plastic and an insulating blanket for seven days. Prior to the application of the wet burlap, the deck was fogged with water to prevent drying. Due to the winter weather, the sides and bottom of the deck were enclosed with plastic and space heaters on the pier caps were utilized to provide additional heat. After seven days the burlap was removed and white pigmented curing compound was applied. The transverse grooves were cut several weeks after placement. No milling operations were performed on this deck. Two days of concrete placement were required for the deck. The ambient temperature was 67 °F on the first day and 56 °F the second.

Average Daily Traffic (ADT) and Average Daily Truck Traffic (ADTT)

The district identifies this bridge as carrying 3,120 vehicles per day, with 16 % trucks. While the PSI inspection crew was on-site 100 cars were noted per hour. Also, twenty-five trucks per hour were noted during this time period. This represents an ADT of approximately 3,000 and an ADTT of 600.

Exposure Condition of the Bridge

The surrounding area is generally agricultural land use. The National Weather Service reports that the mean July temperature is 75°F, while the mean January temperature is 34°F. The mean annual rainfall is 43.3 in., 62% of the annual amount falls from March-September. The bridge is exposed to freezing and thawing as well as wetting and drying on a seasonal cycle basis.

Performed Maintenance

The 1998 inspection report indicates that cracks in the girders, noted in the 1996 inspection report, had been sealed with epoxy. The 2002 inspection report notes that debris in the waterway along Pier 1 had been removed.

Inspection Reports

Bridge inspection reports dated 5/3/96, 6/22/98, 5/3/00 and 4/29/02 were identified for this bridge. The 1996 inspection report summarizes the Condition of Structure as "GOOD although small horizontal cracks exist along edge of steel plate in beams at bearing areas; back corner of several beams cracked or delaminated slightly; and small hairline cracks exist on abutments and piers. Also several nuts loose on plate connections to beams for sewer line."

The 1998 inspection report summarizes the Condition of Structure as "GOOD-Horizontal cracks in edge of steel plate in beams at bearing areas. Back corner of several beams cracked, delaminated and spalled. Hairline cracks exist in abutments and piers. Several nuts loose on plate connections to beams for sewer lines. Split in elastomeric joint sealer."

The 2000 inspection report summarizes the Condition of Structure as "GOOD-Horizontal cracks in edge of steel plate in beams at bearing areas. Back corner of several beams cracked, delaminated and spalled. Hairline cracks exist in abutments and piers. Several nuts loose on plate connections to beams for sewer lines. Split in elastomeric joint sealer. Pier footing exposed. Drains blocked. Debris lodged against pier."

The 2002 inspection report summarizes the Condition of Structure as "GOOD- Cracks in deck surface, parapets, beam ends, abutment backwalls and pier caps. Debris in drain grates. Loose bolts in utility connections to beams."

IV. BRIDGE DECK INSPECTION

The bridge deck received a close visual inspection November 19 through 21, 2002, the findings of this inspection are summarized as follows.

General Condition of the Deck Top Surface

Defects in the top surface include longitudinal cracks, transverse cracks, and diagonal corner cracks in the acute corners, small spalls and areas of fractured fins from deep grooving.

Longitudinal Cracks: Longitudinal cracks were found primarily over each of the girder lines. Figure 2 illustrates the 465 ft of longitudinal cracks that were identified on the top surface of the deck (see Table 5). The crack widths ranged from 0.003 to 0.016 in. for the 27 cracks (see photos 4 through 7).

TABLE 5: Longitudinal Cracks

Span	Number	Total Length of Cracks (ft.)	Deck Area (ft.^{2})	Crack Density (ft/ft^2)
A	7	92	1840	0.0500
B	6	101	1840	0.0549
C	6	128	1840	0.0696
D	8	144	1840	0.0783
Cumulative	27	465	7360	0.0632

Transverse Cracks: Transverse cracks were found throughout the length of the bridge. Figure 2 illustrates the 201-ft of transverse cracks that were identified on the top surface of the deck (see Table 6). The crack widths ranged from 0.003 to 0.010 in. for the 21 cracks (see photo 8).

TABLE 6: Transverse Cracks

Span	Number	Total Length of Cracks (ft.)	Deck Area (ft.^{2})	Crack Density (ft/ft^2)
A	7	84	1840	0.0457
B	9	69	1840	0.0375
C	1	8	1840	0.0043
D	4	40	1840	0.0217
Cumulative	21	201	7360	0.0273

Diagonal Cracks: Diagonal cracks were found primarily perpendicular to the expansion joints at both abutments and Pier 2. Figure 2 illustrates the 52 ft of diagonal cracks that were identified on the top surface of the deck (see Table 7). The crack widths ranged from 0.003 to 0.025 in. for the 9 cracks (see photos 9 and 10).

TABLE 7: Diagonal Cracks

Span	Number	Total Length of Cracks (ft.)	Deck Area (ft.^{2})	Crack Density (ft/ft^2)
A	2	15	1840	0.0082
B	4	21	1840	0.0114
C	0	0	1840	0.0000
D	3	16	1840	0.0087
Cumulative	9	52	7360	0.0071

In addition to the different types of cracking noted, a few isolated small defects were found. These defects included two small spalls in Span A, D-Spalls at the expansion joints, irregular grooving and debris collecting at the drains.

Small spalls: Two small spalls are located in Span A. One spall is 2-in. diameter by ½-in. deep, while the other is 2-in. by 3-in. by ½-in. deep.

D-Spalls: One D-spall is located along the expansion joint over Pier 1 and two D-spalls are located over Pier 3 (see photos 11 and 12). The D-spall over Pier 1 is 3-in. by 1-in. and 1-in. deep and located at the right edge of the westbound lane on the Span B side of the joint. Over Pier 3 one of the D-spalls is 4 ½-in. by 1-in. and 1-in. deep and the other D-spall is 2 ½-in. by 1-in. and 1-in. deep. They are located at the right edge of the westbound lane on the Span D side of the joint.

Irregular texturing: Deep irregular grooving is found in Spans A, C and D. The grooves are deep enough to contribute to the fracturing of the exposed fins (see photos 13 and 14). On the other hand, in Span D, the grooves are too shallow and the texturing is almost non-existent (see photos 15 and 16). The shallow grooves in Span D appear to have been constructed that way, not worn down to that point.

Deck drains: Moderate accumulation of debris was noted generally around the deck drains (see photo 16).

Maximum Crack Width

The maximum width of longitudinal cracks and transverse cracks was measured to be 0.016 in. and 0.010 in., respectively. The maximum width of diagonal cracks was measured to be 0.025 in. According to ACI 201, these crack widths are classified as hairline cracks.

General Condition of the Deck Underside

The undersurface of the deck was not visible due to the presence of stay-in-place deck forms. However, the exterior cantilever portions of the deck were exposed, which exhibit no signs of distress. Photos 17 and 18 show general views of the underside of the deck.

General Condition of the Girders

The girders were inspected from the ground, without the aide of any access equipment. No signs of distress were noted on any of the girders. Photos 19 through 22 show general views of the girders.

Concrete Core Samples

Six cores were retrieved from the deck, Figure 1 illustrates their locations. The locations were selected to distribute the samples along each shoulder of the bridge, since Virginia DOT had requested that the coring and patching operation avoid the traveled lanes. The cores were labeled VAB-1 through VAB-6 (see Table 8) and transferred to FHWA on January 7, 2003, for further investigation.

TABLE 8: Core Dimensions

Sample	VAB-1	VAB-2	VAB-3	VAB-4	VAB-5	VAB-6
Diameter (in.)	3-¾	3-¾	3-¾	3-¾	3-¾	3-¾
Length (in.)	5-¾	5-¾	5-¾	5-½	5-½	5-¾

Preliminary Conclusions

The Route 40 Bridge over Falling River in Campbell County was constructed during the winter of 1995 - 1996. Construction of the deck occurred in December 1995. The abutments, piers, girders and deck were constructed with high performance concrete (HPC).

Bridge inspection reports dated 5/3/96, 6/22/98, 5/3/00 and 4/29/02 were identified for this bridge. The 1996 inspection report documented that small horizontal cracks existed along edge of steel plate in beams at bearing areas; back corner of several beams cracked or delaminated slightly; and small hairline cracks existed on abutments and piers. The 1998 and 2000 inspection reports identified the same conditions. Cracks in deck surface were first documented in the 2002 inspection report. Other defects included cracks in parapets, beam ends, abutment backwalls and pier caps.

The visual inspection was performed on November 19 through 21, 2002. There were longitudinal, transverse, and diagonal corner cracks on the surface of the decks. According to ACI 201, these crack widths are classified as hairline cracks. Small spalls and fractured fins from deep grooving were also observed.

Longitudinal cracks were found primarily over each of the girder lines. A total of 27 longitudinal cracks were identified, with a combined total crack length of 465 ft. The crack widths varied from 0.003 to 0.016 in.

Transverse cracks were found throughout the length of the bridge. A total of 21 transverse cracks were identified on the top surface of the deck, with a combined total crack length of 201 ft. The crack widths ranged from 0.003 to 0.010 in.

Diagonal cracks were found primarily perpendicular to the expansion joints at both abutments and Pier 2. A total of 9 diagonal cracks were identified on the top surface of the deck, with a combined crack length of 52 ft. The crack widths varied from 0.003 to 0.025 in.

In addition to the different types of cracking noted, a few isolated small defects were found on the deck. These defects included small spalls and D-spalls. Deep irregular grooving was found in some areas of the deck, and the grooves were deep enough to contribute to the fracturing of the fins. On the other hand, very shallow grooves were also found on the deck surface.

Petrographic examination was performed on six concrete cores that were retrieved from the bridge decks. All six cores appeared intact. There were no visible defects or deterioration on the cores. The concrete was well consolidated, with no noticeable entrapped air voids.

The crushed stone coarse aggregate was from carbonate rock. Coarse aggregate particles were angular, and the maximum size, measured from the prepared specimens, was about 3/4 inch. A small portion of the coarse aggregate was elongated or flaky particles. Preferential orientation of aggregate particles was not observed, nor was segregation. The fine aggregate was natural sand and the particles appeared rounded to angular. The fine aggregate was mainly composed of quartz.

The cement paste was reasonably hydrated with respect to the age of the concrete. There were some unhydrated cement particles present in the cement paste. Ground granulated blast furnace slag (GGBFS) particles were present in the concrete. Hence, the concrete mixture contained GGBFS as supplementary cementitious material.

The concrete was air entrained. Small, spherical air voids were observed in the concrete. It was found that in some portions of the concrete, there was a tendency that air voids accumulate the coarse aggregate-cement paste interfacial region.

Isolated micro-cracks were sporadically observed in cement paste. Conspicuous cracks were also found in the paste/coarse aggregate interfacial region. Typically, these cracks meandered along the aggregate peripherals and extended into the cement paste matrix. Cracks were also found in the fine aggregate/paste interfacial region. Occasionally, cracked fine aggregate particles were also observed. Gap/crack between coarse aggregate and paste, although very rare, was found in the concrete.

Ettringite was observed in some air voids in the concrete. Ettringite crystals normally filled up a portion of a void. There was no evidence of deterioration associated with the existence of the ettringite in the concrete.

When observed under the microscope, the cement paste in the paste/aggregate interfacial region was porous. Both the accumulative air voids and the porous paste might weaken the bond between the paste and the aggregate. The bonding was further adversely affected by the sporadic gaps/cracks between coarse aggregate and cement paste. Cracks were present in the cement paste and in the fine aggregate/paste interfacial region. Cracked fine aggregate particles were also found in the concrete.

Respectfully Submitted,
Professional Service Industries, Inc.
Structural Investigation & Petrography Department

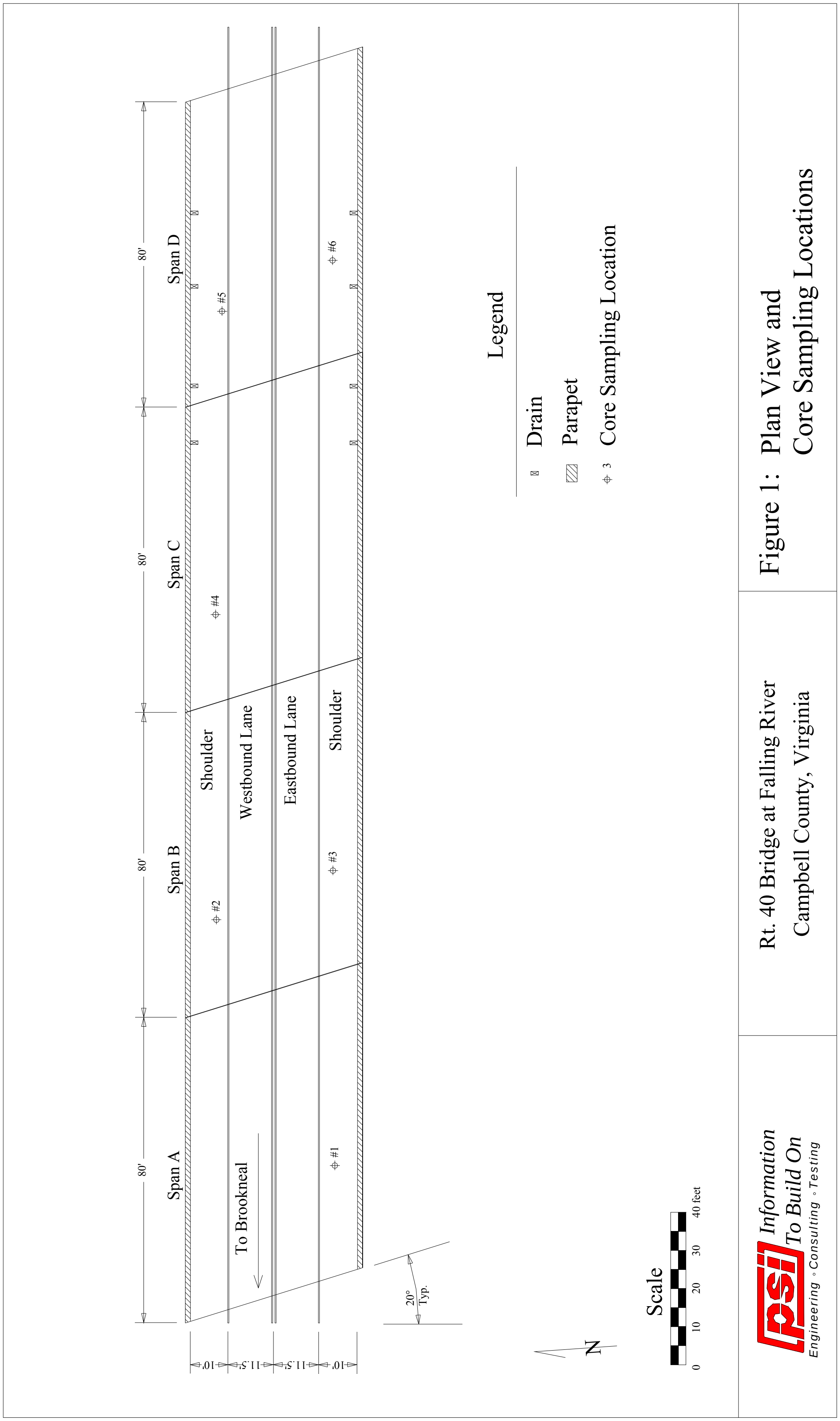


Figure 1: Plan View and Core Sampling Locations

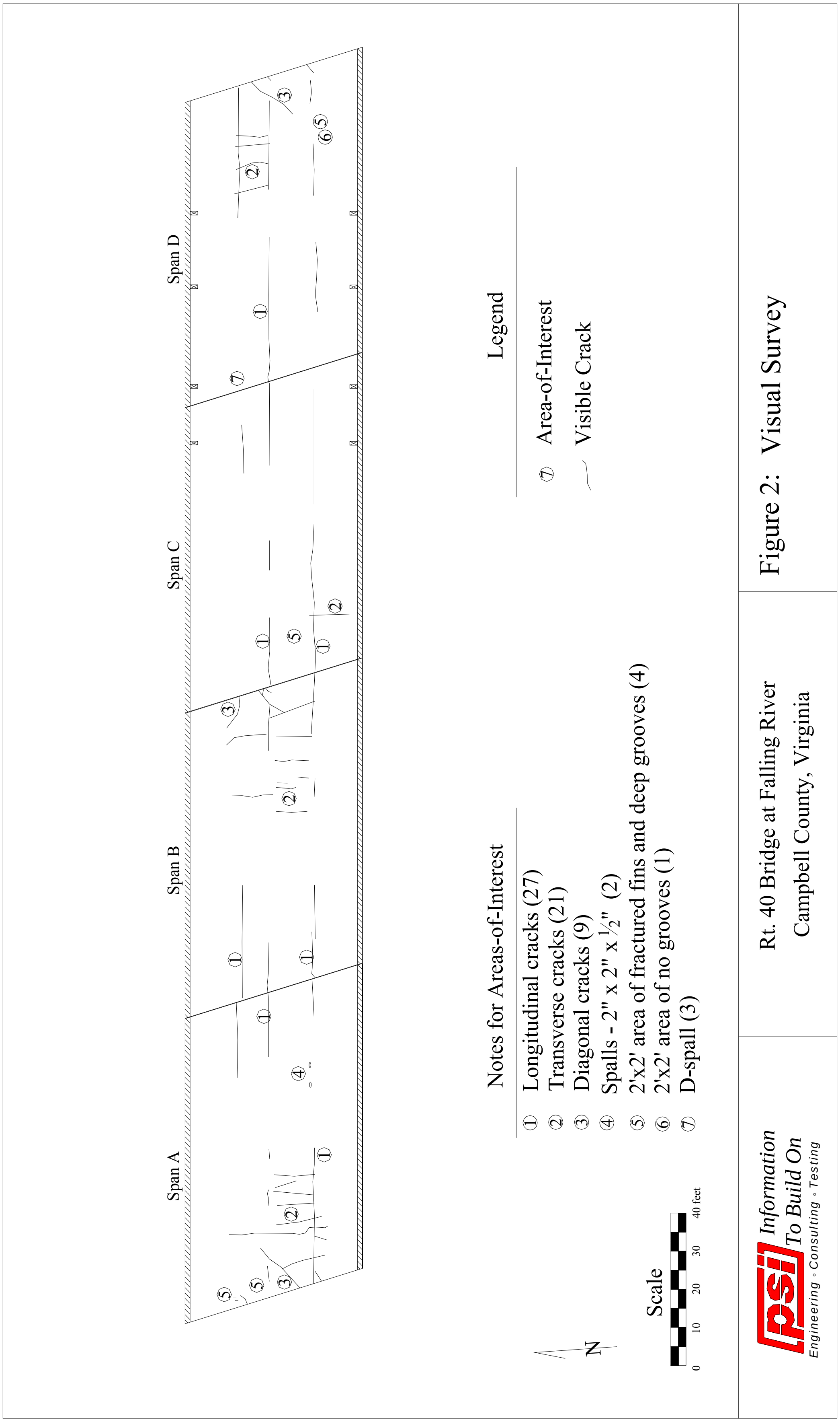


Figure 2: Visual Survey

Photographic Documentation

	Photo 1: General view, south elevation.
	Photo 2: General view, east approach .
	Photo 3: General view, west approach

	Photo 4: General view longitudinal cracks east bound.
	Photo 5: General view, longitudinal cracks, east bound.
	Photo 6: Detail view, longitudinal cracks, 0.008 in. width. .

	Photo 7: General view, longitudinal cracks, west bound
	Photo 8: Transverse cracks.
	Photo 9: Typical diagonal crack, detail view, 0.018 in. width.

Photo 10: Typical diagonal crack, detail view, 0.025 in. width.

Photo 11: Typical D-Spall.

Photo 12: Typical D-Spall, detail view.

Photo 13: Typical deep-grooving.

Photo 14: Typical deep-grooving.

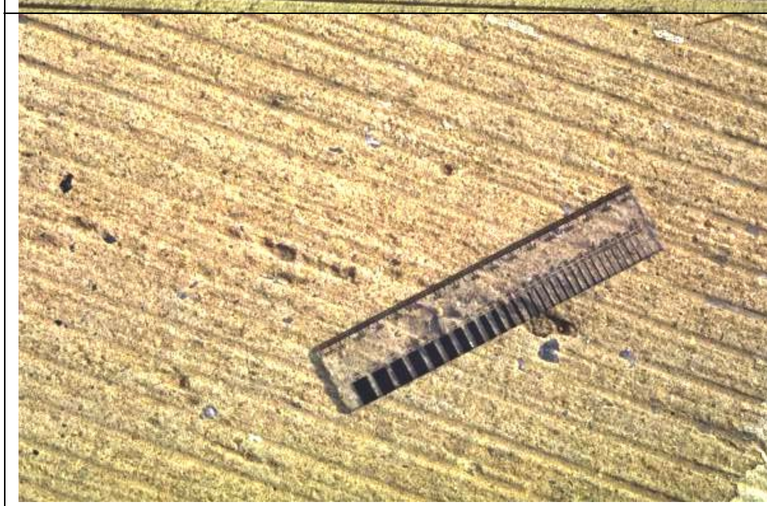


Photo 15: Typical shallow-grooving

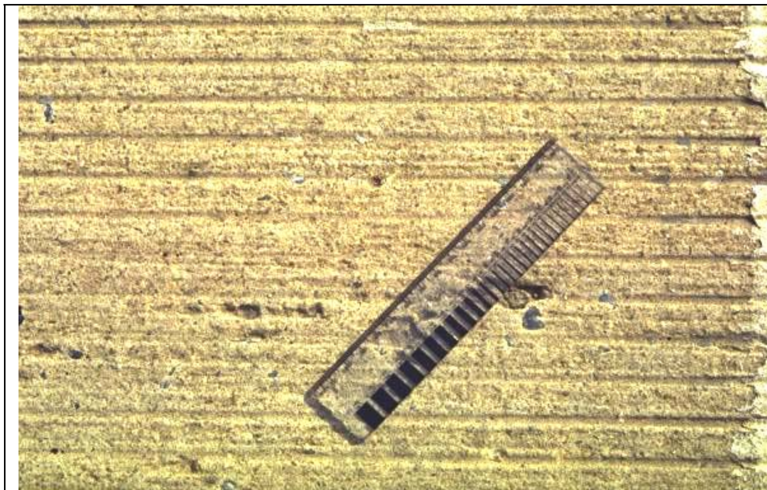


Photo 16: Typical shallow-grooving.

Photo 17: Typical deck drain with moderate debris.

Photo 18: Underside of deck, note stay-in-place forms.

	Photo 19: Underside of deck, under Span D. .
	Photo 20: Span A, north elevation, exterior Girder A in foreground.
	Photo 21: Underside of deck Pier 3, exterior Girder A on right.

	Photo 22: Underside of deck abutment B, exterior Girder A on left.
	Photo 23: South elevation, exterior Girder E. Note 8-in. sewer pipe below Girders D & E. .

APPENDIX Q – Supplement 1

Route 40 Bridge, Brookneal, Virginia
Petrographic Report

PETROGRAPHIC EXAMINATION OF CONCRETE CORES FROM A VIRGINIA BRIDGE (VAB)

A report submitted to the Structures Laboratory at TFHRC

Rongtang Liu
Petrographic Laboratory
FHWA TFHRC

March 28, 2005

1. Abstract

Six concrete cores were received from the Structures Laboratory of the Turner-Fairbank Highway Research Center for petrographic examination. Reportedly, the concrete cores were collected from a concrete bridge.

Petrographic examination was performed on samples using optical microscopes. Thin sections, as well as polished slabs, were made from the concrete cores for microscopic examination.

The concrete was generally well consolidated. The concrete appeared solid and sound, and no deterioration was shown visually. The findings from microscopical examination indicate that the concrete has entrained air voids. But in some portions of the sample, there is a tendency that air voids accumulate in the coarse aggregate-cement paste interfacial region. That may weaken the aggregate- paste interface. There are micro-cracks in the cement paste and in the paste/aggregate interfacial region. Fractures are also found in some aggregate particles. The presence of unhydrated cement particles and the existence of slag particles are also observed.

2. Introduction

The Petrographic Laboratory of the FHWA Turner-Fairbank Highway Research Center was asked by the Structures Laboratory to examine a set of concrete cores retrieved from a bridge. Six concrete cores of 3 3/4-in. diameter, 5-½- to 6-in. long were received by the Petrographic Laboratory. The identification on the cores was as follows: VAB-1, VAB-2, VAB-3, VAB-4, VAB-5, and VAB-6.

3. Laboratory Procedures

Petrographic examination of the concrete samples was performed in accordance with ASTM C 856, “Standard Practice for Petrographic Examination of Hardened Concrete.” Sections were polished and examined using a stereomicroscope at magnifications up to

350×. Small rectangular blocks were cut from concrete samples. One surface of each block was polished, dried, placed on petrographic slide with low–viscosity epoxy resin, and reduced to a thickness of approximately 25 micrometers (0.001 in.). These thin sections were examined using a polarized–light microscope at magnification up to 400×, to determine aggregate mineralogy, paste characteristics, and microstructure.

