PDHonline Course C418 (5 PDH)

High Performance Concrete

Instructor: Vincent D. Reynolds, MBA, PE

2020

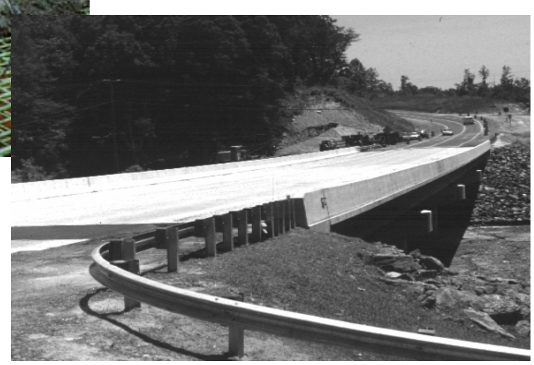
PDH Online | PDH Center

5272 Meadow Estates Drive
Fairfax, VA 22030-6658
Phone: 703-988-0088
www.PDHonline.com

U.S. Department
of Transportation
Federal Highway
Administration

HIGH PERFORMANCE CONCRETE
STRUCTURAL DESIGNERS' GUIDE

by the High Performance Concrete Technology Delivery Team



First Edition
March 2005

Disclaimer

HIGH PERFORMANCE CONCRETE
STRUCTURAL DESIGNERS' GUIDE

by the High Performance Concrete Technology Delivery Team

HIGH PERFORMANCE CONCRETE
STRUCTURAL DESIGNER'S GUIDE
TABLE OF CONTENTS

ORIENTATION

High Performance Concrete Technology Delivery Team

Introducing . . .

the Federal Highway Administration's
HIGH PERFORMANCE CONCRETE
TECHNOLOGY DELIVERY TEAM

Created to implement a mandate of the Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA) legislation, the Federal Highway Administration's (FHWA) High Performance Concrete Technology Delivery Team (HPC TDT) motivated and helped State DOT's to build more economical and durable bridges using high performance concrete. The TDT, created in 1997, assisted 13 States in design and construction of HPC bridges. Hundreds of State, Federal and industry personnel were introduced to HPC technology at workshops and showcases planned by the TDT and hosted by participating DOT's. Working with the American Association of State Highway and Transportation Officials (AASHTO) *Lead States Team on HPC Implementation*, the TDT influenced many additional State DOT's to try HPC in their highway bridges.

By the time the ISTEA legislation expired, about 25 States had used HPC. Today, the TDT continues to promote HPC and encourage states to build HPC bridges through the Innovative Bridge Research & Construction Program (IBRCP) created under the current highway program of TEA-21. HPC is considered an innovative material and projects can be funded under the guidelines of the IBRCP.

Two primary factors led to the rejuvenation of the HPC TDT. In 1998, the FHWA created Resource Center offices in Atlanta, Baltimore, Olympia Fields (IL), and San Francisco. These Centers were staffed to bring training, technical expertise and technology transfer specialists closer to state and local highway agencies. In addition, the TDT was being renewed with a focus on field delivery of HPC technology. Accordingly, TDT members represent the FHWA Resource Center; the Division Offices; the Agency's Headquarters Offices of Bridge and Pavement Technology; the Office of Infrastructure Research and Development; the Eastern Federal Lands Highway Division; and various State DOT's. Recognizing that earlier technology delivery efforts were the result of key partnerships and coordination, the new TDT also includes representatives from academia and industry.

One major initiative aimed at achieving our goal to educate users involves use of the world wide web, where a new "community of practice" website has been established. The site allows users to post questions on HPC, participate in discussions, share documents, and review works in progress. Visit the site at

http://knowledge.fhwa.dot.gov/cops/hpcx.nsf/home

Users will have the option to subscribe to an e-mail notification system where they will receive a summary of postings to the Community of Practice site for any of the following HPC subject areas that they choose:

- Definition and Research
- Structural Design/Specifications
- Mix Design/Proportioning
- Precast/Prestressed Beam Fabrication/ Transportation/Erection
- Cast-in-Place Construction
- Instrumentation/Monitoring/Evaluation
- Costs
- Case Studies/Lessons Learned

A new link has been added to results of a 2003-04 national survey on State DOT HPC implementation.