Two ¾-inch thick slabs were sawn from the concrete cores. The slabs were ground and polished to obtain a smooth and plane surface. They were examined using a stereomicroscope at a magnification of 100×.

4. Findings

Eight thin section samples taken from the received cores were examined using the polarized light microscope. Two polished sections were examined using the stereomicroscope. The findings are summarized as follows:

Aggregate

The coarse aggregate in the concrete is crushed limestone. Coarse aggregate particles are angular, and the maximum size is about 3/4 inch. A small portion of the coarse aggregate is elongated or flaky particles. Preferential orientation of aggregate particles is not observed, nor is segregation.

The fine aggregate is mainly composed of quartz. The fine aggregate is natural sand and the particles appear rounded to angular.

Cement Paste

The cement is reasonably hydrated and the cement paste contains some unhydrated cement particles as seen under the microscope (Figure Q1-1). Slag particles are present in the concrete, also shown in Figure Q1-1.

Air Voids

Small, spherical air voids are observed in the concrete (Figure Q1-2), hence the concrete was air entrained. It was found that in some portions of the concrete, there is a tendency that air voids accumulate the coarse aggregate-cement paste interfacial region, as shown in Figure Q1-3 and Figure Q1-4.

Cracks

Isolated cracks in cement paste were sporadically observed in the concrete, as shown in Figure Q1-5. Conspicuous cracks were also found in the paste/aggregate interfacial region (Figure Q1-6). Typically, these cracks meander along the aggregate peripherals and extend into the paste (Figure Q1-7). Occasionally, cracked fine aggregate particles were also observed, such as the one shown in Figure Q1-8.

Gap/crack between coarse aggregate and paste, although very rare, was found in the concrete. Figure Q1-11 shows such a crack that is located underneath a piece of flaky coarse aggregate.

Secondary Deposit

Ettringite was observed in some air voids in the concrete. Ettringite crystals normally fill up a portion of a void, as shown in Figure Q1-9Figure and Figure Q1-10. There was no evidence of deterioration associated with the existence of the ettringite in the concrete. It is very common to see ettringite as secondary deposit in concrete.

5. Summary

The concrete appeared solid and sound, and no deterioration was shown visually. Microscopic examination revealed that the concrete was air entrained, but the entrained air voids were not well distributed in the concrete: In some portions of the concrete, there is a tendency that air voids accumulate the coarse aggregate-cement paste interfacial region. The cement paste in the interfacial region is porous. Both the accumulative air voids and the porous paste may weaken the bond between the paste and the aggregate. Sporadic gap/crack between coarse aggregate and cement paste was found in the concrete. Cracks were present in the cement paste and in the fine aggregate/paste interfacial region. Cracked fine aggregate particles were also found in the concrete.

Ettringite crystals have formed in some air voids, occupying a portion of the opening space of a void. There was no evidence of deterioration associated with the existence of the ettringite in the concrete. It is very common to see ettringite as secondary deposit in concrete.

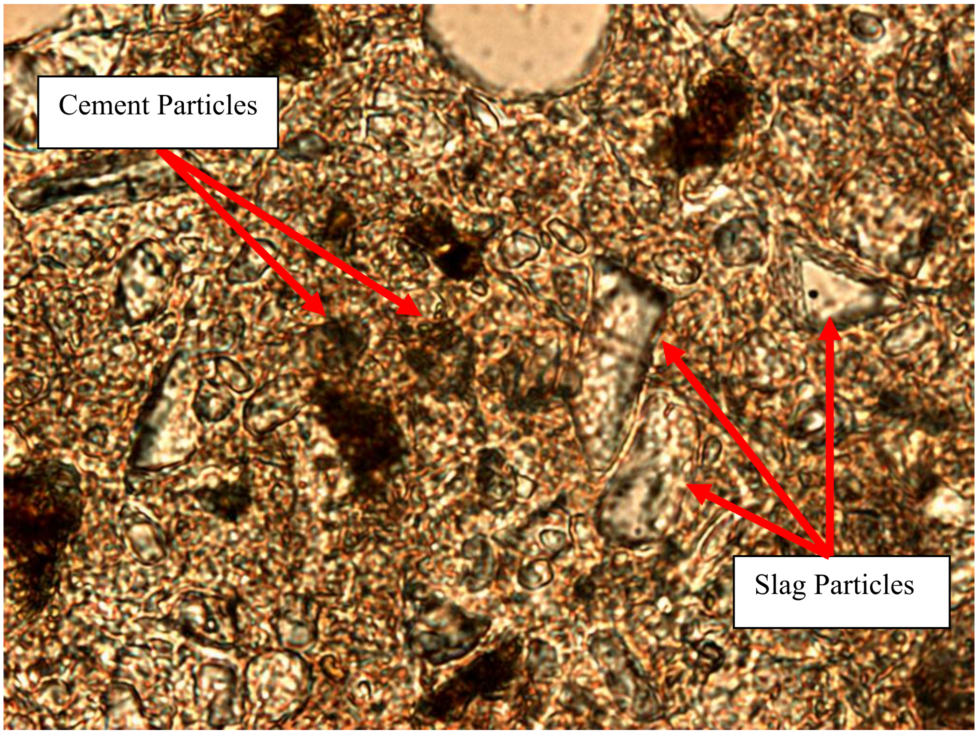


Figure Q1-1. Unhydrated cement and slag particles in paste. Width of field is 0.165 mm. Thin section image.

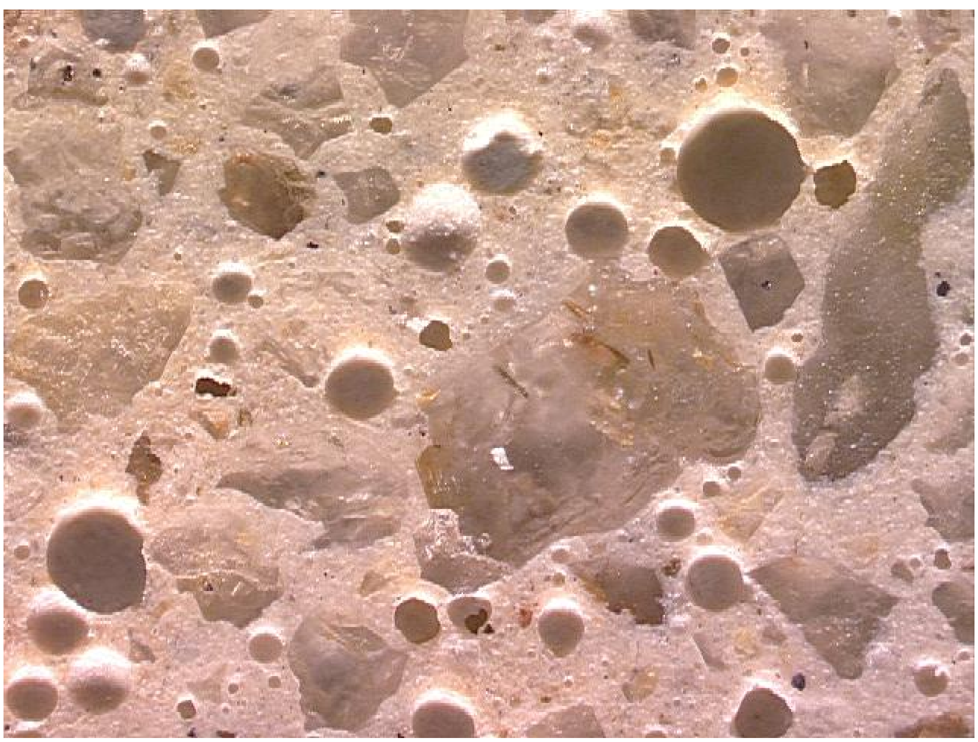


Figure Q1-2. Air voids in the concrete. Width of field is 4.0 mm. Polished concrete surface image.

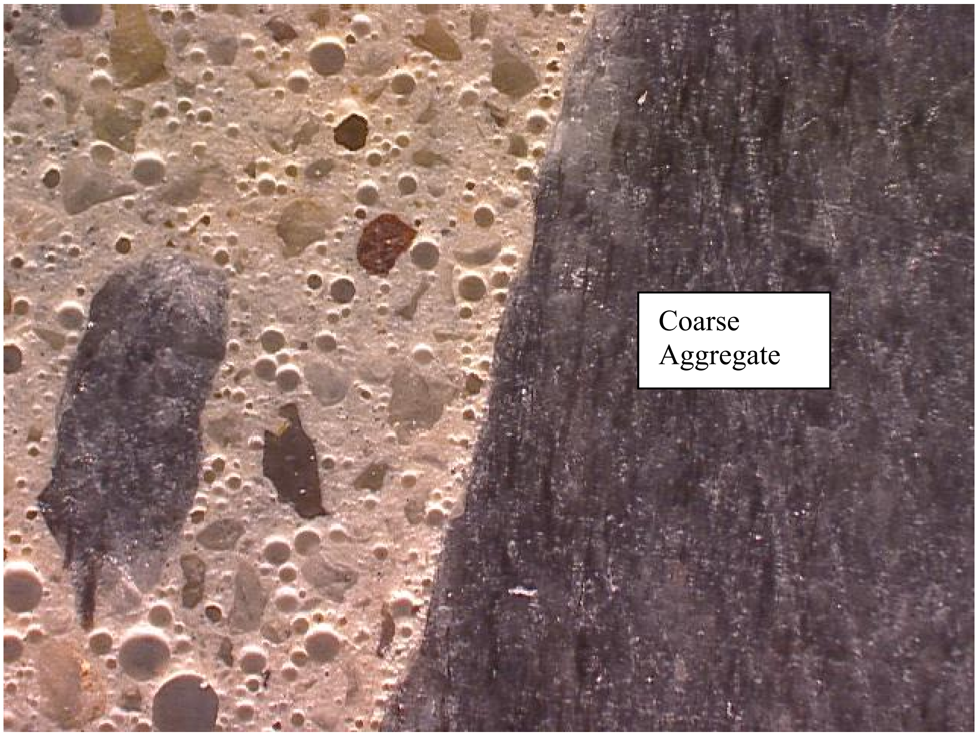


Figure Q1-3. Air voids line up around the coarse aggregate perimeter. Width of field is 6.5 mm. Polished surface image.

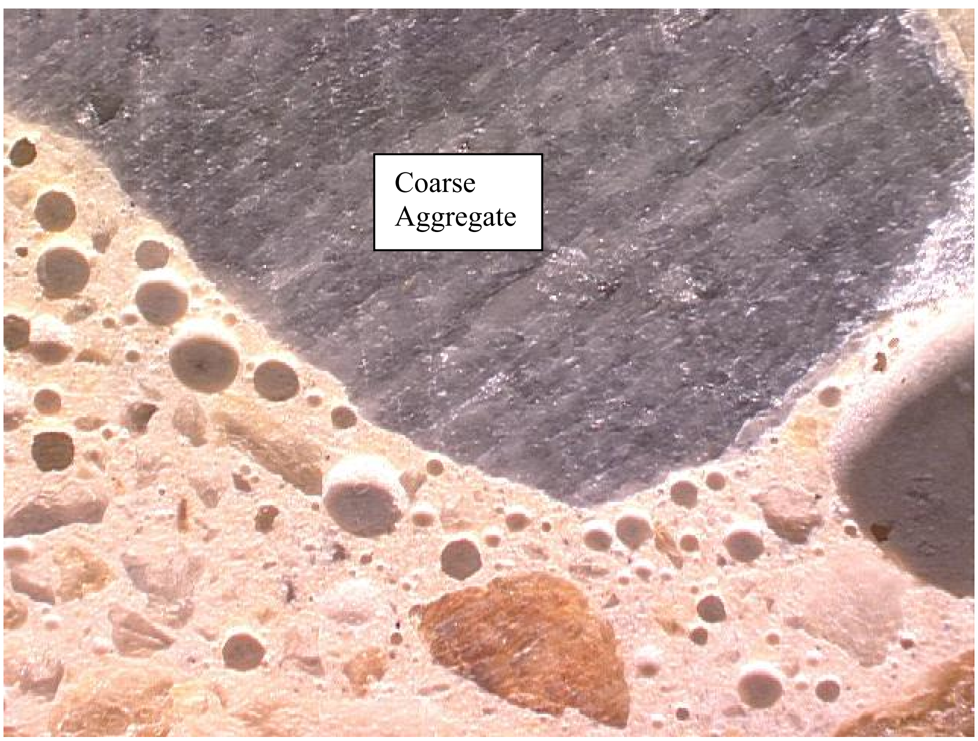


Figure Q1-4. Another image of air voids surrounding coarse aggregate. Width of field is 6.5 mm. Polished surface image.

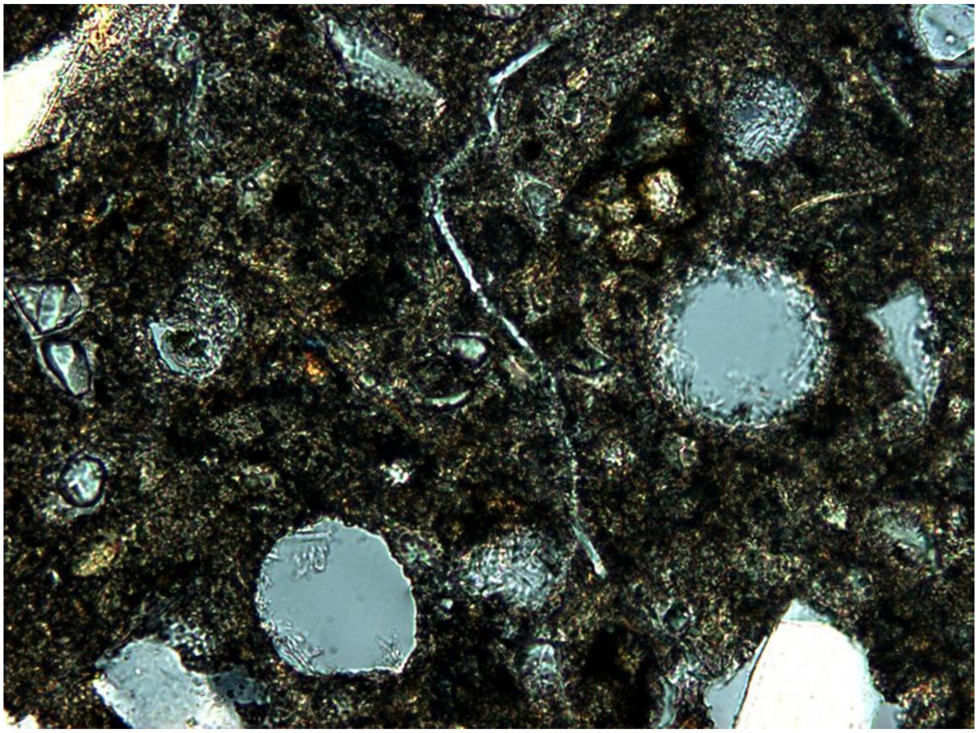


Figure Q1-5. Crack in cement paste. Width of field is 0.33 mm. Thin section image.

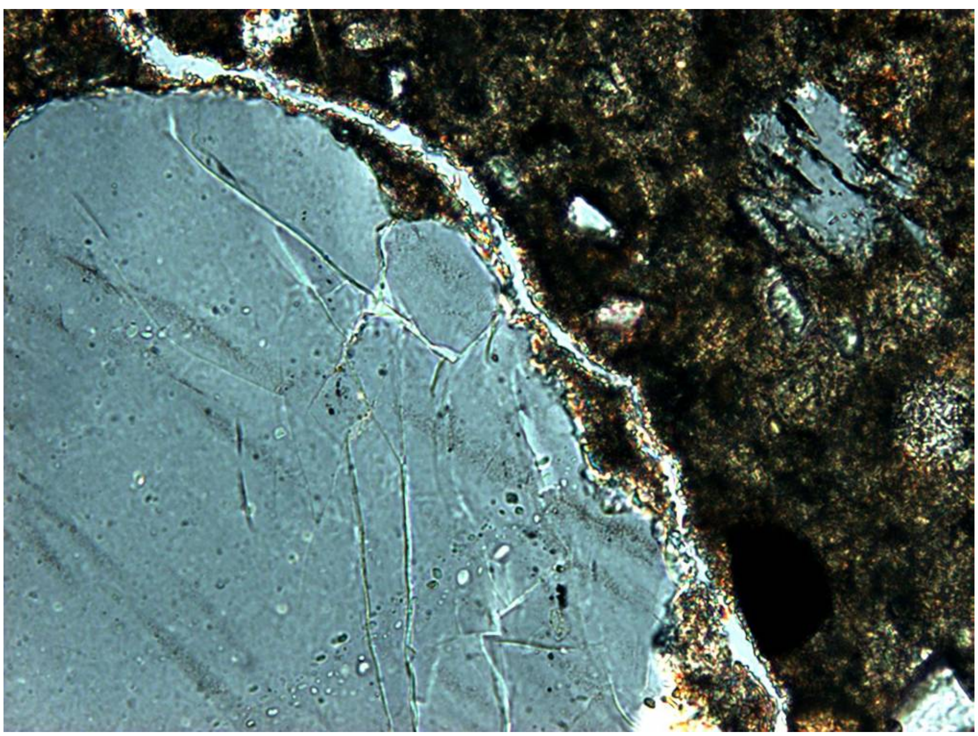


Figure Q1-6. Crack close to the fine aggregate/paste interface. Width of field is 0.33 mm. Thin section image.

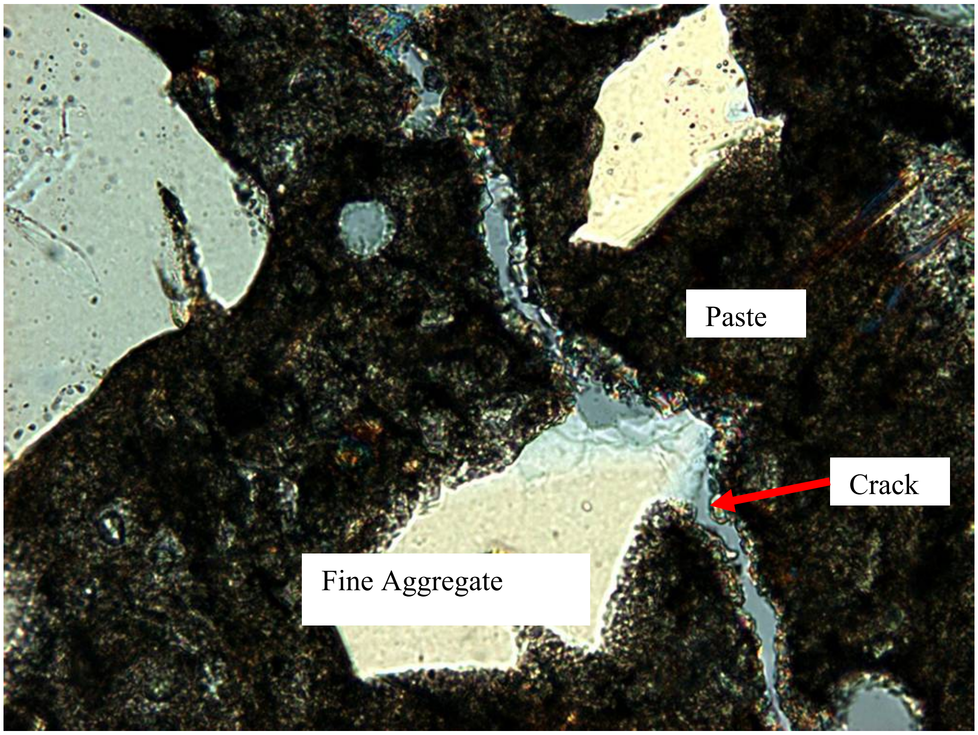


Figure Q1-7. Crack in the cement paste and along the aggregate/paste interface. Width of field is 0.33 mm. Thin section image.

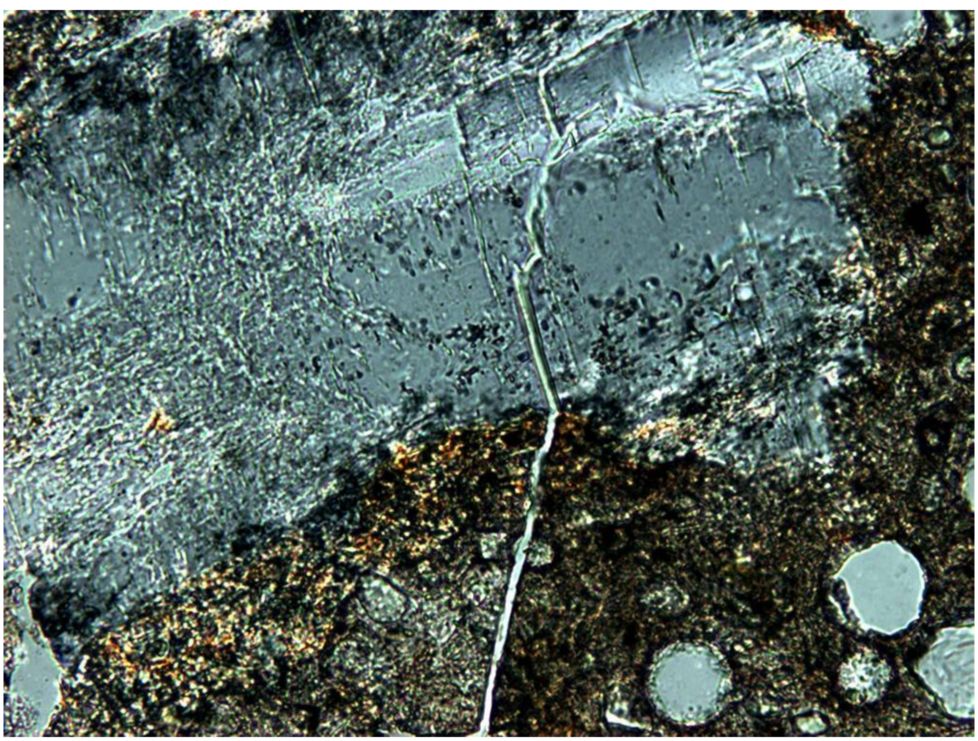


Figure Q1-8. Fracture in fine aggregate. Width of field is 0.33 mm. Thin section image.

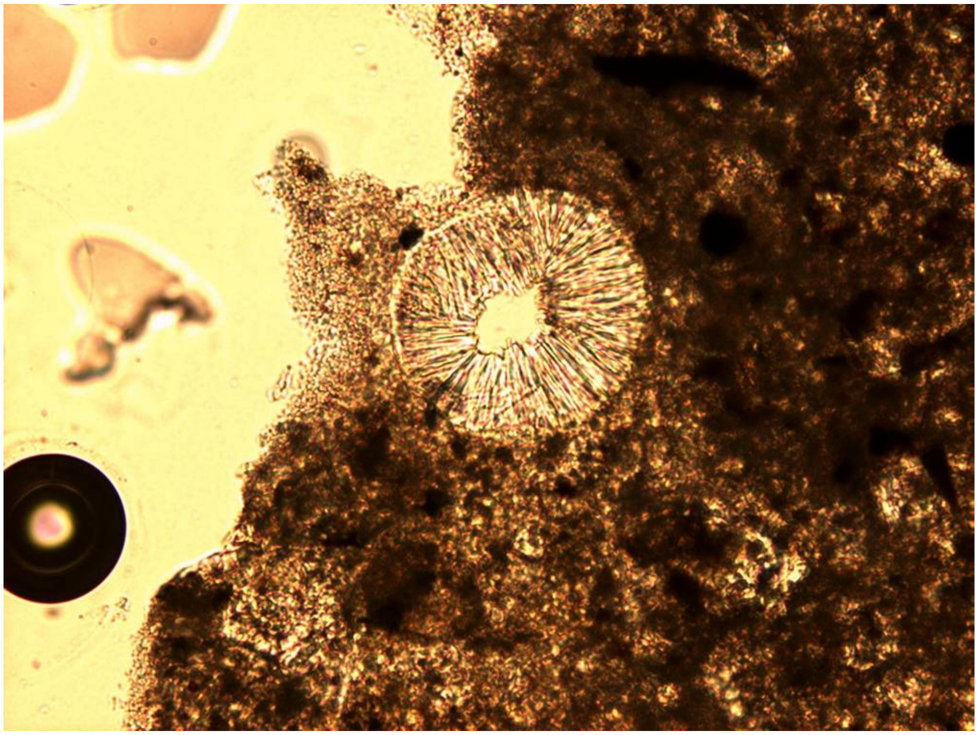


Figure Q1-9. Ettringite in an air void. Width of field is 0.33 mm. Thin section image.

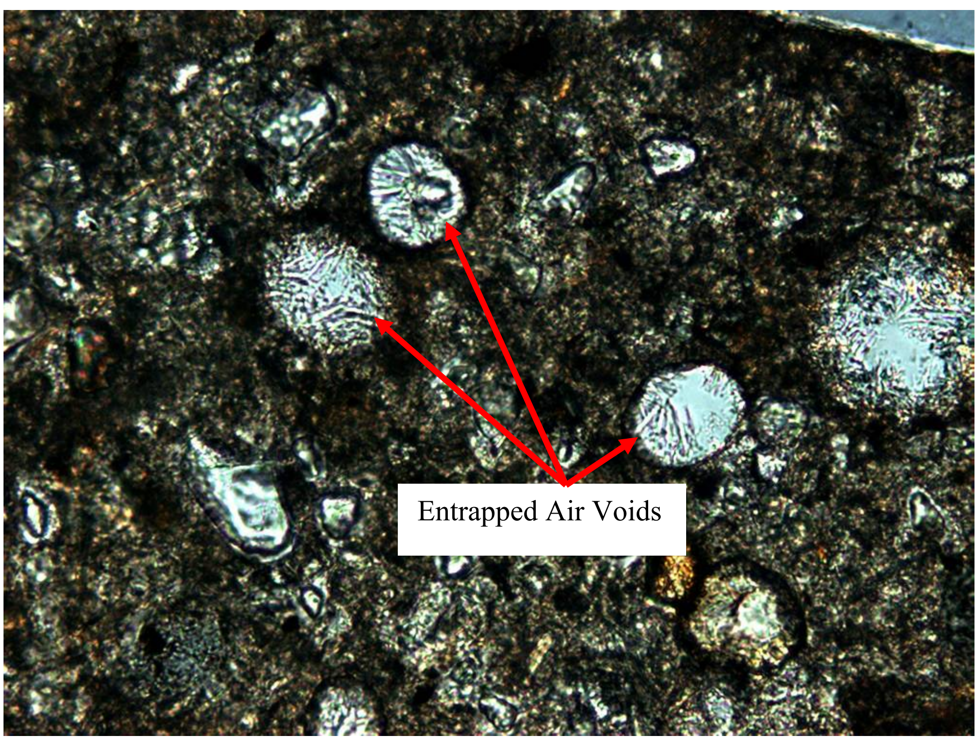


Figure Q1-10. Air voids filled with ettringite. Width of field is 0.33 mm. Thin section image.

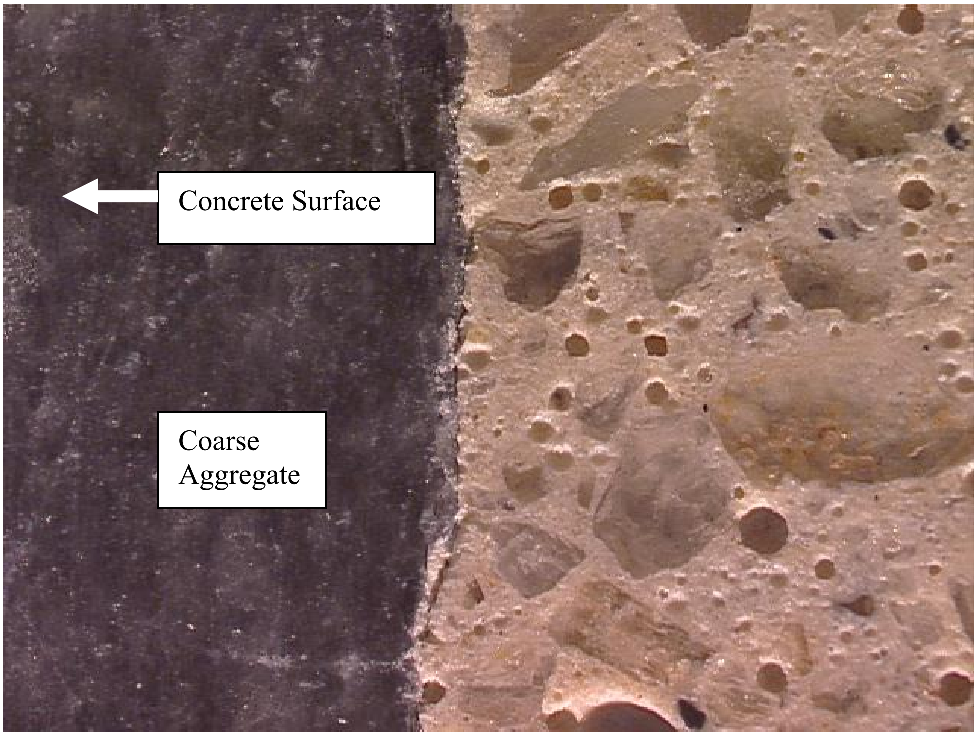


Figure Q1-11. Gap/crack underneath a flaky coarse aggregate. Width of field is 4.0 mm. Polished concrete surface image.

APPENDIX Q – Supplement 2

Route 40 Bridge, Brookneal, Virginia
Survey Checklist

Checklist

The following checklist is adapted from 201.1 R-2, and provided to facilitate a thorough survey. The definition of terms and associated photographs in 201.1 R-2, are utilized to standardize the reporting of the condition of the concrete in the structures.

1. Description of structure or pavement
   - 1.2 Name, location, type, and size Route 40 Over Falling River, Brookneal, Lynchburg District; (4) 80 ft prestressed simple spans, 320 ft.
   - 1.2 Owner, project engineer, contractor, when built VDOT, Built winter 1995, Opened 1996
   - 1.3 Design
     - 1.3.1 Architect and/or engineer
     - 1.3.2 Intended use and history of use
     - 1.3.3 Special features
   - 1.4 Construction ____________
     - 1.4.1 Contractor-general
     - 1.4.2 Subcontractors concrete placement
     - 1.4.3 Concrete supplier
     - 1.4.4 Agency responsible for testing
     - 1.4.5 Other subcontractors
   - 1.5 Photographs
     - 1.5.1 General view Yes
     - 1.5.2 Detailed close up of condition of area Yes
   - 1.22 Sketch map-orientation showing sunny and shady and well and poorly drained regions Drainage OK
2. Present condition of structure Date of Evaluation 11/20/02
   - 2.1 Overall alignment of structure Good
     - 2.1.1 Settlement None
     - 2.1.2 Deflection Negligible
     - 2.1.3 Expansion Normal
     - 2.1.4 Contraction Normal
   - 2.2 Portions showing distress (beams, columns, pavement, walls, etc., subjected to strains and pressures) Parapets have vert. cracks
   - 2.3 Surface condition of concrete
     - 2.3.1 General (good, satisfactory, poor, dusting, chalking, blisters) Good
     - 2.3.2 Cracks Longitudinal, Trans., Diag.
       - 2.3.2.1 Location and frequency See Fig. 2
       - 2.3.2.32 Type and size (see Definitions) See below

         Longitudinal Over Each Girder

         Width (from Crack comparator) 0.003-0.016 in.

Hairline	(Less than 1/32 in.)
Fine	(1/32 in. - 1/16 in.)
Medium	(1/16 – 1/8 in.)
Wide	(Greater than 1/8 in.)

Transverse Random
Width (from Crack comparator) 0.003-0.010 in.
Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)
Craze N/A
Width (from Crack comparator) in.
Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)
Map N/A
Width (from Crack comparator) in.
Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)
D-Cracking N/A
Width (from Crack comparator) in.
Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)
Diagonal At Joints
Width (from Crack comparator) 0.003-0.025 in.
Hairline (Less than 1/32 in.)
Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)

2.3.2.33 Leaching, stalactites None

2.3.3 Scaling None

2.3.3.1 Area, depth n/a

2.3.3.17 Type (see Definitions) n/a
Light (Less than 1/8 in.)
Medium (1/8 in. – 3/8 in.)
Severe (3/8 in. – 3/4 in.)
Very Severe (Greater than 3/4 in.)

2.3.4 Spalls and popouts Minor

2.3.4.1 Number, size, and depth 2) 2 x 3 x ½

2.3.4.17 Type (see Definitions) Small
Spalls
Small (Less than 3/4 in. depth)
Large (Greater than 3/4 in. depth)
Popouts
Small (Less than 3/8 in. diameter)

Medium (3/8 in. – 2 in. diameter)
Large (Greater than 2 in. diameter)

2.3.5 Extent of corrosion or chemical attack, abrasion, impact, cavitation ___None___
2.3.6 Stains, efflorescence ___None___
2.3.7 Exposed reinforcement ___None___
2.3.8 Curling and warping ___None___
2.3.9 Previous patching or other repair ___None___
2.3.10 Surface coatings ___Unknown___
- 2.3.10.1 Type and thickness ___n/a___
- 2.3.10.2 Bond to concrete ___n/a___
- 2.3.10.3 Condition ___n/a___

2.3.11 Abrasion ___None___
2.3.12 Penetrating sealers ___Unknown___
- 2.3.12.1 Type ___n/a___
- 2.3.12.2 Effectiveness ___n/a___
- 2.3.12.18 Discoloration ___n/a___

2.4 Interior condition of concrete (in situ and samples) ______
2.4.1 Strength of cores
2.4.2 Density of cores
2.4.3 Moisture content
2.4.4 Evidence of alkali-aggregate or other reactions ___None___
2.4.5 Bond to aggregate, reinforcing steel, joints ___Good___
2.4.6 Pulse velocity
2.4.7 Volume change
2.4.8 Air content and distribution
2.4.9 Chloride-ion content
2.4.10 Cover over reinforcing steel
2.4.11 Half-cell potential to reinforcing steel.
2.4.12 Evidence of reinforcement corrosion
2.4.13 Evidence of corrosion of dissimilar metals
2.4.29 Delaminations ___None___
- 2.4.29.1 Location ___n/a___
- 2.4.29.2 Number, and size ___n/a___

2.4.15 Depth of carbonation
2.4.16 Freezing and thawing distress (frost damage)
2.4.17 Extent of deterioration
2.4.33 Aggregate proportioning, and distribution

3. Nature of loading and detrimental elements

3.1 Exposure

3.1.1 Environment (arid, subtropical, marine, freshwater, industrial, etc.) ___Agricultural___
3.1.2 Weather-(July and January mean temperatures) ___75/34___
mean annual rainfall ___43.3 in.___
months in which 60 percent of it occurs ___Mar.-Sept.___
3.1.3 Freezing and thawing ___Yes___

	3.1.4	Wetting and drying	Yes
	3.1.14	Drying under dry atmosphere	No
	3.1.6	Chemical attack-sulfates, acids, chloride	None
	3.1.7	Abrasion, erosion, cavitation, impact	None
	3.1.8	Electric currents	None
	3.1.9	Deicing chemicals which contain chloride ions	Yes
	3.1.10	Heat from adjacent sources	No
3.2	Drainage		8) Deck Drains, Open w/ Minor Debris
	3.2.1	Flashing	n/a
	3.2.2	Weepholes	n/a
	3.2.3	Contour	n/a
	3.2.4	Elevation of drains	Good
3.3	Loading		
	3.3.1	Dead	No distress
	3.3.2	Live	No distress
	3.3.3	Impact	No distress
	3.3.4	Vibration	None
	3.3.5	Traffic index	Moderate
	3.3.6	Other	n/a
3.4	Soils (foundation conditions)		No settlement
	3.4.1	Compressibility	
	3.4.2	Expansive soil	
	3.4.3	Settlement	
	3.4.4	Resistivity	
	3.4.5	Evidence of pumping	
	3.4.6	Water table (level and fluctuations)	

4.	Original condition of structure			
	4.1	Condition of formed and finished surfaces		
		4.1.1	Smoothness	Good
		4.1.2	Air pockets ("bugholes")	Negligible
		4.1.3	Sand streaks	None
		4.1.4	Honeycomb	None
		4.1.5	Soft areas (retarded hydration)	None
		4.1.6	Cold joints	None
		4.1.37	Staining	None
		4.1.38	Sand pockets	None
	4.2	Defects		
		4.2.1	Cracking	Yes
			4.2.1.1 Plastic shrinkage	
			4.2.1.2 Thermal shrinkage	
			4.2.1.3 Drying shrinkage	
		4.2.17	Curling	None
5.	Materials of Construction			Good
6.	Construction Practices			Good

APPENDIX R

Virginia Avenue Bridge, Richlands, Virginia

HIGH PERFORMANCE CONCRETE BRIDGE DECK INVESTIGATION

Virginia Avenue Bridge Town of Richlands, Virginia

I. BACKGROUND

The Virginia Avenue Bridge over Clinch River, located in the Town of Richlands in Tazewell County, Virginia, was constructed in late 1997. The structure is 148-ft long and 40-ft wide (see photos 1 through 3, and Figure 1). It carries one northbound lane and one southbound lane of Virginia Avenue. The structure consists of 8-½-in. thick concrete deck with stay-in-place forms on five 74-ft long simple span concrete pre-stressed superstructure, on one concrete pier and two concrete abutments. The structure was built with no skew at either abutment and at the pier. Five precast AASHTO Type III girders, on 8-ft 9-in. and 9-ft 3-in. centers support each span. The concrete stub abutments are separated from Clinch River with loose riprap slope protection. The concrete pier is comprised of cast-in-place concrete caps on cast-in-place pier stems.

The girders and deck were constructed with high performance concrete (HPC). The factors that led to the use of HPC in this bridge included use of fewer girders and a more durable structure. If HPC was not used, two more lines of girders would have been required. The cost of HPC in this bridge was \$60.43 per square ft, whereas the average cost for similar bridges constructed that year was \$69 per ft^2. This represents a potential saving of \$50,730 for this bridge.

II. SCOPE OF SERVICES

Professional Service Industries Inc. (PSI) is under contract with the Federal Highway Administration (FHWA) to conduct bridge deck inspections for HPC bridges. Our scope of services on this bridge included a series of tasks and sub-tasks, which are described as follows:

- Collect all available information relevant to the construction of each bridge, including;
    - Deck Concrete Properties
    - Specified Deck Concrete Construction Procedures
    - Approved Concrete Mixture Proportions
    - Measured Properties from QC
    - Other Measured Properties
    - Actual Method of Deck Placement
    - Average Daily Traffic (ADT)
    - Exposure Condition of the Bridge
    - Any Performed Maintenance
    - Any Inspection Reports
- Visually inspect the bridge, and obtain the following:

- General condition of the deck top surface
- Determination of the maximum crack width
- General condition of the deck underside
- General condition of the girders
- Photograph areas of significant deterioration
- Prepare drawings locating defects
- Extract 6 concrete core samples

III. COMPILATION of BRIDGE CONSTRUCTION INFORMATION

Information sources used for this report include bridge drawings, inspection reports, bridge summary data sheets, and technical information contained in FHWA's "Compilation and Evaluation of Results from High Performance Concrete Bridge Projects" version 3.

Concrete Properties

Table 1 lists the specified properties for concrete used in the girders and decks. Note that the minimum and maximum percentages of mineral admixtures only apply if the materials are used.

TABLE 1: Specified Concrete Properties for Girders and Decks

	Girders	Deck
Minimum Cementitious Materials Content:	635 lb/yd^3	635 lb/yd^3
Max. Water/Cementitious Materials Ratio:	0.40	0.45
Min., Max. Percentage of Fly Ash:	20, 25	20, 25
Min., Max. Percentage of Silica Fume:	7, 10	7, 10
Min., Max. Percentage of GGBFS:	35, 50	35, 50
Maximum Aggregate Size:	1 in	1 in
Slump:	0-4 in (1)	2-4 in (1)
Air Content:	4.5 ± 1.5% (2)	6.5 ± 1.5% (2)
Compressive Strength	Girders	Deck
— Release of Strands:	6800 psi	—
— Design at 28 days:	10,000 psi	5000 psi
Chloride Permeability (AASHTO T 277), 28 day:	1500 coulombs(3)	2500 coulombs(3)
ASR or DEF Prevention:	ASR (4)	ASR (4)
Freeze-Thaw, Deicer Scaling, and Abrasion Resistance:	Not specified	Not specified

NOTES:

(2) Maximum of 7 in. when a high-range water-reducing admixture (HRWR) is used.
(3) Target air content is increased by 1 percent when a HRWR is used.
(4) Curing procedure of one week at 73 °F and three weeks at 100 °F with the AASHTO T 277 test.
(5) Cement shall be Type II with a maximum alkali content of 0.40% or Type I-P, unless otherwise specified in the contract. Fly ash or granulated iron blast-furnace slag shall not be added to concrete when Type I-P cement is used. Fly ash, granulated iron blast-furnace slag, silica fume, or other VDOT approved mineral admixtures shall be used with Types I, II (if above 0.40% alkali content), or III cements.

Specified Deck Concrete Construction Procedures

For the cast-in-place deck, moist curing was required. It was specified that the concrete shall be covered with wet burlap and plastic sheeting for 7 days. The 4 x 8 in. concrete cylinders shall be moist cured. For QA/QC purposes, air content, slump and concrete temperature were required to be measured.

Approved Concrete Mix Proportions

The approved concrete mix proportions are listed in Table 2.

TABLE 2: Approved Concrete Mix Proportions for Girders and Cast-in-Place Deck

	Girders	Cast-in-Place Deck
Cement Brand:	Not available	Not available
Cement Type:	I	II
Cement Composition:	Not available	Not available
Cement Fineness:	Not available	Not available
Cement Quantity:	752 lb/yd^3	560 lb/yd^3
GGBFS Brand:	—	—
GGBFS Quantity:	—	—
Fly Ash Brand:	—	Not available
Fly Ash Type:	—	F
Fly Ash Quantity:	—	140 lb/yd^3
Silica Fume Brand:	Not available	—
Silica Fume Quantity:	75 lb/yd^3	—
Fine Aggregate Type:	Crushed limestone	Natural sand
Fine Aggregate FM:	3.00	2.80
Fine Aggregate SG:	2.75	2.65
Fine Aggregate Quantity:	1350 lb/yd^3	1004 lb/yd^3
Coarse Aggregate, Max. Size:	½ in	1 in
Coarse Aggregate Type:	No. 7 limestone	No. 57 quartzite
Coarse Aggregate SG:	2.76	2.65
Coarse Aggregate Quantity:	1671 lb/yd^3	1724 lb/yd^3
Water:	235 lb/yd^3	315 lb/yd^3
Water Reducer Brand:	—	—
Water Reducer Type:	—	—
Water Reducer Quantity:	—	—
High-Range Water-Reducer Brand:	Rheobuild 1000	—
High-Range Water-Reducer Type:	A and F	—
High-Range Water-Reducer Quantity:	207 fl oz/yd^3	—

continued

TABLE 2 (continued): Approved Concrete Mix Proportions for Girders and Cast-in-Place Deck

	Girders	Cast-in-Place Deck
Retarder Brand:	Pozzolith 122 R	Daratard 17
Retarder Type:	D	D
Retarder Quantity:	25 to 30 fl oz/yd^3	21 fl oz/yd^3
Corrosion Inhibitor Brand:	—	—
Corrosion Inhibitor Type:	—	—
Corrosion Inhibitor Quantity:	—	—
Air Entrainment Brand:	MBAE-90	Daravair 1000
Air Entrainment Type:	Anion surfactant	Saponified rosin
Air Entrainment Quantity:	7 fl oz/yd^3	5 fl oz/yd^3
Water/Cementitious Materials Ratio:	0.28	0.45

Measured Properties from QC

Table 3 summarizes the measured concrete properties from QC tests and curing procedures.

TABLE 3: Measured Concrete Properties from QC

	Girders	Deck (1)
Cement Composition:	Not available	Not available
Actual Curing Procedure:	Steam	After screeding, a curing compound was applied. When the surface was hard enough to walk on, wet burlap, covered with white plastic sheeting was applied and remained in place for seven days
Average Slump:	6.6 in	3.6 in
Maximum Temperature:	161 ^{o}F	—
Average Air Content:	4.4%	5.8%
Unit Weight:	—	—
Average Compressive Strength:	8840 psi at 18 hours 11,200 psi at 28 days	5400 psi at 28 days
Chloride Permeability:		1457 coulombs at 28 days
Curing Procedure for Cylinders:	Alongside girders on precasting bed	—

NOTE: (1) Concrete samples taken before pumping.

Other Measured Properties

Table 4 shows the measured fresh concrete properties, including air content, slump, concrete temperature, and air temperature.

TABLE 4: Air Content, Slump, Concrete Temperature, and Air Temperature

Batch No.		1	2
Air, %	Before Pumping	5.5	6.0
	After Pumping	3.5	4.5
Slump, in.	Before Pumping	3.7	—
Concrete Temp. at Time of Placement, ^{o}F		72	78
Air Temperature, ^{o}F		57	60

In addition to the measurement for the fresh concrete, concrete cylinders were prepared and cured for both batches of concrete. The tested properties included: compressive strength, modulus of elasticity, flexural strength, splitting tensile strength, chloride permeability, and shrinkage. Table 5 summarizes the measured concrete properties.

TABLE 5: Measured Concrete Properties from the Two Batches

Test	Specimen			Batch No.		
	Size	Age	No.	1	2	Average
Compressive Strength (1), psi	4x8 in.	7 d	3	4160	4360	4260
		28 d	3	6150	6380	6265
		56 d	3	6630	6790	6710
		1 yr	3	8115	7995	8055
Modulus of Elasticity (2), ksi	4x8 in.	1 yr	3	5120	5360	5240
Splitting Tensile Strength (3), psi	4x8 in.	28 d	3	570	645	608
Chloride Permeability (4), coulombs	2x4 in.	28 d	2	1261	1375	1318

NOTES:(1) AASHTO T 22 with neoprene pads in steel end caps.
(2) ASTM C 469.
(3) AASHTO T 198 (ASTM C 496).
(4) AASHTO T 277 (ASTM C 1202). Cured one week at 73 ^{o}F and three weeks at 100 ^{o}F.

ACTUAL METHOD OF DECK PLACEMENT

Construction of the deck occurred in October 1997, with the concrete for the deck delivered by truck and pumped to the deck surface. Surface finishing consisted of vibratory screed followed by a roller screed. Transverse grooves were tined into the surface of the uncured concrete. The edges were consolidated with immersion type vibrators and hand floated. The deck was cured using water soaked burlap covered with white plastic for seven days. No milling operations were performed on this deck. The ambient temperature at the time of placement was generally reported in the mid 60's ^{o}F.

AVERAGE DAILY TRAFFIC (ADT) & AVERAGE DAILY TRUCK TRAFFIC (ADTT)

The district identifies the 1994 ADT for this bridge as 161 vehicles per day, with 2% trucks. The Structure Inventory Data Sheet indicates that an ADT of 241 is anticipated in the year 2022. While the PSI inspection crew was on-site 21 cars were noted per hour. Also, one truck per hour was noted during this time period. This represents an ADT of approximately 500, and an ADTT of 25.

EXPOSURE CONDITION OF THE BRIDGE

The surrounding area to the north is the Town of Richlands, to the south is generally residential property. The National Weather Service reports that the mean July temperature is 72°F, while the mean January temperature is 32°F. The mean annual rainfall is 40.4 in., 64% of the annual amount falls from March-September. The bridge is exposed to freezing and thawing as well as wetting and drying on a seasonal cycle basis.

PERFORMED MAINTENANCE

The 2000 inspection report indicates that welds on the expansion bearing washers, which were noted in the 1998 inspection report, had been removed. The 2002 inspection report indicates that no work had been done.

INSPECTION REPORTS

Bridge inspection reports dated 3/5/1998, 2/2/2000 and 1/15/2002 were identified for this bridge. Deck cracks ranging in width from 0.016 in. to 0.030 in., totaling 160 linear feet were noted over the pier, in the 2000 inspection. The same cracks are noted again in the 2002 report, with random transverse cracks up to 0.030 in. noted on the sidewalks.

The 1998 inspection report summarizes the Condition of Structure as "GOOD- Expansion bearing washers welded on expansion bearings at both abutments; and heavy debris lodged in channel against upstream end of pier."

The 2000 inspection report summarizes the Condition of Structure as "GOOD- Deck, approach slabs, and breastwall at both abutments have cracks; and heavy debris lodged in channel against upstream end of pier."

The 2002 inspection report summarizes the Condition of Structure as "GOOD- Deck, sidewalks, approach slabs, and breastwall at both abutments have cracks. One anchor bolt nut is missing. Heavy debris lodged in channel against upstream end of pier."

IV. BRIDGE DECK INSPECTION

The bridge deck received a close visual inspection in April 29 and 30, 2003. The findings of this inspection are summarized as follows.

GENERAL CONDITION OF THE DECK TOP SURFACE

Defects in the top surface include longitudinal cracks, transverse cracks, one diagonal crack, and a small gouge.