The focus of the new HPC TDT is to be the leader in advancing HPC technology for the benefit of our Nation's infrastructure. The business plan includes:

VISION:

"Be the leader in advancing HPC technology"

MISSION:

"Improve the durability and cost-effectiveness of the Nation's transportation infrastructure"

GOALS:

1) Establish HPC as standard practice for every State DOT

2) Partner with AASHTO and Industry to develop and lead a National agenda on HPC technology

3) Implement the AASHTO HPC Lead States' Team transition plan

4) Educate users on HPC practices in design, construction, and materials

To request more information about the HPC TDT contact the Web Administrator:

LOU TRIANDAFILOU (410) 962-3648
lou.triandafilou@fhwa.dot.gov
FHWA Resource Center

Or, other HPC TDT Members:

FHWA FIELD OFFICE CONTACTS

MATTHEW GREER
(720) 963-3008
matt.greer@fhwa.dot.gov
Colorado Division Office

DOUG EDWARDS
(404) 562-3673
douglas.edwards@fhwa.dot.gov
FHWA Resource Center

EDWARD PARKER
(404) 562-3643
edward.parker@fhwa.dot.gov
Georgia Division Office

RICH PAKHCHANIAN
(703) 404-6246
hratch.pakhchanian@fhwa.dot.gov
Federal Lands Highways

FRANK RICH
(402) 437-5967
frank.rich@fhwa.dot.gov
Nebraska Division Office

CLAUDE NAPIER
(804) 775-3363
claude.napier@fhwa.dot.gov
Virginia Division Office

MICHAEL PRAUL
(207) 622-8350 x109
michael.praul@fhwa.dot.gov
Maine Division Office

TOM SAAD
(708) 283-3521
thomas.saad@fhwa.dot.gov
FHWA Resource Center

JEFF SMITH
(404) 562-3905
jeff.smith@fhwa.dot.gov
FHWA Resource Center

FHWA HEADQUARTERS CONTACTS

GARY CRAWFORD
(202) 366-1286
gary.crawford@fhwa.dot.gov
Office of Pavement Technology

JOSEPH HARTMANN
(202) 493-3059
joey.hartmann@fhwa.dot.gov
Office of Infrastructure R&D

JON MULLARKY
(202) 366-6606
jon.mullarky@fhwa.dot.gov
Office of Pavement Technology

JERRY POTTER
(202) 366-4596
jerry.potter@fhwa.dot.gov
Office of Bridge Technology

MYINT LWIN
(202) 366-4589
myint.lwin@fhwa.dot.gov
Office of Bridge Technology

STATE DOT, INDUSTRY and ACADEMIA CONTACTS

MICHAEL BERGIN
(352) 955-6666
michael.bergin@dot.state.fl.us
Florida DOT

SHRI BHIDE
(847) 972-9100
sbhide@cement.org
National Concrete Bridge Council

PAUL TIKALSKY
(814) 863-5615
tikalsky@engr.psu.edu
Penn State University

DONALD STREETER
(518) 457-4593
dstreeter@dot.state.ny.us
New York State DOT

CELIK OZYILDIRIM
(434) 293-1977
celik@vdot.virginia.gov
Virginia Transportation Research Council

Madhwesh Raghavendrachar
(916) 227-7116
madhwesh.raghavendrachar@dot.ca.gov
Caltrans

KEVIN PRUSKI
(512) 416-2306
kpruski@dot.state.tx.us
(Texas DOT)

October 22, 2004
FHWA Pub. No. FHWA-ERC-02-006

ACRONYMS

AASHTO – American Association of State Highway and Transportation Officials

ACI – American Concrete Institute

ASR – Alkali-silica reactivity

ASTM – American Society of Testing and Materials

BLCCA – Bridge Life Cycle Cost Analysis

BMS – Bridge management system

CD – compact disk

C.I.P. – cast-in-place

CTH – Chloride Test, Hardened

CTL – Construction Technology Laboratories

DOT – Department of Transportation

E_c – modulus of elasticity (also MOE)

f'_c – concrete compressive strength

F_r – modulus of rupture (also MOR)