Longitudinal Cracks: Longitudinal cracks were found primarily over each of the girder lines. Figure 2 illustrates the 410 ft of longitudinal cracks that were identified on the top surface of the deck (see Table 1). The longitudinal crack density was 0.0923 ft/ft^2. The crack widths ranged from 0.008 to 0.022 in. for the 62 cracks (see photos 4 through 10).

TABLE 6: Longitudinal Cracks

Span	Number	Total Length of Cracks (ft.)	Deck Area (ft^2)	Crack Density (ft/ft^2)
A	27	125	2220	0.0563
B	35	285	2220	0.1284
Cumulative	62	410	4440	0.0923

Transverse Cracks: Transverse cracks were found throughout the length of the bridge. Figure 2 illustrates the 107 ft of transverse cracks that were identified on the top surface of the deck (see Table 2). The transverse crack density was 0.0241 ft/ft^2.The crack widths ranged from 0.006 to 0.014 in. for the 15 cracks (see photos 11 and 12).

TABLE 7: Transverse Cracks

Span	Number	Total Length of Cracks (ft.)	Deck Area (ft^2)	Crack Density (ft/ft^2)
A	8	57	2220	0.0257
B	6	50	2220	0.0225
Cumulative	14	107	4440	0.0241

Diagonal Cracks: One Diagonal crack was found at the expansion joint at the south abutment. Figure 2 illustrates the 4 ft of diagonal cracking that was identified on the top surface of the deck (see Table 3). The diagonal crack density was 0.0009 ft/ft^2. The crack width ranged was measured as 0.010 in. for the crack (see photo 13).

TABLE 8: Diagonal Crack

Span	Number	Total Length of Cracks (ft.)	Deck Area (ft^2)	Crack Density (ft/ft^2)
A	1	4	2220	0.0018
B	0	0	2220	0.0000
Cumulative	1	4	4440	0.0009

In addition to the different types of cracking noted, a few isolated small defects were found. These defects included a small gouge in Span B, and transverse cracks in the sidewalks.

Small gouge: A small gouge is located in Span B. The gouge is 12-in. long by 3-in. wide by ½-in. deep (see photo 14).

Sidewalk cracks: The west sidewalk has thirty-three 3.5-ft long cracks, while the east has twenty-eight 4.5-ft long cracks. The cracks range from 0.010 in. to 0.020 in. in width, and are spaced on 2- to 4-ft centers (see photo 15).

Deck drains: No deck drains exist on this structure.

MAXIMUM CRACK WIDTH

The maximum width of longitudinal cracks and transverse cracks was measured to be 0.022 in. and 0.014 in., respectively. The maximum width of the diagonal crack was measured to be 0.010 in. According to ACI 201, these crack widths are classified as hairline cracks.

GENERAL CONDITION OF THE DECK UNDERSIDE

The undersurface of the deck was not visible due to the presence of stay-in-place deck forms. However, the exterior cantilever portions of the deck were exposed, which exhibit no signs of distress. Photo 16 shows a general view of the underside of the deck.

GENERAL CONDITION OF THE GIRDERS

The girders were inspected from the ground, without the aide of any access equipment. No signs of distress were noted on any of the girders. Photos 17 through 19 show general views of the girders.

EXTRACT CORES

Six cores were retrieved from the deck, Figure 1 illustrates their locations. The locations were selected to distribute the samples along each shoulder of the bridge since Virginia DOT had requested that the coring and patching operation avoid the traveled lanes. The cores were labeled VAR-1 through VAR-6 (see Table 4.) and transferred to FHWA on May 27, 2003 for further investigation.

TABLE 9: Core Dimensions

Sample	VAR-1	VAR-2	VAR-3	VAR-4	VAR-5	VAR-6
Diameter (in.)	3-¾	3-¾	3-¾	3-¾	3-¾	3-¾
Length (in.)	5-¼	4-¾	5-½	4-¾	5	5

PRELIMINARY CONCLUSIONS

The Virginia Avenue Bridge was constructed in late 1997. It carries one northbound lane and one southbound lane of Virginia Avenue over Clinch River in the Town of Richlands, in Tazewell County. The structure consists of 8-½-in. thick concrete deck with stay-in-place forms on five 74-ft long simple span concrete pre-stressed superstructure, on one concrete pier and two concrete abutments.

Previous inspection indicated cracks on the bridge deck. In the 2000 inspection, a total of 160 linear feet deck cracks over the pier were documented, ranging in width from 0.016 in. to 0.030 in. The same cracks were noted again in the 2002 report, with random transverse cracks up to 0.030 in. on the sidewalks.

The bridge deck received a close visual inspection on April 29 and 30, 2003. Defects in the top surface include longitudinal cracks, transverse cracks, one diagonal crack, and small gouges.

Longitudinal cracks were found primarily over each of the girder lines. A total of 62 longitudinal cracks, with an accumulative linear length of 410 ft, were identified on the deck surface. The crack widths ranged from 0.008 to 0.022 in. Transverse cracks were found throughout the length of the bridge. A total of 15 transverse cracks, with an accumulative linear length of 107 ft, were identified on the deck surface. The crack widths ranged from 0.006 to 0.014 in. One Diagonal crack was found at the expansion joint at the south abutment. The 4 ft of diagonal crack had a width of 0.010 in.

Other defects found on the deck included a small gouge in Span B and transverse cracks in the sidewalks. The small gouge was located in Span B, with a dimension of 12-in. long, 3-in. wide, and ½-in. deep. The west sidewalk had thirty-three 3.5-ft long cracks, while the east had twenty-eight 4.5-ft long cracks. The cracks range from 0.010 in. to 0.020 in. in width, and were spaced on 2- to 4-ft centers.

Petrographic examination was performed on six concrete cores that were retrieved from the bridge deck. The concrete cores were 3-3/4-in. in diameter and 5- to 6-in. long. The identification on the cores was as follows: VAR-1, VAR-2, VAR-3, VAR-4, VAR-5, and VAR-6. Visual inspection of the concrete cores revealed no defects.

The coarse aggregate in the concrete was crushed stone of sedimentary rocks (sandstone and quartzite) and metamorphic rocks (hornfels and mylonite). The maximum size, as measured from the prepared samples, was about 3/4 inch. A portion of the coarse aggregate particles was elongated or flat pieces. The natural sand fine aggregate was mainly composed of quartz, and the particles appeared rounded to angular.

The cement was reasonably hydrated with respect to the age of the concrete. The cement paste contained some unhydrated cement particles. Isolated cracks in cement paste were sporadically observed.

The concrete was air entrained. Small, spherical air voids were well distributed. Entrapped air voids were also present in the concrete, often underneath coarse aggregate particles.

Gaps and cracks formed in the paste-aggregate interfacial region in the concrete. The coexistence of some air voids in the interfacial region further weakened the bond between the coarse aggregate and the cement paste. In general, the gaps/cracks were often present at the aggregate-paste interface on the underside of a coarse aggregate particle, especially that of an elongated or flat piece. Thus, internal bleeding was believed to be the major cause of the gaps and cracks in the interfacial region. This leads to the formation of very porous paste in the transition zone.

Ettringite was observed in some air voids in the concrete. Very often, ettringite crystals filled up a portion of a void. Occasionally, voids fully filled with ettringite were also found in the concrete.

Respectfully Submitted,
Professional Service Industries, Inc.
Structural Investigation & Petrography Department

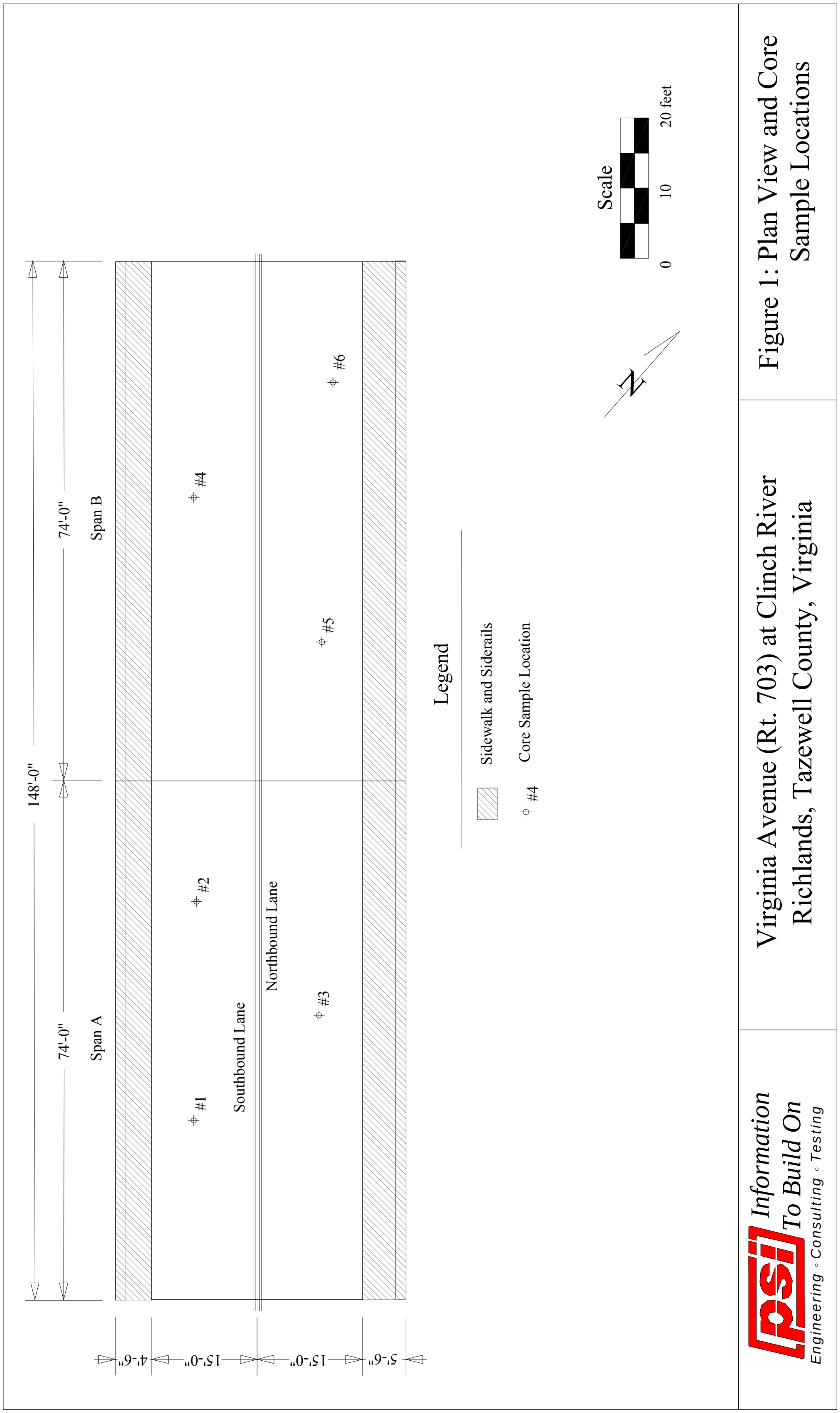
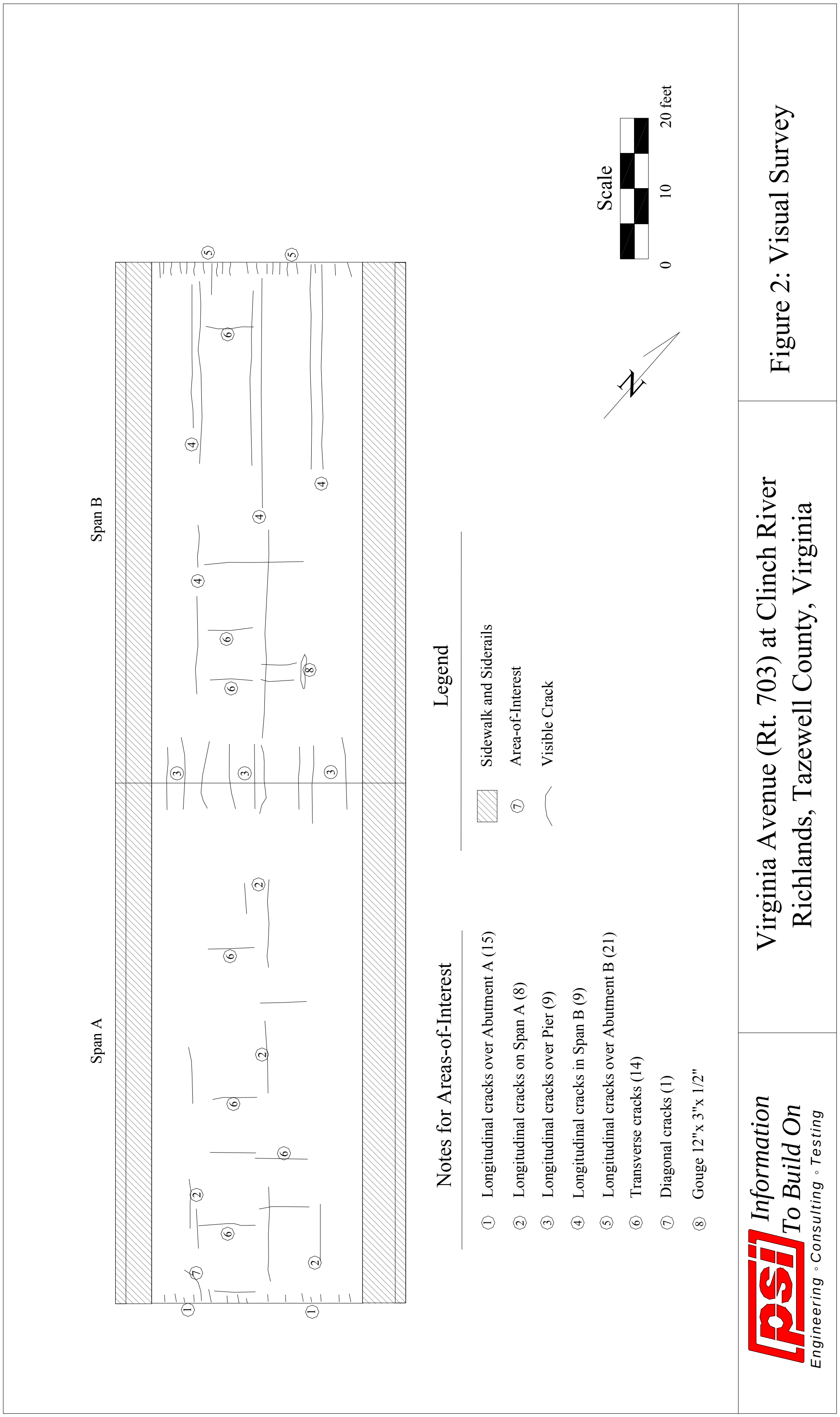


Figure 1: Plan View and Core Sample Locations


Span A
Span B
Notes for Areas-of-Interest
① Longitudinal cracks over Abutment A (15)
② Longitudinal cracks on Span A (8)
③ Longitudinal cracks over Pier (9)
④ Longitudinal cracks in Span B (9)
⑤ Longitudinal cracks over Abutment B (21)
⑥ Transverse cracks (14)
⑦ Diagonal cracks (1)
⑧ Gouge 12"x 3"x 1/2"
Legend
Sidewalk and Siderails
Area-of-Interest
Visible Crack
N
Scale
0
10
20 feet
Information To Build On
Engineering • Consulting • Testing
Virginia Avenue (Rt. 703) at Clinch River
Richlands, Tazewell County, Virginia
Figure 2: Visual Survey

Photographic Documentation

Photo 1: General view of the west elevation. Looking upstream.

Photo 2: View looking at the north approach.

Photo 3: View looking at the south approach.

	Photo 4: General view of the longitudinal cracks at the south end of the bridge.
	Photo 5: General view of the longitudinal cracks at the north end of the bridge.

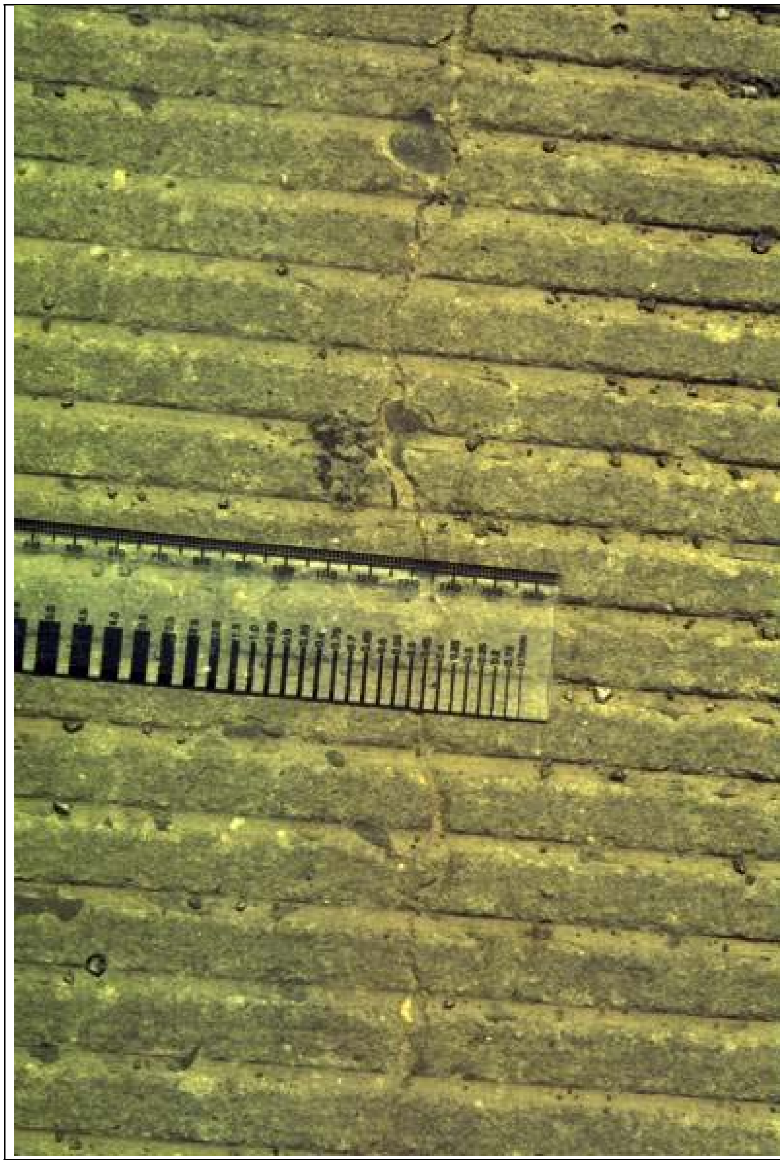


Photo 6: Close-up view of the longitudinal cracks. The width of the crack is 0.016 in.

Photo 7: View of the longitudinal cracks on the south bound.

	Photo 8: Close-up view of the longitudinal cracks. The width of the crack is 0.022 in.
	Photo 9: General view of the longitudinal cracks over the pier.

Photo 10: General view of the longitudinal cracks on the north bound.

Photo 11: View of transverse cracks.

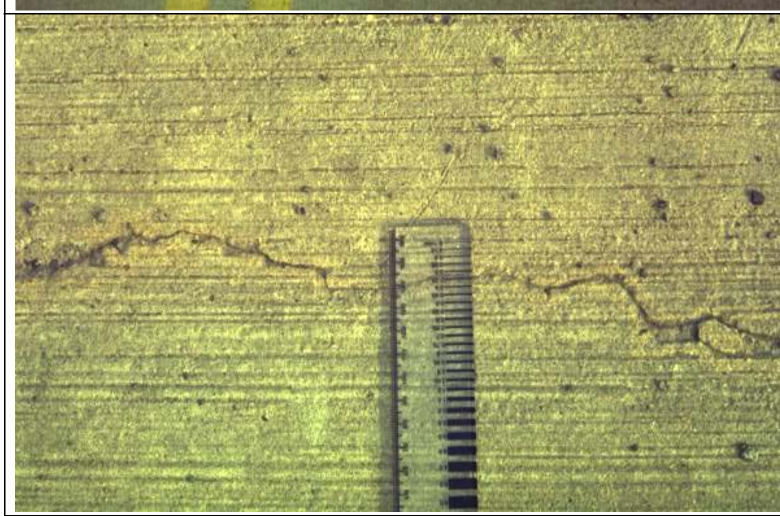


Photo 12: Close-up view of the transverse cracks. The width of the crack is 0.014 in.

	Photo 13: View of the diagonal crack. The width of the crack is 0.010 in.
	Photo 14: A small gouge with a dimension of 12-in. x 3-in. x ½-in.
	Photo 15: Cracks on the sidewalk.

Photo 16: View of the underside of the deck showing the stay-in-place forms.

Photo 17: View of the girders A through E, Span A, Pier 1 (fascia girder A is on the left).

Photo 18: View of the girders A and B, Span B, abutment B (Fascia girder A is on the left).

Photo 19: View of the girders B and C, abutment B (interior girder B on the left).

APPENDIX R – Supplement 1

Virginia Avenue Bridge, Richlands, Virginia
Petrographic Report

PETROGRAPHIC EXAMINATION OF CONCRETE CORES FROM A VIRGINIA BRIDGE (VAR)

A report submitted to the Structures Laboratory at TFHRC

Rongtang Liu
Petrographic Laboratory
FHWA TFHRC

March 26, 2005

1. Abstract

Six concrete cores were received from the Structures Laboratory of the Turner-Fairbank Highway Research Center for petrographic examination. Reportedly, the concrete cores were collected from a concrete bridge.

Petrographic examination was performed on samples using optical microscopes. Thin sections, as well as polished slabs, were made from the concrete cores for microscopic examination.

Visual inspection of the concrete cores revealed no defects, and the concrete appeared solid and sound. The findings from microscopic examination indicate that the concrete has entrained air voids. The hydration of the cement was reasonable. The presence of unhydrated cement particles was also observed in the cement paste. Ettringite as secondary deposit formed in air voids. Gaps and cracks were present in the paste/aggregate interfacial region, as well as in the paste.

2. Introduction

The Petrographic Laboratory of the FHWA Turner-Fairbank Highway Research Center was asked by the Structures Laboratory to examine a set of concrete cores retrieved from a bridge. Six concrete cores of 3-3/4-in. diameter, 5- to 6-in. long were received by the Petrographic Laboratory. The identification on the cores was as follows: VAR-1, VAR-2, VAR-3, VAR-4, VAR-5, and VAR-6

3. Laboratory Procedures

Petrographic examination of the concrete samples was performed in accordance with ASTM C 856, “Standard Practice for Petrographic Examination of Hardened Concrete.” Sections were polished and examined using a stereomicroscope at magnifications up to

350×. Small rectangular blocks were cut from concrete samples. One surface of each block was polished, dried, placed on petrographic slide with low−viscosity epoxy resin, and reduced to a thickness of approximately 25 micrometers (0.001 in.). These thin sections were examined using a polarized−light microscope at magnification up to 400×, to determine aggregate mineralogy, paste characteristics, and microstructure.

Two ¾-inch thick slabs were sawn from the concrete cores. The slabs were ground and polished to obtain a smooth and plane surface. They were examined using a stereomicroscope at magnifications up to 200×.

4. Findings

Eight thin section samples taken from the received cores were examined using the polarized light microscope. Two polished sections were examined using the stereomicroscope. The findings are summarized as follows:

Aggregate

The coarse aggregate in the concrete is crushed stone, and the rocks include metamorphic rocks (hornfels and mylonite) and sedimentary rocks (sandstone and quartzite). Coarse aggregate particles are mostly angular, and the maximum size is about 3/4 inch. A portion of the coarse aggregate particles is elongated or flaky. Preferential orientation of coarse aggregate particles is not observed in this concrete.

The fine aggregate fraction is mainly composed of quartz. The fine aggregate is from natural sand and the particles appear rounded to angular.

Cement Paste

The cement is reasonably hydrated and the paste contains some unhydrated cement particles as seen under the microscope (Figure R1-1).

Air Voids

Small, spherical air voids are observed in the concrete (Figure R1-2), hence the concrete was air entrained. Entrained air voids were well distributed in the concrete. Entrapped air (and occasionally, water) voids are also present in the concrete, often underneath coarse aggregate particles.

Gaps/Cracks in the Interfacial Region

It is observed that gaps and cracks formed in the paste-aggregate interfacial region of the concrete, as shown in Figure R1-3 and Figure R1-4. The coexistence of some air voids in the interfacial region further weakens the bond between the coarse aggregate and the cement paste (Figure R1-5). In general, the gaps/cracks are often present at the aggregate-paste interface on the underside of a coarse aggregate particle, especially that of an elongated or flaky particle. Thus, internal bleeding is believed to be the cause of the gaps and cracks in the interfacial region. This leads to the formation of very porous paste in the transition zone.

Isolated cracks in cement paste are sporadically observed in the concrete, as shown in Figure R1-6. Other cracks meander along some fine aggregate peripherals and through the paste (Figure R1-7 and Figure R1-8). Similar patterns are also found in the interfacial region of coarse aggregate and paste (Figure R1-9).

Secondary Deposit

Ettringite is observed in some air voids in the concrete. Very often, ettringite crystals filled up a portion of a void, as shown in Figure R1-10 and Figure R1-11. Occasionally, voids fully filled with ettringite are also found in the concrete (Figure R1-12).

5. Summary

The concrete was air entrained, and the entrained air voids were well distributed in the concrete. Gaps and cracks that have formed in the coarse aggregate/paste interfacial region are the result of bleeding—the rising of water in the concrete due to the settling of more dense constituents while the concrete is still plastic. This results in water being trapped underneath of coarse aggregate particles. The cement paste in the interfacial region is porous. The bond between the aggregate and the paste is weakened by the gaps/cracks and the porous paste in the transition zone. Cracks also exist in cement paste and in the fine aggregate/paste interfacial region. Despite the defects in microscopical scale, the concrete appeared solid and sound, and no deterioration was shown visually.

Ettringite crystals formed in air voids. Often, ettringite filled part of a void. But voids fully filled with ettringite are also found in the concrete. There was no evidence of deterioration associated with the existence of the ettringite in the concrete. It is very common to see ettringite as secondary deposit in concrete.

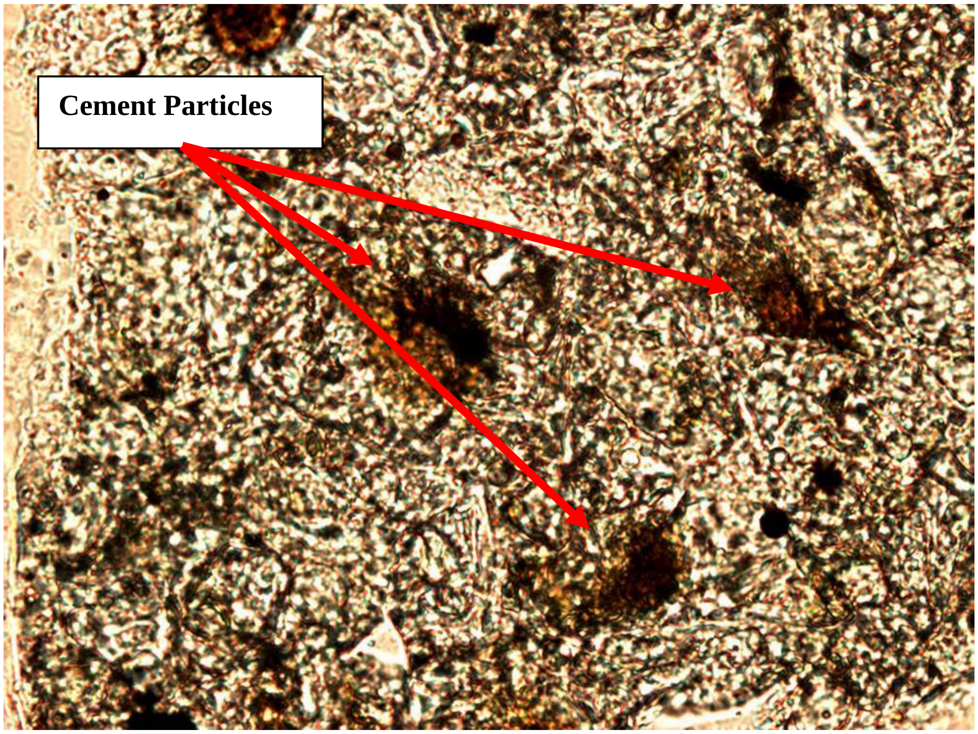


Figure R1-1: Unhydrated cement particles in paste. Width of field is 0.165 mm. Thin section image.

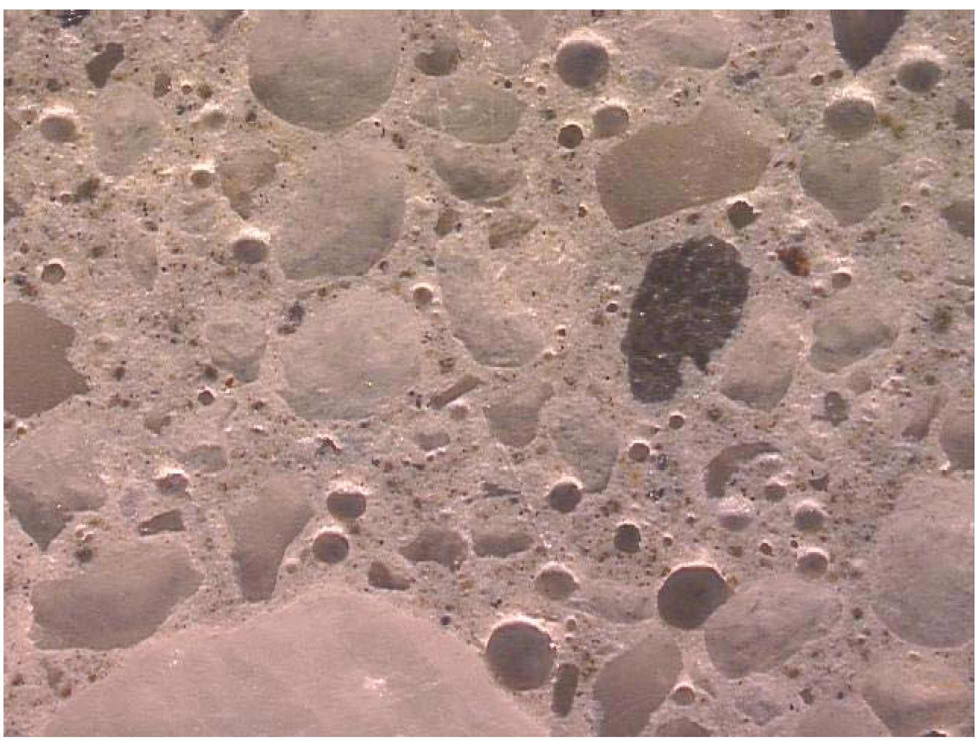


Figure R1-2: Air voids in the concrete. Width of field is 4.0 mm. Polished concrete surface image.

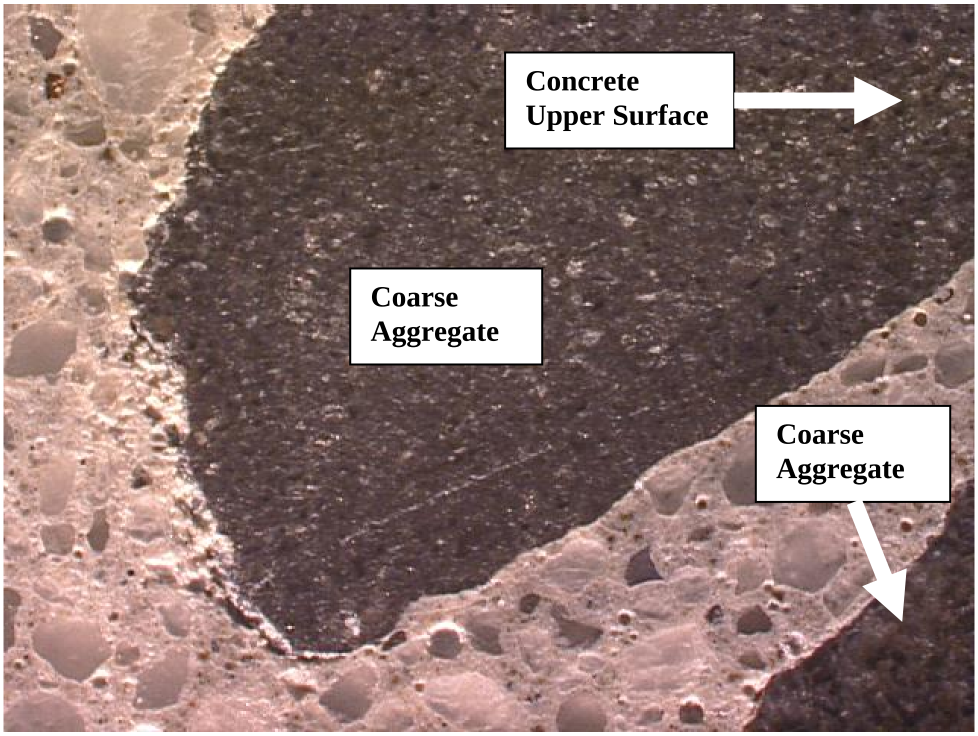


Figure R1-3: Gap and crack, as well as some air voids, line up around the coarse aggregate perimeter. Width of field is 4.0 mm. Polished surface image.

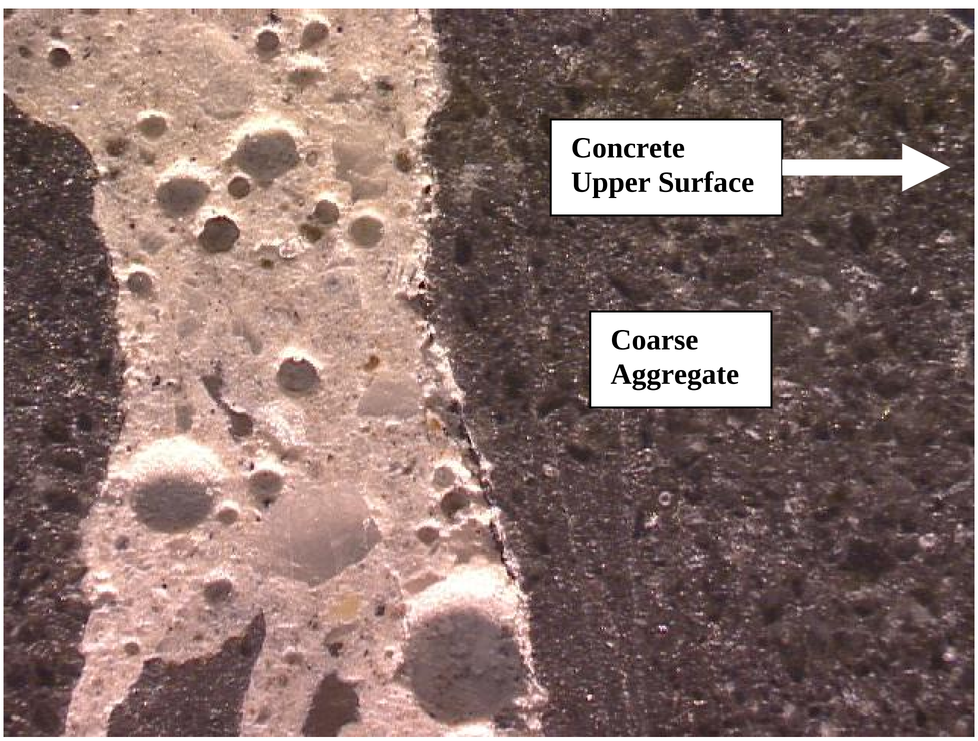


Figure R1-4: Another image of air voids and gap surrounding coarse aggregate. Width of field is 4.0 mm. Polished surface image.

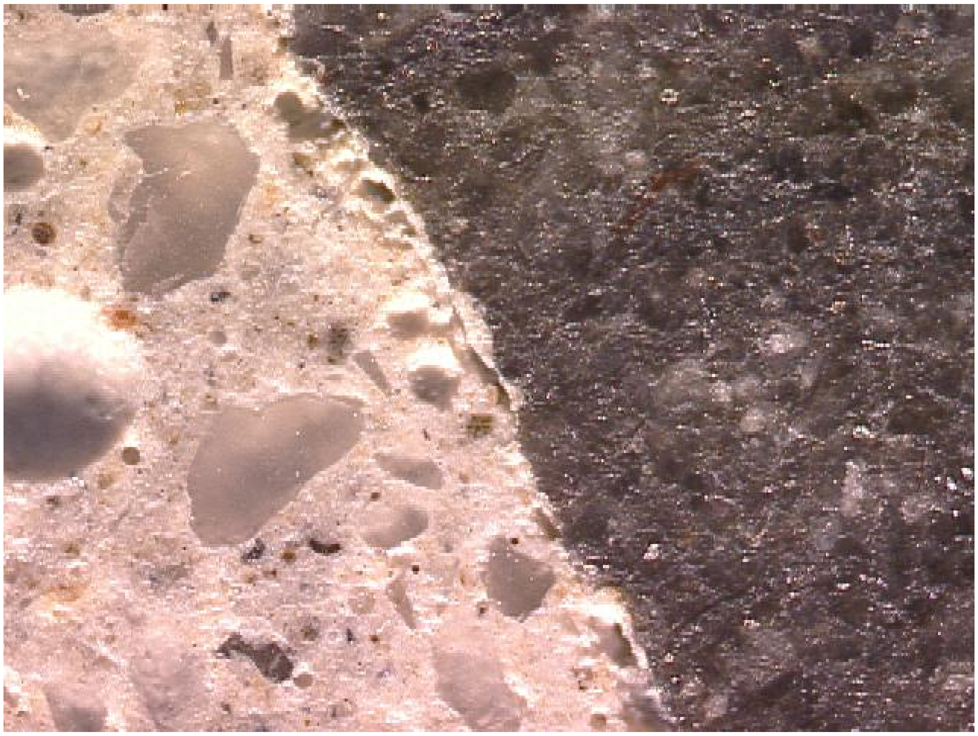


Figure R1-5: Gaps and voids in the paste/coarse aggregate interfacial region. Width of field is 2.0 mm. Polished surface image.

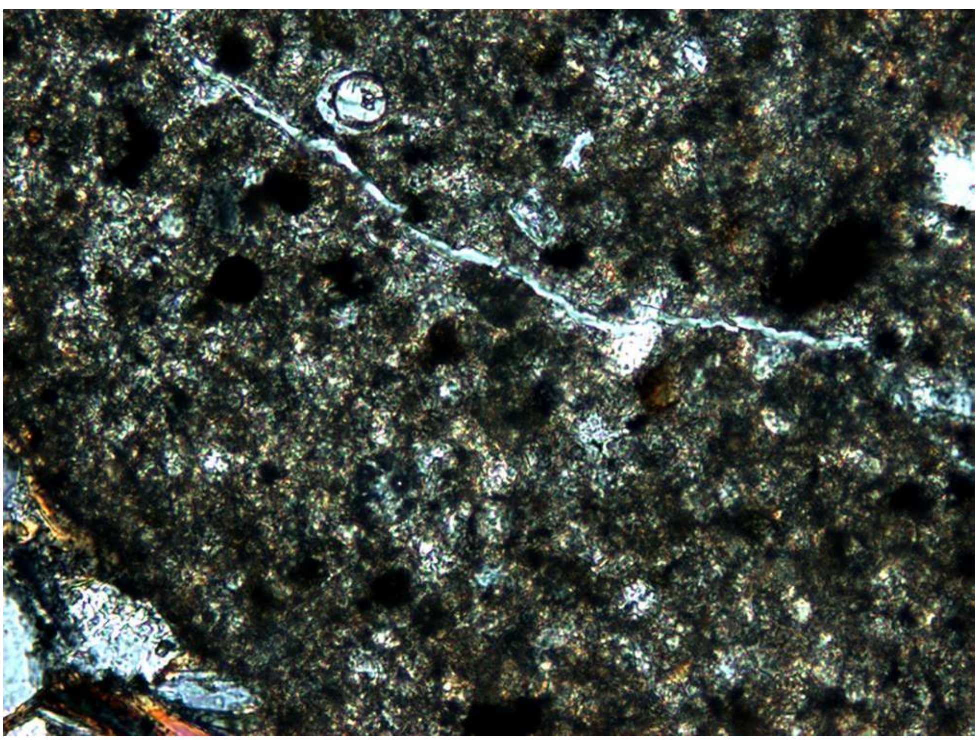


Figure R1-6: Crack in the paste. Width of field is 0.33 mm. Thin section image.

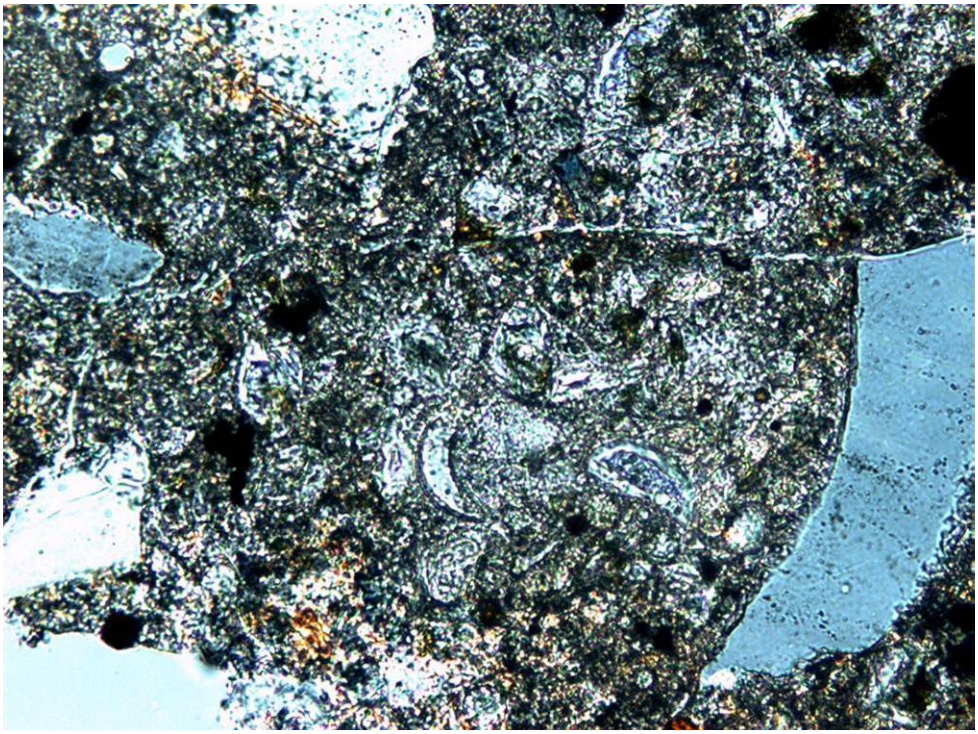


Figure R1-7: A crack connecting two fine aggregate particles. The paste/fine aggregate interface was also cracked. Width of field is 0.33 mm. Thin section image.

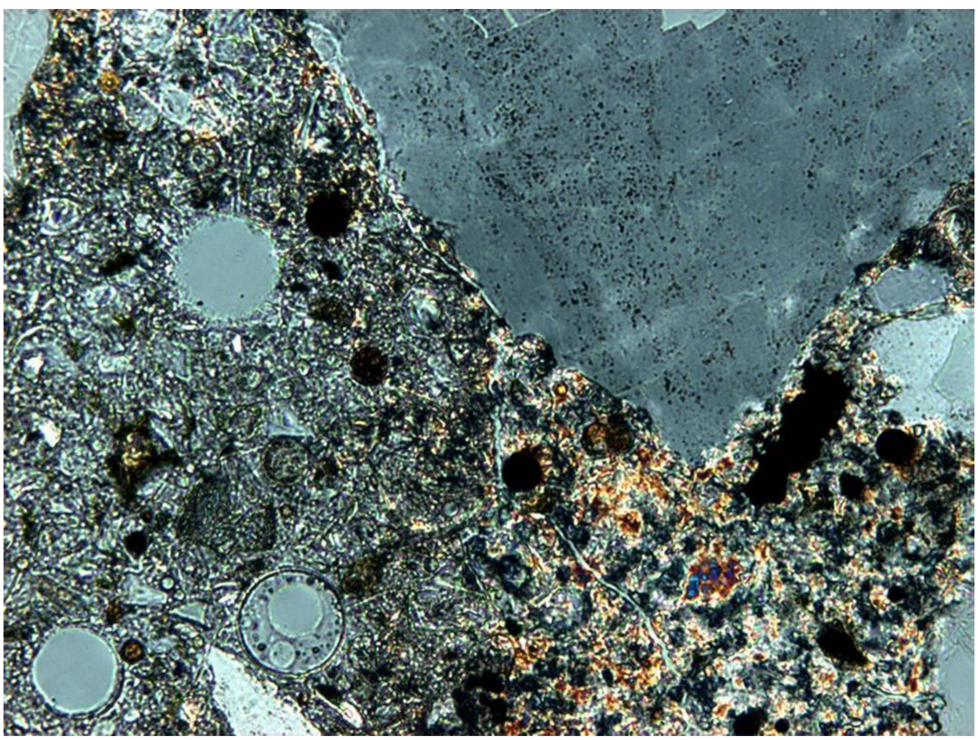


Figure R1-8: Crack in the paste/fine aggregate interfacial region and adjacent paste. Width of field is 0.33 mm. Thin section image.

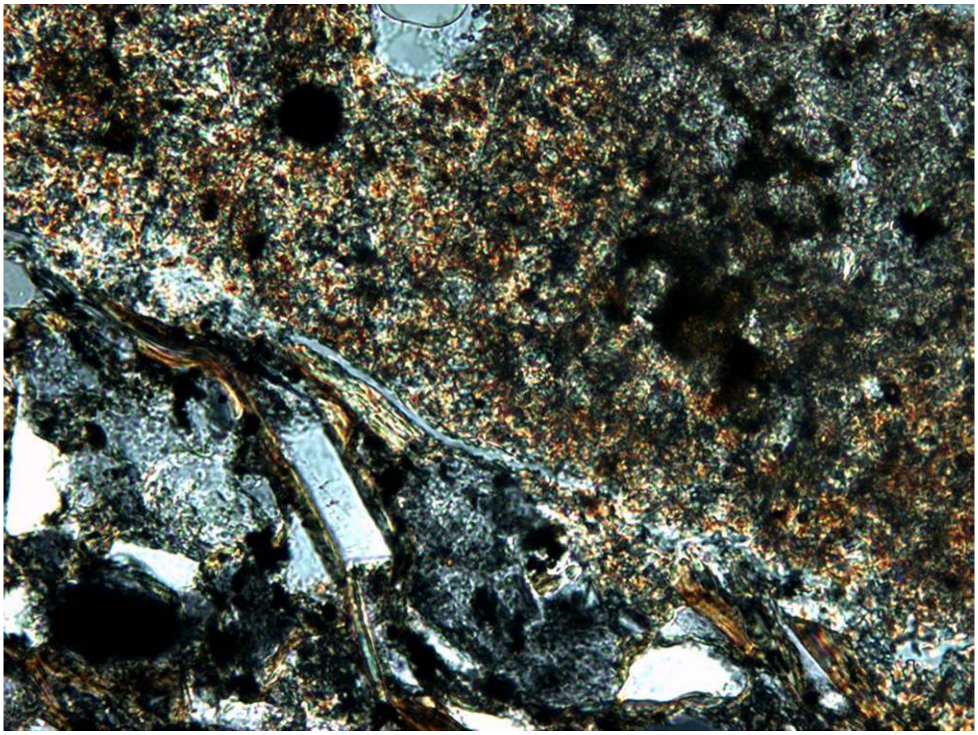


Figure R1-9: Crack in the paste/coarse aggregate interfacial region. Width of field is 0.33 mm. Thin section image.

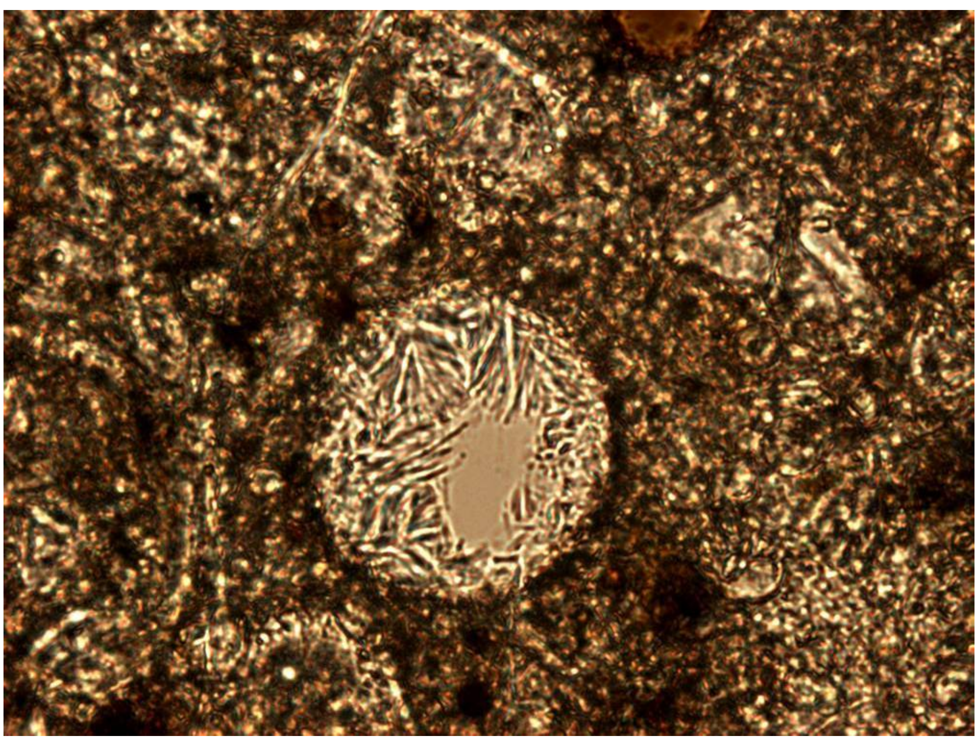


Figure R1-10: Ettringite in an air void. Width of field is 0.165 mm. Thin section image.