FC – future cost

FHWA – Federal Highway Administration

HPC – High Performance Concrete

HRWR – High range water reducer

HSC – High Strength Concrete

IC – initial construction

LCCA – Life Cycle Cost Analysis

LRFD – Load and Resistance Factor Design

NCHRP – National Cooperative Highway Research Program

NIST – National Institute of Standards and Technology

NSC – Normal strength concrete

OM&R – Operations, maintenance, repair and rehabilitation

PCA – Portland Cement Association

PCC – Portland Cement Concrete

PCI – Precast/Prestressed Concrete Institute

PONTIS – AASHTOWare BMS support tool for bridge maintenance, repairs, rehabilitiation and replacement

PV – Present value

QC/QA – Quality Control/Quality Assurance

RCPT – Rapid Chloride Permeability Test

RMT – Rapid Migration Test

SCC – Self-consolidating concrete

SHA – State Highway Agency (ies)

SHRP – Strategic Highway Research Program

TS & L – Type, size and location

U&TP – User and third party

UHPC – Ultra high performance concrete

w – unit weight

w/cm – water-cementitious materials ratio

WSDOT – Washington State DOT

SECTION 1

OBJECTIVE AND SCOPE

by Myint Lwin, P.E., FHWA and Lou Triandafilou, P.E., FHWA

1.1 Objective

The main objective of this High Performance Concrete (HPC) Structural Designers' Guide, referred to as the "Designers' Guide" throughout, is to provide a source of information to structural designers for the design and construction of highway bridges and related structures using HPC. This Guide will be updated periodically to keep pace with the latest developments in HPC, particularly as the American Association of State Highway and Transportation Officials (AASHTO) and industry organizations including the American Concrete Institute (ACI), the Precast/Prestressed Concrete Institute (PCI), the Portland Cement Association (PCA), the National Concrete Bridge Council (NCBC), the Post-Tensioning Institute (PTI), the American Segmental Bridge Institute (ASBI), etc. modify their codes or guide specifications to reflect new research findings and construction experiences.

1.2 Scope

The scope of the Designers' Guide is fairly comprehensive. It addresses all basic aspects of developing and producing HPC with desirable and beneficial characteristics for the transportation community.

Section 2 introduces the topic of HPC implementation in the United States highway infrastructure and provides historical context of this development. Section 3 addresses the characteristics and grades of HPC for various applications and environment. Section 4 is devoted to recently- completed national research and ongoing testing into the next generation of HPC, along with web links to State Department of Transportation research reports. Section 5 highlights material properties of HPC that are important to owners and designers in assuring long-term structural performance. Section 6 provides guidelines for developing HPC mix designs and proportioning of materials.

Section 7 focuses on the fabrication, transportation and erection of precast, prestressed HPC beams. Section 8 applies to HPC cast-in-place construction in substructures and superstructures, with special attention to the construction of bridge decks. Section 9 identifies the most common instruments that can be used for field measurement and recording of strain, deflection, rotation, acceleration and temperature of HPC members. Section 10 provides cost information and methods for assessing the cost-effectiveness of HPC with guidelines for estimating initial construction cost and life-cycle cost. Finally, Section 11 provides an overview of several HPC projects across the U.S. with lessons learned and contact information or web links for further details.

SECTION 2

INTRODUCTION

by Myint Lwin, P.E. , FHWA and Lou Triandafilou, P.E., FHWA

2.1 The Beginning and Advancement

For over ten years, the international community has taken great strides with implementing High Performance Concrete (HPC) technology in an effort to extend the service life of pavements and bridges. Forty-five U.S. Departments of Transportation, the District of Columbia, Puerto Rico and several Federal agencies responded to a recent survey that they have incorporated HPC specifications in projects involving either bridge decks, superstructures and/or substructures (See enclosed map). These projects took advantage of either the high strength or high durability attributes of HPC, or both.