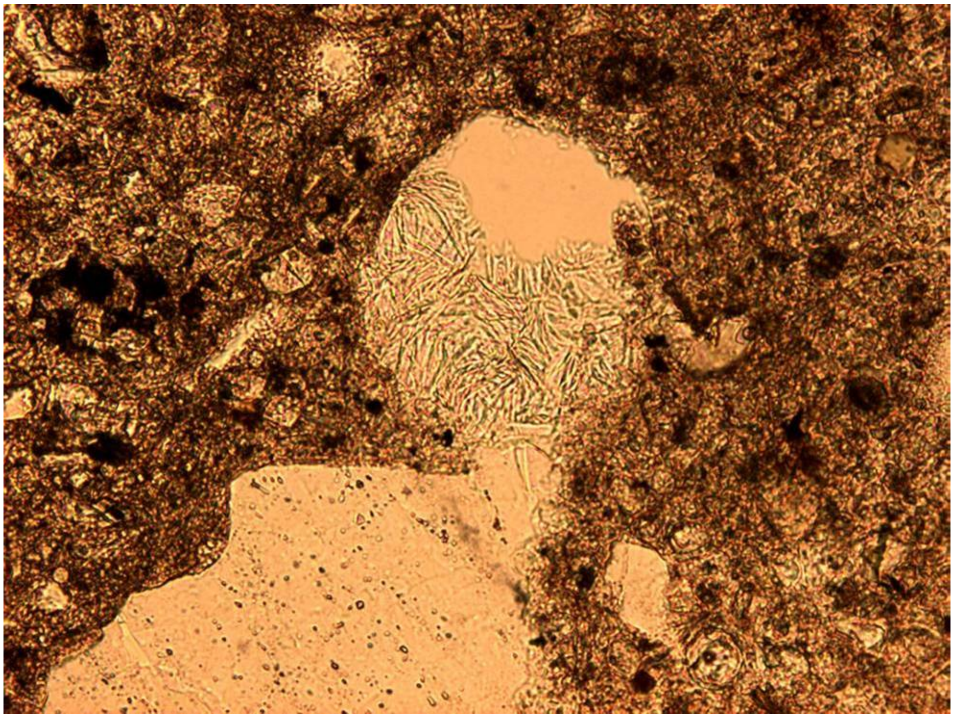


Figure R1-11: Another image of ettringite in an air void. Width of field is 0.33 mm. Thin section image.

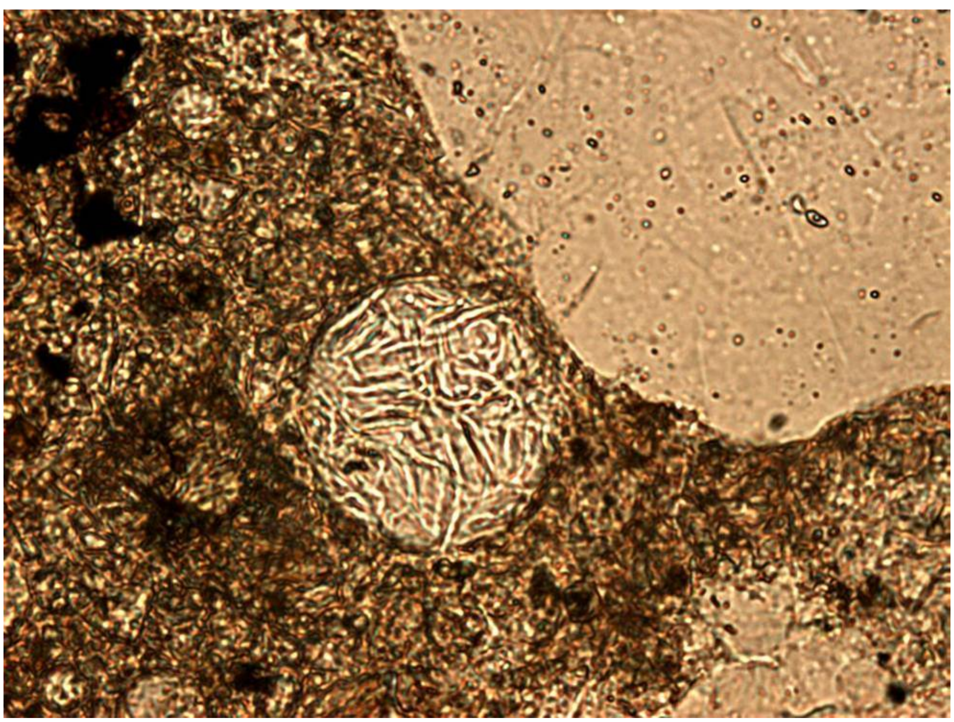


Figure R1-12: An air void fully filled with ettringite. Width of field is 0.165 mm. Thin section image.

APPENDIX R – Supplement 2

Virginia Avenue Bridge, Richlands, Virginia
Survey Checklist

Checklist

The following checklist is adapted from 201.1 R-2, and provided to facilitate a thorough survey. The definition of terms and associated photographs in 201.1 R-2, are utilized to standardize the reporting of the condition of the concrete in the structures.

1. Description of structure or pavement
    - 1.3 Name, location, type, and size: Richlands, VA, Route 0000, Virginia Avenue , Two 74 ft span continuous for live load, 148 ft.
    - 1.2 Owner, project engineer, contractor, when built: VDOT, Richlands, 1997
    - 1.3 Design
        - 1.3.1 Architect and/or engineer
        - 1.3.2 Intended use and history of use
        - 1.3.3 Special features
    - 1.4 Construction ____________
        - 1.4.1 Contractor-general
        - 1.4.2 Subcontractors concrete placement
        - 1.4.3 Concrete supplier
        - 1.4.4 Agency responsible for testing
        - 1.4.5 Other subcontractors
    - 1.5 Photographs
        - 1.5.1 General view ____________
        - 1.5.2 Detailed close up of condition of area ____________
    - 1.23 Sketch map-orientation showing sunny and shady and well and poorly drained regions Not Shaded

2. Present condition of structure Date of Evaluation 4/29-30/03
    - 2.1 Overall alignment of structure Good
        - 2.1.1 Settlement None
        - 2.1.2 Deflection None
        - 2.1.3 Expansion Normal
        - 2.1.4 Contraction Normal
    - 2.2 Portions showing distress (beams, columns, pavement, walls, etc., subjected to strains and pressures) Cracks on Deck & Sidewalks
    - 2.3 Surface condition of concrete
        - 2.3.1 General (good, satisfactory, poor, dusting, chalking, blisters) Good
        - 2.3.2 Cracks Longitudinal, Trans, Diagonal
            - 2.3.2.1 Location and frequency See Fig. 2
            - 2.3.2.34 Type and size (see Definitions) See Below

              Longitudinal Over Girders

              Width (from Crack comparator) 0.008-0.022 in.

Hairline	(Less than 1/32 in.)
Fine	(1/32 in. - 1/16 in.)
Medium	(1/16 – 1/8 in.)
Wide	(Greater than 1/8 in.)

              Transverse Random

		Width (from Crack comparator)	0.006-0.014 in.
		Hairline	(Less than 1/32 in.)
		Fine	(1/32 in. - 1/16 in.)
		Medium	(1/16 – 1/8 in.)
		Wide	(Greater than 1/8 in.)
		Craze	
		Width (from Crack comparator)	in.
		Hairline	(Less than 1/32 in.)
		Fine	(1/32 in. - 1/16 in.)
		Medium	(1/16 – 1/8 in.)
		Wide	(Greater than 1/8 in.)
		Map	
		Width (from Crack comparator)	in.
		Hairline	(Less than 1/32 in.)
		Fine	(1/32 in. - 1/16 in.)
		Medium	(1/16 – 1/8 in.)
		Wide	(Greater than 1/8 in.)
		D-Cracking	
		Width (from Crack comparator)	in.
		Hairline	(Less than 1/32 in.)
		Fine	(1/32 in. - 1/16 in.)
		Medium	(1/16 – 1/8 in.)
		Wide	(Greater than 1/8 in.)
		Diagonal	At South Abut.
		Width (from Crack comparator)	0.010 in.
		Hairline	(Less than 1/32 in.)
		Fine	(1/32 in. - 1/16 in.)
		Medium	(1/16 – 1/8 in.)
		Wide	(Greater than 1/8 in.)
	2.3.2.35	Leaching, stalactites	None
2.3.3	Scaling		None
	2.3.3.1	Area, depth	--
	2.3.3.18	Type (see Definitions)	--
		Light	(Less than 1/8 in.)
		Medium	(1/8 in. – 3/8 in.)
		Severe	(3/8 in. – 3/4 in.)
		Very Severe	(Greater than 3/4 in.)
2.3.4	Spalls and popouts		None
	2.3.4.1	Number, size, and depth	--
	2.3.4.18	Type (see Definitions)	--
		Spalls	
		Small	(Less than 3/4 in. depth)
		Large	(Greater than 3/4 in. depth)
		Popouts	
		Small	(Less than 3/8 in. diameter)
		Medium	(3/8 in. – 2 in. diameter)

			Large (Greater than 2 in. diameter)	
	2.3.5	Extent of corrosion or chemical attack, abrasion, impact, cavitation		Small gouges
	2.3.6	Stains, efflorescence		None
	2.3.7	Exposed reinforcement		None
	2.3.8	Curling and warping		None
	2.3.9	Previous patching or other repair		None
	2.3.10	Surface coatings		N/A
		2.3.10.1	Type and thickness	--
		2.3.10.2	Bond to concrete	--
		2.3.10.3	Condition	--
	2.3.11	Abrasion		None
	2.3.12	Penetrating sealers		N/A
		2.3.12.1	Type	--
		2.3.12.2	Effectiveness	--
		2.3.12.19	Discoloration	--
2.4	Interior condition of concrete (in situ and samples)			Sound
	2.4.1	Strength of cores		
	2.4.2	Density of cores		
	2.4.3	Moisture content		
	2.4.4	Evidence of alkali-aggregate or other reactions		None
	2.4.5	Bond to aggregate, reinforcing steel, joints		Good
	2.4.6	Pulse velocity		
	2.4.7	Volume change		
	2.4.8	Air content and distribution		
	2.4.9	Chloride-ion content		
	2.4.10	Cover over reinforcing steel		
	2.4.11	Half-cell potential to reinforcing steel.		
	2.4.12	Evidence of reinforcement corrosion		
	2.4.13	Evidence of corrosion of dissimilar metals		
	2.4.30	Delaminations		None
		2.4.30.1	Location	--
		2.4.30.2	Number, and size	--
	2.4.15	Depth of carbonation		
	2.4.16	Freezing and thawing distress (frost damage)		
	2.4.17	Extent of deterioration		
	2.4.34	Aggregate proportioning, and distribution		

3. Nature of loading and detrimental elements

3.1	Exposure		
	3.1.1	Environment (arid, subtropical, marine, freshwater, industrial, etc.)	Suburban
	3.1.2	Weather-(July and January mean temperatures,	72/32 ^{o}F
		mean annual rainfall	40.5
		and months in which 60 percent of it occurs)	Mar-Sept.
	3.1.3	Freezing and thawing	Yes

	3.1.4	Wetting and drying	Yes
	3.1.15	Drying under dry atmosphere	No
	3.1.6	Chemical attack-sulfates, acids, chloride	N/A
	3.1.7	Abrasion, erosion, cavitation, impact	Minor
	3.1.8	Electric currents	N/A
	3.1.9	Deicing chemicals which contain chloride ions	Yes
	3.1.10	Heat from adjacent sources	None
3.2		Drainage	Along Gutter
	3.2.1	Flashing	None
	3.2.2	Weepholes	In abutments only
	3.2.3	Contour	Toward South
	3.2.4	Elevation of drains	No Drains
3.3		Loading	
	3.3.1	Dead	No distress
	3.3.2	Live	No distress
	3.3.3	Impact	No distress
	3.3.4	Vibration	None
	3.3.5	Traffic index	Normal
	3.3.6	Other	--
3.4		Soils (foundation conditions)	Approach settlement along shoulder
	3.4.1	Compressibility	
	3.4.2	Expansive soil	
	3.4.3	Settlement	
	3.4.4	Resistivity	
	3.4.5	Evidence of pumping	
	3.4.6	Water table (level and fluctuations)	
4.		Original condition of structure	Good
4.1		Condition of formed and finished surfaces	Good
	4.1.1	Smoothness	Good
	4.1.2	Air pockets ("bugholes")	None
	4.1.3	Sand streaks	None
	4.1.4	Honeycomb	None
	4.1.5	Soft areas (retarded hydration)	None
	4.1.6	Cold joints	Closure pour over pier
	4.1.39	Staining	None
	4.1.40	Sand pockets	None
4.2		Defects	Cracking
	4.2.1	Cracking	
	4.2.1.1	Plastic shrinkage	
	4.2.1.2	Thermal shrinkage	
	4.2.1.3	Drying shrinkage	
	4.2.18	Curling	None
5.		Materials of Construction	Good
6.		Construction Practices	Good

APPENDIX S

Eastbound SR18 over SR516, King County, Washington

HIGH PERFORMANCE CONCRETE BRIDGE DECK INVESTIGATION

Eastbound SR18 over SR 516, King County, Washington

I. BACKGROUND

The eastbound SR18 / SR516 Over-crossing Bridge in King County, just north of Seattle, Washington is the first High Performance Concrete (HPC) Bridge built in Washington. It is a two-lane, three-span structure. HPC was used in all girders and decks. The bridge is 297-ft long. Clear width of the bridge is 38 ft, and it consists of two 12-ft lanes, one 4-ft bike lane on the left side and one 10-ft shoulder on the right. The eastbound SR18 / SR 516 Over-crossing Bridge opened to traffic in March 1998.

The eastbound SR18 / SR 516 Over-crossing Bridge was designed for earthquake zone "C" (acceleration coefficient = 0.25g). Pretensioned concrete girders (WSDOT W74G) with a compressive strength of 10,000 psi at 56 days were used in this HPC bridge construction project. The use of HPC improves construction economy by enabling longer span, increased girder spacing, and shallower girder. WSDOT Class 4000D concrete mix design with a compressive strength of 4,000 psi at 28 days was used in the construction of cast-in-place concrete deck. The concrete mixture contained fly ash and required continuous wet curing for 14 days.

The eastbound SR18 / SR 516 Over-crossing Bridge have three spans (80-, 137-, and 80-ft long, respectively). The skew of the bridge is 40^o at both ends. Each span consists of five WSDOT W74G girders made of precast, prestressed HPC. The girders are evenly spaced at 8-ft centers and support the cast-in-place concrete deck. The bridge decks are 7.5-in. thick. Longitudinal deck reinforcing steel has 2½-in. cover on the top and 1-in cover on the bottom.

The eastbound SR18 / SR 516 Over-crossing Bridge was part of a demonstration project for HPC in bridge structures which was co-sponsored by the Federal Highway Administration (FHWA) and the Washington State Department of Transportation (WSDOT). As part of that program, the University of Washington and the Washington State Department of Transportation (WSDOT) undertook this project to investigate the long-term behavior of HPC pretensioned concrete girders. It is evident that the structures are intended to be compared for relative durability and performance based on the extensive use of HPC. Experience gained through the design and fabrication of the eastbound SR18 / SR 516 Over-crossing Bridge in King County, Washington contributed significantly to the high performance concrete bridges constructed in Washington State thereafter.

II. SCOPE OF SERVICES

Professional Service Industries (PSI) is under contract with the Federal Highway Administration (FHWA) to conduct bridge deck inspections for HPC bridges. Our scope of services on this bridge included a series of tasks and sub-tasks, which are described as follows:

- Collect all available information relevant to the construction of each bridge, including;
    - Deck Concrete Properties
    - Specified Deck Concrete Construction Procedures
    - Approved Concrete Mix Proportions
    - Measured Properties from QC
    - Other Measured Properties
    - Actual Method of Deck Placement
    - Average Daily Traffic (ADT) and Average Daily Truck Traffic (ADTT)
    - Exposure Condition of the Bridge
    - Any Performed Maintenance
    - Any Inspection Reports
- Visually inspect the bridge, and obtain the following:
    - General condition of the deck top surface
    - Determination of the maximum crack width
    - General condition of the deck underside
    - General condition of the girders
    - Photograph areas of significant deterioration
    - Prepare drawings locating defects
    - Extract 7 concrete core samples

III. COMPILATION OF BRIDGE CONSTRUCTION INFORMATION

Information sources used for this report include bridge drawings, research reports from University of Washington, and technical information contained in FHWA's "Compilation and Evaluation of Results from High Performance Concrete Bridge Projects" version 3.

Deck Concrete Properties

WSDOT Class 4000D concrete mixture containing fly ash was used in the eastbound SR18 / SR 516 Over-crossing Bridge. The bridge deck concrete had specified compressive strength of 4000 psi (28 MPa) at 28 days. While the concrete has a relatively low compressive strength, it is expected to be more durable than conventional concrete due to the use of fly ash and the requirement of 14-day water curing. Originally, a rapid chloride permeability of 1000 coulombs or less at 56 days was specified for cast-in-place deck. However, the requirement for chloride permeability was changed to a monitoring measurement for the deck rather than an acceptance criterion. In addition, WSDOT performed abrasion resistance tests on the deck concrete. Table 1 lists the specified properties of WSDOT Class 4000D HPC used in cast-in-place concrete bridge deck.

TABLE 1: Specified Concrete Properties

Property	Deck Class AA (HPC)
Minimum Cementitious Materials Content:	735 lb/yd^3
Max. Water/Cementitious Materials Ratio:	0.39
Min. Percentage of Fly Ash:	75 lb/yd^3
Max. Percentage of Fly Ash:	75 lb/yd^3
Maximum Aggregate Size:	¾-in
Air Content:	6%
Compressive Strength - Design:	4000 psi @ 28 days
Water Reducer:	Type A water reducer required

NOTES: Contractor provided the mixture design.

Specified Deck Concrete Construction Procedures

The cast-in-place deck was placed in September 1997. Contractor was required to comply with ACI 302—Guide for Concrete Floor and Slab Construction, and ACI 308—Standard Practice for Curing Concrete. The compressive strength, tensile strength, and elastic modulus variation of the deck concrete were measured.

A continuous wet curing, using two coats of curing compound along with quilted blankets or burlap for 14 days, is required for the HPC bridge project. The contractor was required to adhere strictly to the manufacture's written recommendations regarding the use of admixtures, including storage, transportation, and method of mixing.

Approved Concrete Mix Proportions

Deck

The approved proportions for WSDOT Class 4000D concrete used in the cast-in-place deck are shown in Table 2.

TABLE 2: Approved Mix Proportions for Cast-in-Place Deck

Mix Parameters	Deck (WSDOT Class 4000D)
Cement Brand:	Lone Star
Cement Type:	I
Cement Quantity:	660 lb/yd^3
Fly Ash Type:	C
Fly Ash Quality:	75 lb/yd^3
Fine Aggregate Quantity:	1100 lb/yd^3
Coarse Aggregate, Max. Size:	½-in.
Coarse Aggregate Type:	No. 5
Coarse Aggregate Quantity:	1700 lb/yd^3
Water:	290 lb/yd^3
Water Reducer Type:	A
Water Reducer Quantity:	6 fl oz/yd^3
Water/Cementitious Materials Ratio:	0.39

Measured properties of approved concrete mixture for the cast-in-place deck are summarized in Table 3.

TABLE 3: Measured Properties of Approved Concrete Mixtures for Cast-in-Place Deck

Measured Concrete Properties	Deck (WSDOT Class 4000D)
Slump:	6-9 in.
Air Content:	5.7%
Unit weight:	158 lb/yd^3
Compressive strength (AASHTO T 22):	5300 psi at 28 days
Rapid Chloride Permeability (AASHTO T 277):	2800 coulombs at 56 days
Abrasion Resistance (ASTM C 944-95):	4.5%

Measured Properties from QC Tests of Production Concrete

Cast-in-Place Deck

The measured properties from QC tests of WSDOT Class 4000D production concrete used in the cast-in-place deck are shown in Tables 4 through 6.

TABLE 4: Measured Compressive Strength from QC Tests of WSDOT Class 4000D Deck Concrete

Laboratory ID Number	Approx. Slump, in.	Air Content, %	Compressive Strength [(1)], psi
135556	3	6.8	5080, 5000
135557	3-1/2	5.9	4880, 4840
135558	3-1/4	4.6	5430, 5470
135559	3-1/2	5.5	5320, 5470
135560	2-3/4	5.6	5650, 6110
135561	3-1/4	4.6	6390, 6290
135562	3-1/2	6.1	5390, 5530
Average	3.3	5.6	5490

NOTES: [(1)] Tested specimens are 6×12-in. cylinders, stored in an insulated box for 24 hours, then transported to the laboratory, and tested at 28 days.

TABLE 5: Measured Abrasion Resistance from QC Tests of WSDOT Class 4000D Deck Concrete

Cylinder ID	Surface	Cumulative Weight Loss, grams		
		Cycle 1	Cycle 2	Cycle 3
EF-D 118	Top	1.5	1.9	3.7
	Mid-depth	0.9	1.0	1.1
EF-D 303	Top	1.0	2.5	3.1
	Mid-depth	0.4	0.5	1.7
EF-D 309	Top	0.9	2.7	3.2
	Mid-depth	0.3	1.4	1.5
Average	Top	1.13	2.37	3.33
	Mid-depth	0.53	0.97	1.43

NOTES: Abrasion tests (ASTM C944) consisted of 3 cycles of 2 minutes duration, each with an applied load of 22.1 lb. Test specimens were made from 6x12-in. cylinders cut in half at mid-length to produce an upper and lower test specimen. The test was performed on the original cylinder top surface of the upper test specimen and on the upper cut surface of the lower test specimen.

In addition, the measured rapid chloride permeability using AASHTO T 277 method was reported to be 2338, 2164, 3434 coulombs at concrete age between 3-1/2 and 6-1/2 months.

Actual Method of Deck Placement

Construction of the deck occurred in the summer of 1997. The bridge was completed in January 1998. HPC specifications developed by WSDOT had clearly defined the materials, construction practice, and the quality control program that should be used for the eastbound SR18 / SR 516 Over-crossing Bridge construction. It is believed that the HPC mix proportion with a low water-cementitious material ratio would be greatly affected by the slightest addition of water. Water-to-cement ratio conformance therefore has been required.

The concrete was internally vibrated to provide proper consolidation and avoid internal segregation. Fogging curing of the concrete deck started when the concrete was in the plastic state. This procedure avoided the surface moisture evaporation and plastic shrinkage cracks. This construction practice is particularly important for HPC.

Concrete was distributed by a mechanical spreader. A final troweled finish was applied followed by the tining for enhanced skidding resistance. The deck was cured using two coats of curing compound along with blankets or burlap for 14 days. Wet blankets were kept moist.

Average Daily Traffic (ADT) and Average Daily Truck Traffic (ADTT)

Average daily traffic for both eastbound and westbound lanes was calculated based on a count of all vehicles crossing the bridge during a 10 minutes period beginning at 1215 hrs on May 12, 2004. These vehicle counts gave an ADT of 11,088 and an ADTT of 1,584. The estimation of traffic flow was made while the PSI inspection crew was on-site.

Exposure Condition of the Bridge

The area surrounding the eastbound SR18 / SR516 Over-crossing Bridge is developed with mixed residential and commercial land use. Next to the eastbound HPC Over-crossing Bridge is the westbound bridge constructed using normal concrete. The bridge is located in a seismic region.

The National Weather Service reports that the mean maximum temperature varies between 75°F in July and 45°F in January. The mean minimum temperature varies between 55°F in July and 35°F in January. The normal precipitation varies between 5.9-in. per month in November to 0.8-in. per month in July. A few days per year the temperature drops below 32°F. Based on this information, the bridge has moderate annual exposure to freeze/thaw cycles.

Performed Maintenance

No documents were found which would indicate any maintenance had been performed since the bridge was constructed in 1997.