The term HPC is used to describe concretes that are made with carefully selected high quality ingredients, optimized mixture designs, and which are batched, mixed, placed, consolidated and cured to the highest industry standards. Typically, HPC will have a water-cementitious materials ratio (w/cm) of 0.4 or less. Achievement of these low w/cm concretes often depends on the effective use of admixtures to achieve high workability, another common characteristic of HPC mixes.

Several definitions have emerged over the years to acquaint the engineering community and concrete industry with HPC. According to ACI, HPC is defined 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. The Federal Highway Administration (FHWA) promoted HPC in the 1990's by defining it in 1996 using four durability and four strength parameters. Associated with each definition parameter were performance criteria, testing criteria, testing procedures to measure performance, and recommendations to relate performance to adverse field conditions (per Cook, Goodspeed and Vanikar). More recently, the National Concrete Bridge Council has drafted a definition for HPC as, "…concrete that attains mechanical, durability or constructability properties exceeding those of normal concrete." Section 3 of this Designers' Guide provides information relative to the current definition and performance characteristics of HPC.

Regardless of the definition, HPC is an advancement in concrete technology that has become commonplace and the state-of-the-practice, rather than the exception to the rule. It has provided transportation departments a construction material with characteristics engineered to ensure satisfactory performance throughout its intended service life.

SECTION 3

DEFINITION AND CHARACTERISTICS

by Celik Ozyildirim, P.E., Virginia DOT, Jerry Potter, P.E., FHWA and Don Streeter, P.E., New York State DOT

3.1 Introduction

In response to the need to extend service life and to construct cost-effective structures, FHWA has been promoting HPC. For a clear understanding of HPC, the FHWA has prepared a definition of HPC, based on long-term performance criteria. The HPC definition is expected to stimulate the use of higher quality concrete in highway structures. The proposed definition consists of durability and strength parameters. Associated with each definition parameter are performance criteria, testing procedures to measure performance and recommendations to relate performance to adverse field conditions.

3.2 Definition

HPC is defined by the American Concrete Institute as concrete that meets special combinations of performance and uniformity requirements that cannot always be achieved routinely using conventional constituents and normal mixing, placing, and curing practices (ACI 116R). Section 2 of this Designers' Guide references other definitions that have emerged for HPC.

3.3 Material and Performance Characteristics

Different characteristics of concrete in the fresh and hardened states affect performance. In the fresh state, flowability is an important characteristic. It describes the ease or difficulty of placing the concrete depending on the equipment available. The adequacy of flow for a specific job will affect the quality of the finished product. Concrete with high flowability is easy to place and facilitates the removal of undesirable air voids in concrete. In fact, self-consolidating concrete (SCC) is available that flows through heavily reinforced areas or demanding places and consolidates under its own mass. Well-consolidated concretes (either through mechanical vibration or mix design, as in SCC) are essential in achieving low permeability for long-lasting structures. The important characteristics of concrete in the hardened state mainly relate to durability and structural design.

The performance characteristics related to durability include freeze-thaw resistance, scaling resistance, abrasion resistance, chloride ion penetration, alkali-silica reactivity, and sulfate resistance. The four structural design characteristics are compressive strength, modulus of elasticity, shrinkage, and creep. The characteristics are determined using standard test procedures, and grades of performance are suggested for each characteristic. Durability is of utmost importance for structures exposed to the environment and concrete for each structure may need one or more of these characteristics.

The material characteristics and grades should be selected in accordance with the intended application and the concrete's environment. For example, a bridge deck supported on girders needs a specified compressive strength but is unlikely to require specified values for modulus of elasticity and creep. It is not necessary to require all performance characteristics for a given application. Grades of performance characteristics for high performance structural concrete are given in Table 3-1.

Other important features of HPC are uniformity and consistency. With high variability, the concrete has a high potential for not meeting the specifications.

3.4 References

1. *Compilation and Evaluation of Results from High Performance Concrete Bridge Projects* by H. G. Russell, R. A. Miller, H. C. Ozyildirim, and M. K. Tadros. The report also includes test procedures and examples of characteristics specified and achieved in different states.