Inspection Reports

Bridge instrumentation and bridge monitoring have been performed by University of Seattle in cooperation with the WSDOT as part of the project. A number of 3-in. cores were drilled for previous bridge inspection, as was shown in Figure 2. The researchers have developed an instrumentation program to monitor the structural performance of the bridge and its components. Details were described in “High Performance Concrete in Washington State SR 18/SR 516 Overcrossing: Interim Report on Girder Monitoring” and “High Performance Concrete in Washington State SR 18/SR 516 Overcrossing: Interim Report on Materials Tests”.

IV. BRIDGE DECK INSPECTION

PSI personnel performed a visual inspection of the bridge decks during the week of May 10, 2004. The results of the inspection are summarized as follows.

General Condition of the Deck Top Surface

Figure 1 shows the general layout of the decks for the eastbound SR18 / SR 516 Over-crossing Bridge. Results of visual inspection of the decks are shown in Figure 2. Surface defects observed and documented during visual inspection primarily included transverse cracks and diagonal cracks (see photos 7 and 8). Other defects observed and documented included small spalls at joints and cracks, abrasion, and broken tinned edges (see photos 8 and 9). A distinct patch occurred between the adjacent span ends (see photo 10). It was observed that the concrete barrier wall along the bridge deck showed pattern of map cracking and efflorescence (see photo 11). It is believed that these cracks are due to plastic cracking. However, apparent signs of other serious damages such as freeze-thaw, D-cracking, alkali-silica reaction, and alkali-aggregate reaction were not observed. Efflorescence can be seen on the concrete barrier wall along the bridge. In addition, drilled cores (3-in. diameter) for previous investigation by others were also observed (see Figure 2).

A total of 137 cracks were recorded during visual survey of the bridge decks (see Figure 2). Of the 137 cracks, 89 cracks were transverse crack, 46 cracks were diagonal crack, and 2 cracks were longitudinal cracks. The sum of crack lengths was 971 ft over a bridge deck area of 11,286 ft^2. Crack density (total crack length / deck area) for the eastbound bridges was calculated to be 0.0860 ft/ft^2.

Majority of the cracks recorded were classified as hairline cracks, with widths less than 0.031 in. However, a few cracks had crack width as wide as 1 mm (0.039 in), as shown in photo 8. It can be noted that cracks were typically limited at span ends. Small surface spalls, which either occurred due to breaking of tined edges or the crack edges, were observed (see photo 8). Figure 2 also illustrates the locations of drilled cores from our investigation. The number, length, and density of cracks for each structure are shown in Table 6 through Table 8.

TABLE 6: Measured Transverse Cracks on the Surface of Eastbound Bridge Decks

Eastbound Traverse Cracks	Count	Length Range (feet)	Mean Length of Cracks (feet)	Median Length of Cracks (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span 1	6	4.0 to 7.0	5.83	6	35	3040	0.012
Span 2	44	2.0 to 38.0	15.06	11	662.5	5206	0.127
Span 3	39	2.5 to 14	3.53	5	60	3040	0.020

NOTES: Transverse cracks include cracks oriented parallel to skewed joints

TABLE 7: Measured Diagonal Cracks on the Surface of Eastbound Bridge Decks

Eastbound Diagonal Cracks	Count	Length Range (feet)	Mean Length of Cracks (feet)	Median Length of Cracks (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span 1	18	1.5 to 16.0	5.69	4	102.5	3040	0.0337
Span 2	11	3.0 to 19	4.00	3	44	5206	0.0085
Span 3	17	1.0 to 7.0	3.53	3	60	3040	0.0197

TABLE 8: Measured Longitudinal Cracks on Surface of Eastbound Bridge Decks

Eastbound Longitudinal Cracks	Count	Length Range (feet)	Mean Length of Cracks (feet)	Median Length of Cracks (feet)	Total Length of Cracks (feet)	Deck Area (ft^2)	Crack Density: Crack Length / Deck Area (ft/ft^2)
Span 1	NA	NA	NA	NA	NA	3040	NA
Span 2	NA	NA	NA	NA	NA	5206	NA
Span 3	2	2 to 5	3.5	3.5	7.0	3040	0.002

Maximum Crack Width

The maximum width of transverse cracks was measured to be 0.039 in. (1 mm). The majority of the cracks recorded had widths less than 0.031 in. According to ACI 201, most of the crack widths are classified as hairline cracks.

General Condition of the Deck Underside

The underside of the deck exhibits no signs of distress. At very limited locations, efflorescence was observed. Photos 4 through 6 show a general view of the underside of the deck.

General Condition of the Girders

With the aide of a truck mounted hydraulic platform, the girders were inspected from the underside of the eastbound SR18 / SR 516 Over-crossing Bridge. Cracks and efflorescence were observed on the underside of the bridge decks (see Photo 8).

Concrete Core Samples

Seven cores, approximately 5-inches long and 2-¾ inches in diameter, were retrieved from the decks. The core sample locations are shown on Figure 2. The locations were evenly distributed along each shoulder of the bridge. The cores were labeled WS-1 through WS-7 and were transferred to FHWA for further analysis.

TABLE 9: Core Dimensions

Sample	WS-1	WS-2	WS-3	WS-4	WS-5	WS-6	WS-7
Diameter (in.)	2¾	2¾	2¾	2¾	2¾	2¾	2¾
Length (in.)	3	5	5	4¾	3 ¼	4¾	5

Preliminary Conclusions

The construction of the eastbound SR18 / SR 516 Over-crossing Bridge is part of a demonstration project for HPC in bridge structures. It was completed in 1997. Researchers from University of Washington in cooperation with the WSDOT performed material testing, bridge instrumentation, and bridge monitoring throughout this project.

The visual inspection of the bridge decks as part of our study was performed about six years after the bridge opened to traffic. The eastbound bridges are exhibiting transverse crack, diagonal crack, and longitudinal crack. A total of 137 cracks were recorded on the bridge with a combined total crack length of 971 ft over a bridge deck area of 11,286 ft^2. Majority of these cracks was hairline cracks with width less than 0.016 in. No major distresses were observed in our bridge survey.

The total length of transverse crack and number of cracks for Span 2 are greater than those for other spans at the bridge. The crack density on eastbound Span 2 is the largest (i.e., 0.127 ft/ft^2).

Span 2 has the longest span length (i.e., 137 ft) as compared to other spans of the bridge (i.e., 80 ft). This relatively flexible structural system might have contributed to the development and widening of some cracks in Span 2.

It is also noted that relatively large numbers of short-length diagonal cracks were observed in Span 3 near span ends. The span ends have a 40° skew. Some of these cracks at span ends along the skew were exhibiting spalling due to breaking of the edges. A few fine width cracks (0.039 in.) were observed. At span ends, the cast-in-place decks were skewed but the girder line supporting these deck panels had a straight geometry. The layout of the cast-in-place decks may partly be attributed to the development of these diagonal cracks.

Along the traffic lanes on the concrete barrier wall, map cracking was observed, as shown in Photo 11. It is believed that those irregular cracks on the vertical surface of concrete barrier wall are due to plastic cracking.

In general, the work on the eastbound SR18 / SR 516 Over-crossing Bridge shows that HPC designs provide significantly higher strength that can lead to more efficient designs requiring fewer piers and, more important, improved durability.

Petrographic examination was performed on seven concrete cores that were retrieved from the bridge. The dimension of the cores was 2-3/4 in. in diameter and 3- to 5-in. long. The identification on the cores was as following: WA-1, WA-2, WA-3, WA-4, WA-5, WA-6 and WA-7. Visual inspection of the concrete cores revealed that two cores (WA-3 and WA-6) had cracks along the length of the core (measured from the exposed surface, the crack in core WA-3 was 3-in. long, and the crack in core WA-6 was 4-in. long). Two cores (WA-1 and WA-7) were split longitudinally along the existing cracks. Core WA-5 showed that the rebar level was about 3 in. below the surface.

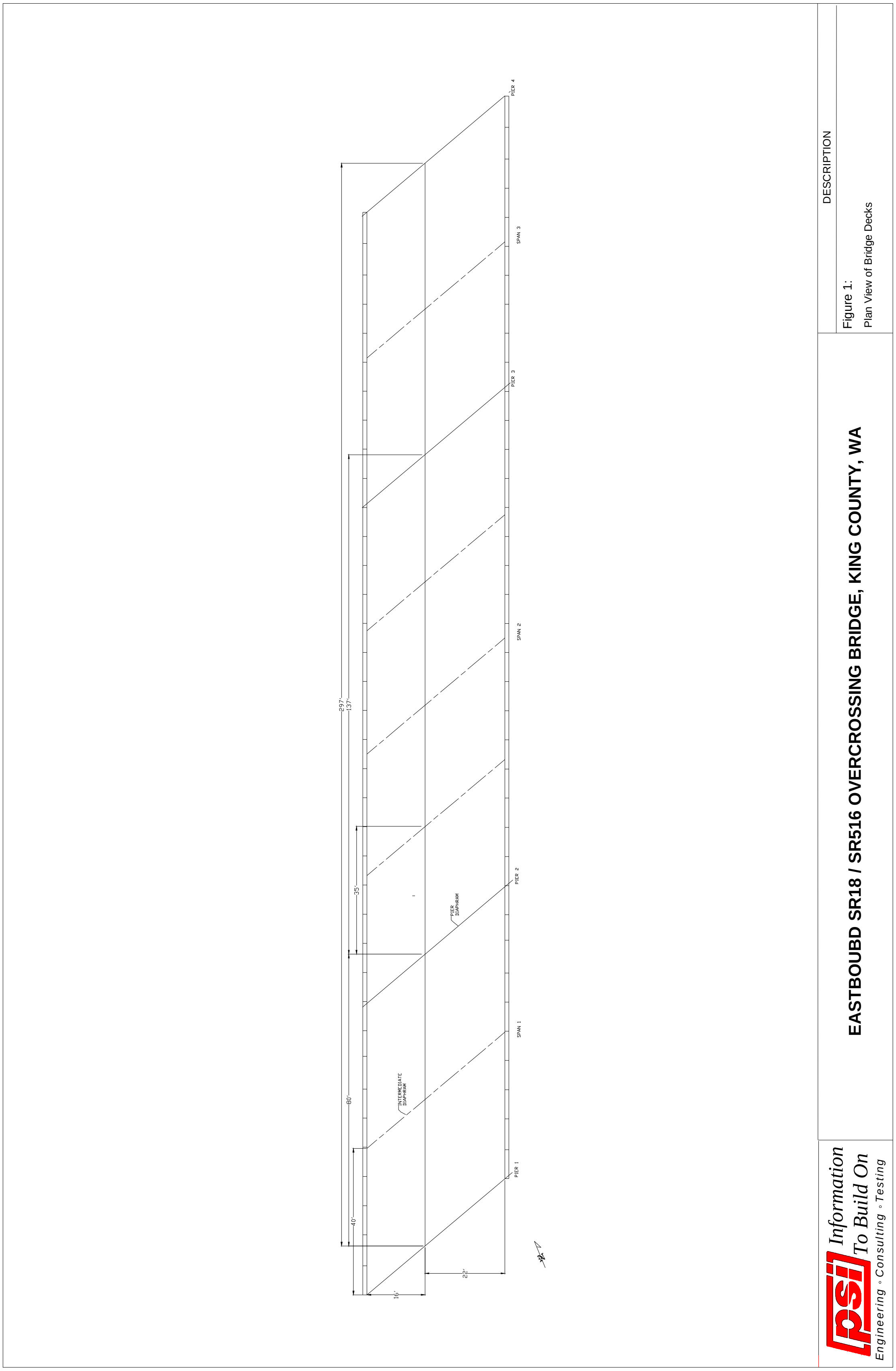
The coarse aggregate in the concrete was gravel, mainly basalt, andesite, diabase, sandstone, quartzite, and schist. Coarse aggregate particles were mostly rounded, and the maximum size, measured from the prepared concrete samples, was about 1 inch. Preferential orientation of coarse aggregate particles was not observed. The natural sand fine aggregate was mainly composed of quartz, basalt, schist, quartzite, and feldspar. The fine aggregate particles appeared rounded to angular.

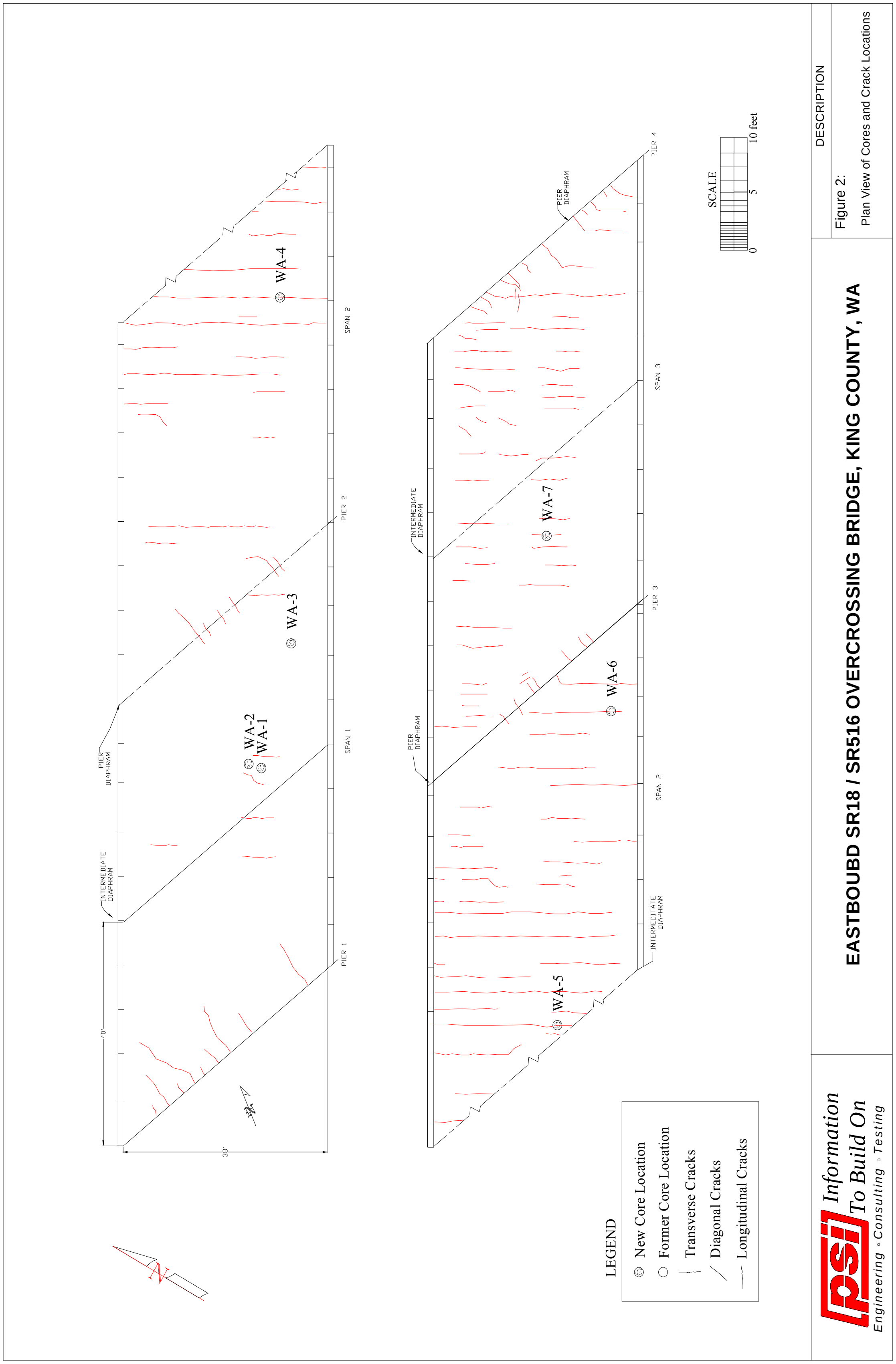
The cement was reasonably hydrated with respect to the age of the concrete. The paste contained some unhydrated cement particles. The paste/aggregate bond appeared to be good. However, cracks were sporadically seen at the interfacial region between the paste and aggregate.

The concrete was air entrained. Small, spherical air voids were well distributed in the concrete. A small amount of entrapped air voids is also present in the concrete. No secondary deposits were found in the concrete.

In addition to the major cracks visible in four of the seven cores, much smaller size cracks were found sporadically in the concrete. These small cracks existed mainly in the cement paste and the interfacial region between the aggregate and paste. It was speculated that shrinkage may be the cause of the cracking.

Respectfully Submitted,
Professional Service Industries, Inc.
Structural Investigation & Petrography Department


PIER 1
PIER 2
PIER 3
PIER 4
SPAN 1
SPAN 2
SPAN 3
INTERMEDIATE DIAPHRAGM
PIER DIAPHRAGM
297'
137'
35'
80'
40'
16'
22'
DESCRIPTION
Figure 1:
Plan View of Bridge Decks
EASTBOUBD SR18 / SR516 OVERCROSSING BRIDGE, KING COUNTY, WA
psi
Information To Build On
Engineering • Consulting • Testing


EASTBOUBD SR18 / SR516 OVERCROSSING BRIDGE, KING COUNTY, WA
DESCRIPTION
Figure 2:
Plan View of Cores and Crack Locations
LEGEND
New Core Location
Former Core Location
Transverse Cracks
Diagonal Cracks
Longitudinal Cracks
SCALE
0
5
10 feet
WA-1
WA-2
WA-3
WA-4
WA-5
WA-6
WA-7
PIER 1
PIER 2
PIER 3
PIER 4
SPAN 1
SPAN 2
SPAN 3
PIER DIAPHRAM
INTERMEDIATE DIAPHRAM
40'
38'
N
Information To Build On
Engineering • Consulting • Testing

Photographic Documentation

Photo 1: View looking northwest. The eastbound bridge is in foreground.

Photo 2: View looking southeast from the western end of the eastbound bridge.

Photo 3: North facing side of the eastbound bridge at the western end.

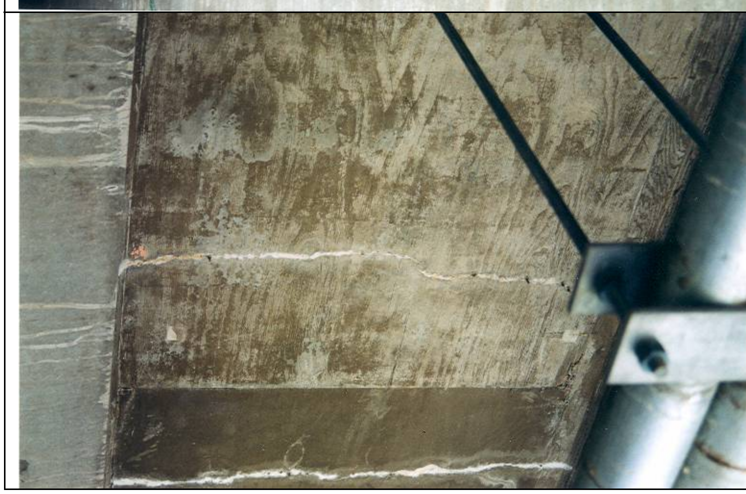
	Photo 4: View of the general condition at underside of bridge.
	Photo 5: View of the underside of bridge showing cracking and efflorescence.
	Photo 6: Close up view of the cracking at underside of bridge deck.

Photo 7: View of the typical transverse cracks.

Photo 8: View of the typical diagonal cracks. Crack width is 1mm.

Photo 9: View of the typical polished surface area.

	Photo 10: Patch in between the adjacent spans on the eastbound bridge.
	Photo 11: View of the map cracking on concrete barrier wall at eastbound bridge south facing side. Concrete barrier wall showing cracking and efflorescence.

APPENDIX S – Supplement 1

SR18 over SR516, King County, Washington
Petrographic Report

PETROGRAPHIC EXAMINATION OF CONCRETE CORES FROM A BRIDGE IN WASHINGTON STATE (WA)

A report submitted to the Structures Laboratory at TFHRC

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1. Introduction

Seven concrete cores were received from the Structures Laboratory of the Turner-Fairbank Highway Research Center for petrographic examination. Reportedly, these cores were collected from a concrete bridge in Washington State.

The dimension of the concrete cores was 2-3/4 in. in diameter, 3- to 5-in. long. The identification on the cores was as following: WA-1, WA-2, WA-3, WA-4, WA-5, WA-6 and WA-7 (Figure S1-1).

Thin sections, as well as polished slabs, were made from the concrete cores for microscopic examination. Petrographic examination was performed on these samples using optical microscopes.

Visual inspection of the concrete cores revealed that two cores (WA-3 and WA-6) have longitudinal cracks (measured from the exposed surface, the crack in core WA-3 is 3-in. long, and the crack in core WA-6 is 4-in. long). Two cores (WA-1 and WA-7) were split longitudinally along the existing cracks, as shown wrapped with string in Figure 1. Core WA-5 shows that the rebar level was about 3 in. below the surface. The findings from microscopic examination indicate that the concrete has entrained air voids, and the air content is estimated to be at a normal level; the hydration of the cement was reasonable, and the presence of unhydrated cement particles was observed in the cement paste; cracks exist in the paste as well as at the aggregate-paste interface; no secondary deposits were found.

2. Laboratory Procedures

Petrographic examination of the concrete samples was performed in accordance with ASTM C 856, “Standard Practice for Petrographic Examination of Hardened Concrete.” Sections were polished and examined using a stereomicroscope at magnifications up to 200×. Small rectangular blocks were cut from concrete samples. One surface of each block was polished, dried, placed on petrographic slide with low−viscosity epoxy resin,

and reduced to a thickness of approximately 25 micrometers (0.001 in.). These thin sections were examined using a polarized–light microscope at magnification up to 400×, to determine aggregate mineralogy, paste characteristics, and microstructure.

Two ¾-inch thick slabs were sawn from the concrete cores. The slabs were ground and polished to obtain a smooth and plane surface. They were examined using a stereomicroscope at magnifications up to 200×.

3. Findings

Eight thin section samples taken from the received cores were examined using the polarized light microscope. Two polished sections were examined using the stereomicroscope. The findings are summarized as follows:

Aggregate

The coarse aggregate in the concrete is gravel, containing mainly basalt, andesite, diabase, sandstone, quartzite, and schist. Coarse aggregate particles are mostly rounded, and the maximum size is about 1 inch. Preferential orientation of coarse aggregate particles is not observed in this concrete.

The fine aggregate fraction is mainly composed of quartz, basalt, schist, quartzite, and feldspar. The fine aggregate is from natural sand and the particles appear rounded to angular.

Cement Paste

The cement is reasonably hydrated and the paste contains some unhydrated cement particles as seen under the microscope (Figure S1-2).

Air Voids

Small, spherical air voids are observed in the concrete (Figure S1-3), hence the concrete was air entrained. Entrained air voids are well distributed in the concrete. The air content is estimated to be at a normal level. A small amount of entrapped air voids is also present in the concrete.

Cement-Aggregate Bonding

The paste/aggregate bond appears to be good, shown in Figure S1-4. However, cracks are sporadically seen at the interfacial region between the pastes and aggregate (Figure S1-5).

Secondary Deposit

No secondary deposits are found in the concrete.

Cracking

Examination of thin sections revealed cracking in the concrete samples (Figures S1-6 through S1-9). Some random cracks are sporadically seen in the cement paste. Cracks are also found at the interfacial region between the paste and aggregates.

Summary

The concrete is air entrained and the air content is estimated to be at a normal level. The entrained air voids are well distributed in the concrete. Cement was reasonably hydrated and unhydrated cement particles are present in the concrete. In addition to the major cracks visible in four of the seven cores, much smaller size cracks are found sporadically in the concrete as observed under the microscope. These small cracks exist mainly in the cement paste and the interfacial region between the aggregate and paste. It is speculated that shrinkage may be the cause of the cracking.

Figure S1-1: Seven concrete cores as received.

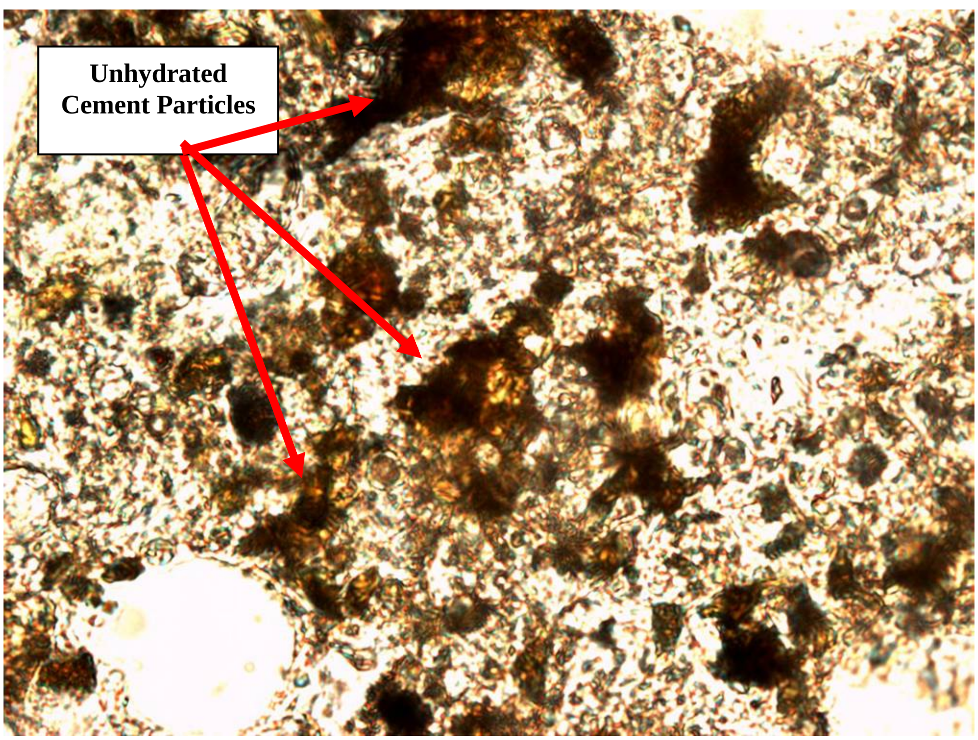

Figure S1-2: Unhydrated cement particles in paste. Width of field is 0.165 mm. Thin section image.

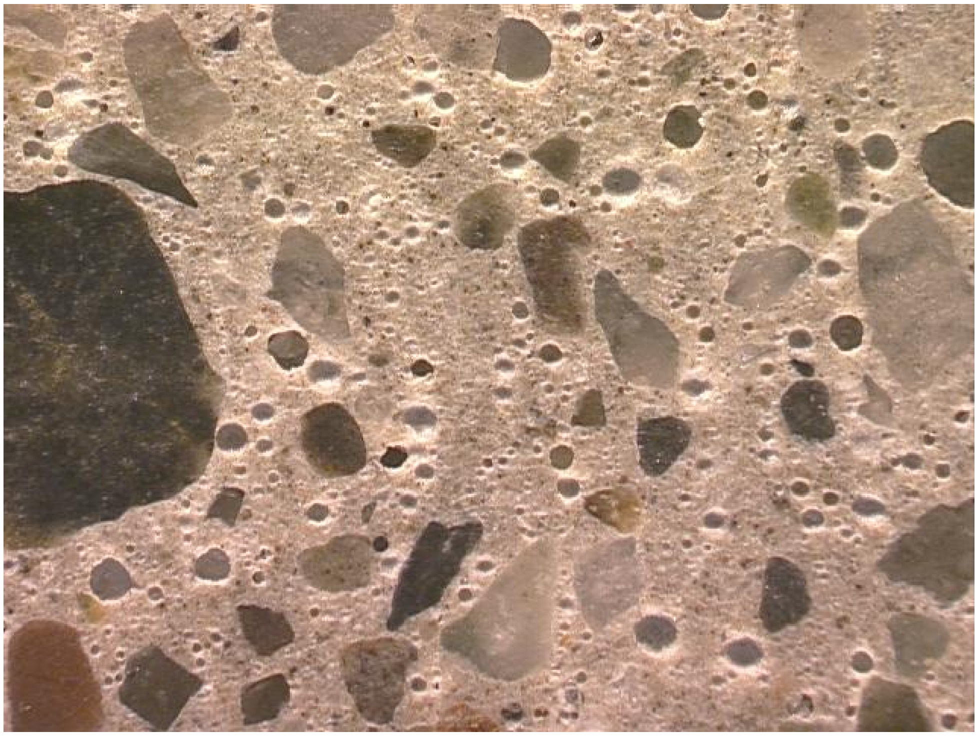

Figure S1-3: Entrained air voids in the concrete. Width of field is 4.0 mm. Polished surface image.

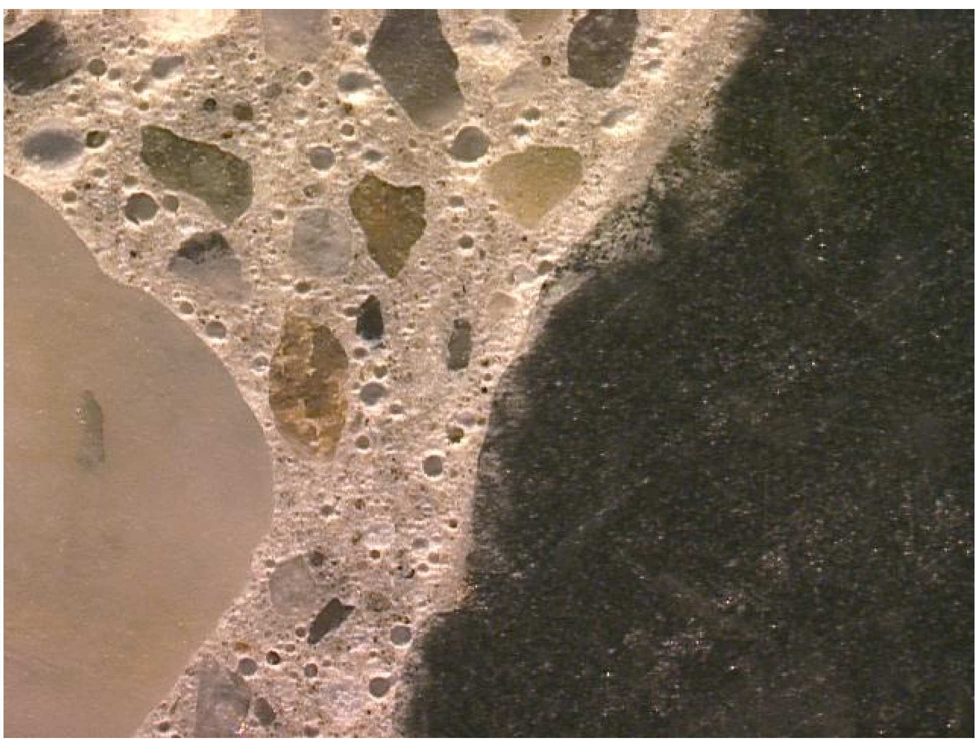

Figure S1-4: The bonding between aggregate and cement paste is good. Width of field is 4.0 mm. Polished surface image.

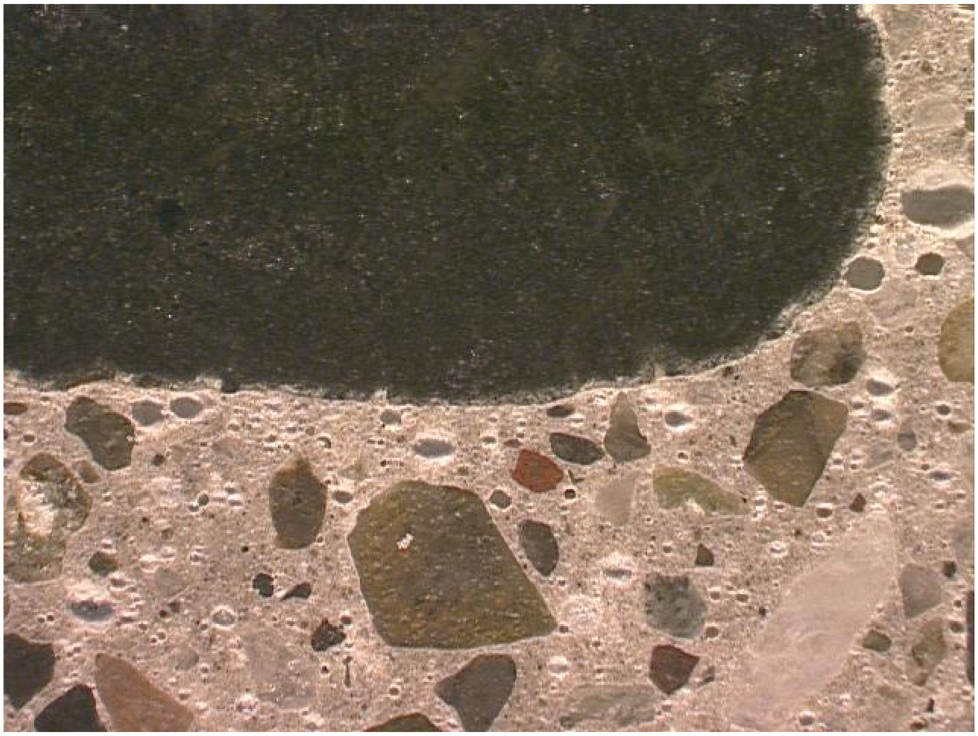

Figure S1-5: A gap has formed between the aggregate and paste. Width of field is 6.5 mm. Polished surface image.

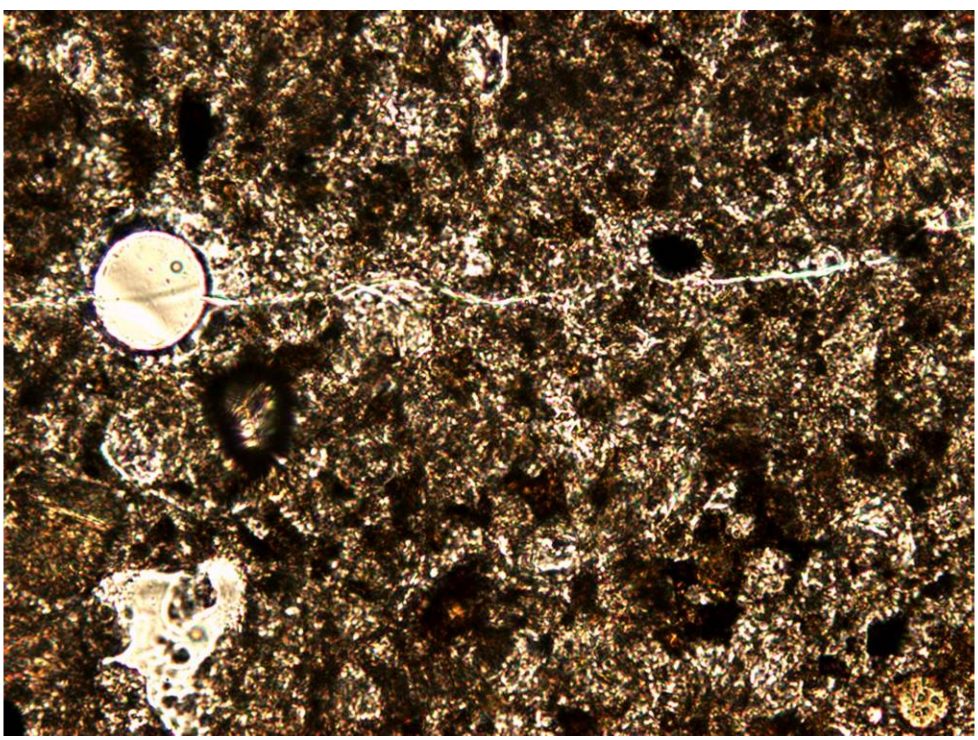

Figure S1-6: A crack in the paste. Width of field is 0.3 mm. Thin section image.

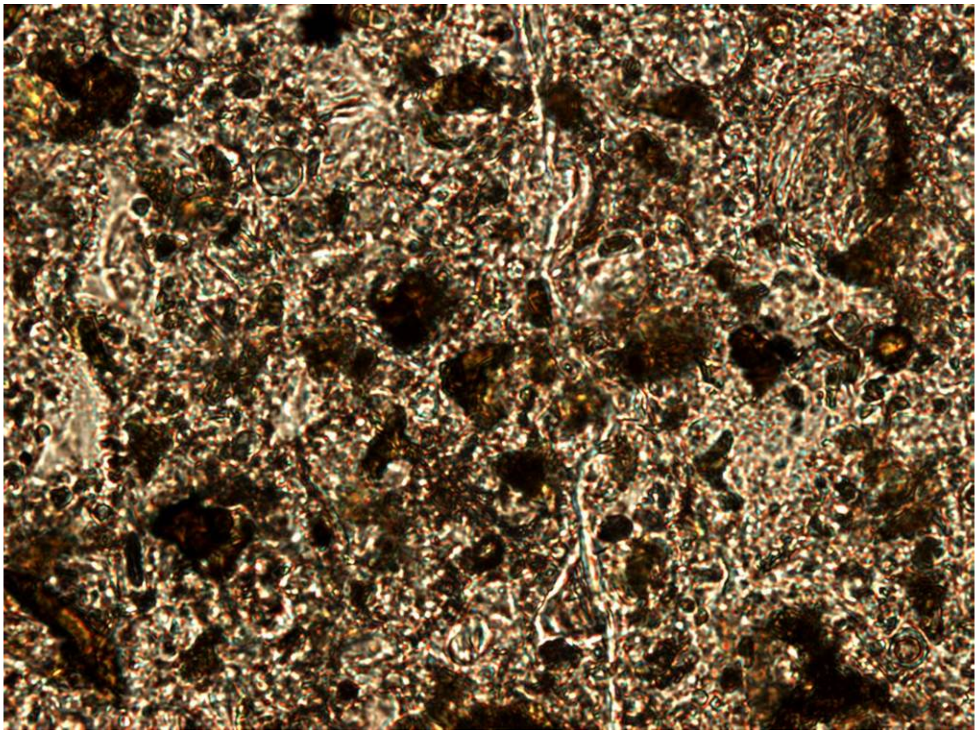

Figure S1-7: Another crack in the paste. Width of field is 0.165 mm. Thin section image.

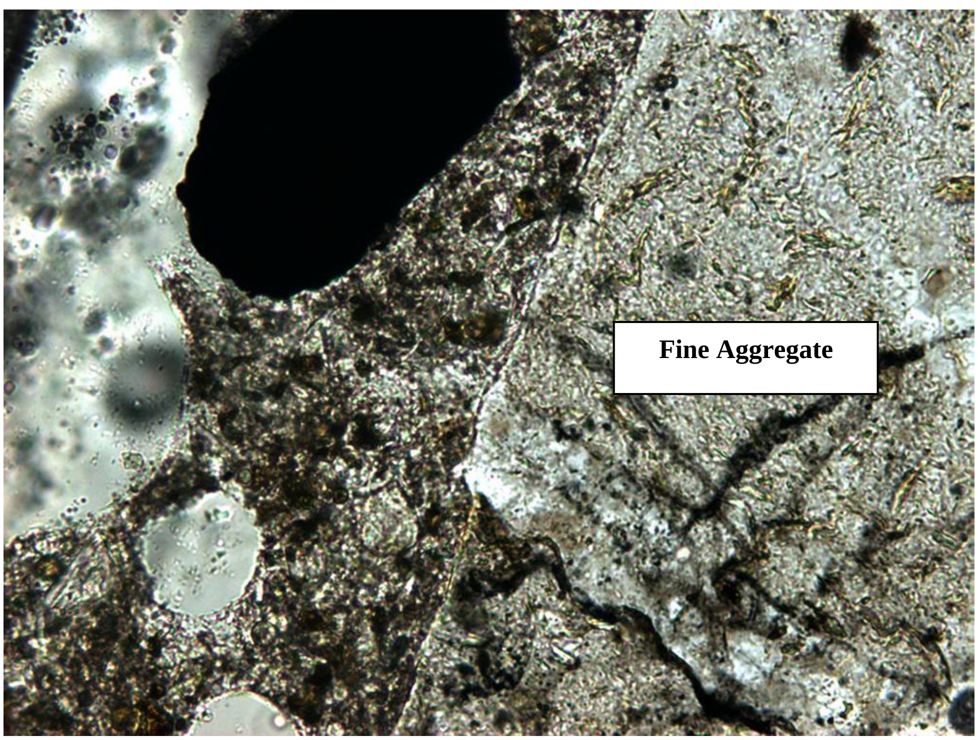

Figure S1-8: A crack at the interfacial region between the fine aggregate and paste. Width of field is 0.33 mm. Thin section image.

Figure S1-9: Cracking at the interfacial region between the paste and coarse aggregate. Width of field is 4.0 mm. Thin section image.

APPENDIX S – Supplement 2

SR18 over SR516, King County, Washington
Survey Checklist

Checklist

The following checklist is adapted from 201.1 R-92, and provided to facilitate a thorough survey. The definition of terms and associated photographs in 201.1 R-92, are utilized to standardize the reporting of the condition of the concrete in the structures.

1. Description of structure or pavement
   1.1 Name, location, type, and size
   The eastbound SR18 / SR516 Over-crossing Bridge in King County, just north of Seattle, Washington is a two-lane three-span structure. The bridge is 297-ft long. Clear width of the bridge is 38-ft, and it consists of two 12-ft lanes, one 4-ft bike lane on the left side and one 10-ft shoulder on the right. The eastbound SR18 / SR 516 Over-crossing Bridge opened to traffic in March 1998.
   1.2 Owner, project engineer, contractor, when built
   Owner-Washington State Department of Transportation. This bridge is part of a demonstration project for HPC in bridge structures which was co-sponsored by the Federal Highway Administration (FHWA) and the Washington State Department of Transportation (WSDOT). The bridge was constructed in 1997 and opened to traffic in March 1998.
   1.3 Design
      1.3.1 Architect and/or engineer: WSDOT.
      1.3.2 Intended use and history of use: To carry traffic over the eastbound SR18 over SR516. Opened to traffic in March 1998, No documents were found which would indicate any maintenance had been performed since bridge construction in 1997.
      1.3.3 Special features: Spans are skewed at 40°. The structures are intended to be compared for relative durability and performance based on the extensive use of HPC in the eastbound structure.
   1.4 Construction
      1.4.1 Contractor-general: Mowat Construction Company
      1.4.2 Subcontractors concrete placement: N/A
      1.4.3 Concrete supplier: Lone Star Northwest of Seattle, WA. The I-girders were fabricated by Central Pre-Mix Prestress Co. of Spokane, WA
      1.4.4 Agency responsible for testing: WSDOT and Univ. of Washington
      1.4.5 Other subcontractors: N/A
   1.5 Photographs
      1.5.1 General view — Photos 1 through 2
      1.5.2 Detailed close up of condition of area — Photos 4 through 11
   1.24 Sketch map-orientation showing sunny and shady and well and poorly drained regions: N/A

2. Present condition of structure — Date of Evaluation: The week of 5/10/2004
   2.1 Overall alignment of structure — No signs of misalignment
      2.1.1 Settlement

2.1.2 Deflection
2.1.3 Expansion
2.1.4 Contraction
2.2 Portions showing distress (beams, columns, pavement, walls, etc., subjected to strains and pressures) N/A
2.3 Surface condition of concrete
2.3.1 General (good, satisfactory, poor, dusting, chalking, blisters) Good
2.3.2 Cracks Transverse, Diagonal, and longitudinal
2.3.2.1 Location and frequency See Figure 2
2.3.2.36 Type and size (see Definitions) See Figure 2

Longitudinal Over each of the girder lines
Width (from Crack comparator) Less than 0.012 in.
- Hairline (Less than 1/32 in.)
- Fine (1/32 in. - 1/16 in.)
- Medium (1/16 – 1/8 in.)
- Wide (Greater than 1/8 in.)

Transverse Throughout the length
Width (from Crack comparator) < 0.014 in.
- Hairline (Less than 1/32 in.)
- Fine (1/32 in. - 1/16 in.)
- Medium (1/16 – 1/8 in.)
- Wide (Greater than 1/8 in.)

Craze N/A
Width (from Crack comparator) ____________
- Hairline (Less than 1/32 in.)
- Fine (1/32 in. - 1/16 in.)
- Medium (1/16 – 1/8 in.)
- Wide (Greater than 1/8 in.)

Map N/A
Width (from Crack comparator) ____________
- Hairline (Less than 1/32 in.)
- Fine (1/32 in. - 1/16 in.)
- Medium (1/16 – 1/8 in.)
- Wide (Greater than 1/8 in.)

D-Cracking N/A
Width (from Crack comparator) ____________
- Hairline (Less than 1/32 in.)
- Fine (1/32 in. - 1/16 in.)
- Medium (1/16 – 1/8 in.)
- Wide (Greater than 1/8 in.)

Diagonal At Skew Ends and acute corners
Width (from Crack comparator) < 0.040 in.
- Hairline (Less than 1/32 in.)

Fine (1/32 in. - 1/16 in.)
Medium (1/16 – 1/8 in.)
Wide (Greater than 1/8 in.)

2.3.2.37 Leaching, stalactites N/A

2.3.3 Scaling N/A

2.3.3.1 Area, depth ______

2.3.3.19 Type (see Definitions) ______

Light (Less than 1/8 in.)
Medium (1/8 in. – 3/8 in.)
Severe (3/8 in. – 3/4 in.)
Very Severe (Greater than 3/4 in.)

2.3.4 Spalls and popouts Minor, insignificant at the span ends near the broken tinned edges, along the skew

2.3.4.1 Number, size, and depth ______

2.3.4.19 Type (see Definitions) ______

Spalls
Small (Less than 3/4 in. depth)
Large (Greater than 3/4 in. depth)

Popouts
Small (Less than 3/8 in. diameter)
Medium (3/8 in. – 2 in. diameter)
Large (Greater than 2 in. diameter)

2.3.5 Extent of corrosion or chemical attack, abrasion, impact, cavitation N/A

2.3.6 Stains, efflorescence at the bottom side of the bridge deck

2.3.7 Exposed reinforcement N/A

2.3.8 Curling and warping N/A

2.3.9 Previous patching or other repair N/A

2.3.10 Surface coatings N/A

2.3.10.1 Type and thickness N/A

2.3.10.2 Bond to concrete N/A

2.3.10.3 Condition N/A

2.3.11 Abrasion N/A

2.3.12 Penetrating sealers N/A

2.3.12.1 Type N/A

2.3.12.2 Effectiveness N/A

2.3.12.20 Discoloration N/A

2.4 Interior condition of concrete (in situ and samples) N/A

2.4.1 Strength of cores

2.4.2 Density of cores

2.4.3 Moisture content

2.4.4 Evidence of alkali-aggregate or other reactions N/A

2.4.5 Bond to aggregate, reinforcing steel, joints N/A

2.4.6 Pulse velocity

2.4.7 Volume change

2.4.8 Air content and distribution

2.4.9 Chloride-ion content
2.4.10 Cover over reinforcing steel
2.4.11 Half-cell potential to reinforcing steel.
2.4.12 Evidence of reinforcement corrosion
2.4.13 Evidence of corrosion of dissimilar metals
2.4.31 Delaminations ____ N/A
2.4.31.1 Location ____ N/A
2.4.31.2 Number, and size ____ N/A
2.4.15 Depth of carbonation
2.4.16 Freezing and thawing distress (frost damage)
2.4.17 Extent of deterioration
2.4.35 Aggregate proportioning, and distribution

3. Nature of loading and detrimental elements
3.1 Exposure
3.1.1 Environment (arid, subtropical, marine, freshwater, industrial, etc.)
____
3.1.2 Weather-(July and January mean temperatures, 75˚F/45˚F
mean annual rainfall and 37.9 in.
months in which 60 percent of it occurs) Oct.-April
3.1.3 Freezing and thawing Moderate annual exposure to F-T
3.1.4 Wetting and drying Moderate annual exposure
3.1.16 Drying under dry atmosphere ____ N/A
3.1.6 Chemical attack-sulfates, acids, chloride ____ N/A
3.1.7 Abrasion, erosion, cavitation, impact ____ N/A
3.1.8 Electric currents ____ N/A
3.1.9 Deicing chemicals which contain chloride ions____ N/A
3.1.10 Heat from adjacent sources ____ N/A
3.2 Drainage ____ N/A
3.2.1 Flashing ____
3.2.2 Weepholes ____
3.2.3 Contour ____
3.2.4 Elevation of drains ____
3.3 Loading Research Test Data Available in Compilation CD Version 3
3.3.1 Dead ____
3.3.2 Live ____
3.3.3 Impact ____
3.3.4 Vibration ____
3.3.5 Traffic index ____
3.3.6 Other ____
3.4 Soils (foundation conditions) ____
3.4.1 Compressibility
3.4.2 Expansive soil
3.4.3 Settlement
3.4.4 Resistivity
3.4.5 Evidence of pumping
3.4.6 Water table (level and fluctuations)

4.	Original condition of structure	Good
4.1	Condition of formed and finished surfaces	Good
4.1.1	Smoothness	
4.1.2	Air pockets ("bugholes")	
4.1.3	Sand streaks	
4.1.4	Honeycomb	
4.1.5	Soft areas (retarded hydration)	
4.1.6	Cold joints	
4.1.41	Staining	
4.1.42	Sand pockets	
4.2	Defects	
4.2.1	Cracking	
4.2.1.1	Plastic shrinkage	
4.2.1.2	Thermal shrinkage	
4.2.1.3	Drying shrinkage	Observed on concrete barrier walls
4.2.19	Curling	
5.	Materials of Construction	See Tables 1 and 2
6.	Construction Practices	See Report pg. 3 and 5