2. *High Performance Concrete Defined for Highway Structures* by Goodspeed, C.H., Vanikar, S., and Cook, R, Concrete International, Vol. 18, No. 2, February 1996, pp. 62-67

Table 3-1. *Grades of performance characteristics for high performance structural concrete*[1]

Performance characteristic[2]	Standard test method	FHWA HPC performance characteristic grade[3]		
		1	2	3
Freeze-thaw durability[4] (F/T=relative dynamic modulus of elasticity after 300 cycles)	AASHTO T 161 ASTM C 666 Proc. A	$70\% \leq F/T < 80\%$	$80\% \leq F/T < 90\%$	$90\% \leq F/T$
Scaling resistance[5] (SR=visual rating of the surface after 50 cycles)	ASTM C 672	$3.0 \geq SR > 2.0$	$2.0 \geq SR > 1.0$	$1.0 \geq SR \geq 0.0$
Abrasion resistance[6] (AR=avg. depth of wear in mm)	ASTM C 944	$2.0 > AR \geq 1.0$	$1.0 > AR \geq 0.5$	$0.5 > AR$
Chloride penetration[7] (CP=coulombs)	AASHTO T 277 ASTM C 1202	$2500 \geq CP > 1500$	$1500 \geq CP > 500$	$500 \geq CP$
Alkali-silica reactivity (ASR=expansion at 56 d) (%)	ASTM C 441	$0.20 \geq ASR > 0.15$	$0.15 \geq ASR > 0.10$	$0.10 \geq ASR$
Sulfate Resistance (SR=expansion) (%)	ASTM C 1012	$SR \leq 0.10$ at 6 months	$SR \leq 0.10$ at 12 months	$SR \leq 0.10$ at 18 months
Flowability (SL=slump, SF=slump flow)	AASHTO T 119 ASTM C 143, and proposed slump flow test	SL>190 mm (SL>7-1/2 in), and SF<500 mm (SF<20 in)	$500 \leq SF \leq 600$ mm ($20 \leq SF \leq 24$ in)	600 mm<SF (24 in<SF)
Strength[8] (f'_c=compressive strength)	AASHTO T 22 ASTM C 39	$55 \leq f'_c < 69$ MPa ($8 \leq f'_c < 10$ ksi)	$69 \leq f'_c < 97$ MPa ($10 \leq f'_c < 14$ ksi)	97 MPa$\leq f'_c$ (14 ksi$\leq f'_c$)
Elasticity[9] (E_c=modulus of elasticity)	ASTM C 469	$34 \leq E_c < 41$ GPa ($5 \leq E_c < 6x10^6$ psi)	$41 \leq E_c < 48$ GPa ($6 \leq E_c < 7x10^6$ psi)	48 GPa$\leq E_c$ ($7x10^6$ psi$\leq E_c$)
Shrinkage[10] (S=microstrain)	AASHTO T 160 ASTM C 157	$800 > S \geq 600$	$600 > S \geq 400$	$400 > S$
Creep[11] (C=microstrain/pressure unit)	ASTM C 512	$75 \geq C > 55$/MPa ($0.52 \geq C > 0.38$/psi)	$55 \geq C > 30$/MPa ($0.38 \geq C > 0.21$/psi)	30/MPa$\geq$C (0.21/psi$\geq$C)

1 This table does not represent a comprehensive list of all characteristics that good concrete should exhibit. It does list characteristics that can quantifiably be divided into different performance groups. Other characteristics should be checked. One characteristic is sufficient for classification as an HPC.
2 For non-heat cured products, all tests to be performed on concrete samples moist, submersion, or match cured for 56 days or until test age. For heat cured products, all tests to be performed on concrete samples cured with the member or match cured until test age. See table 13 of the Henry Russell report for additional information and exceptions, or Table 2 in the FHWA publication located at http://www.fhwa.dot.gov/bridge/hpcdef.htm
3 A given HPC mix design is specified by a grade for each desired performance characteristic. A higher grade number indicates a higher level of performance. Performance characteristics and grades should be selected for the particular project. For example, a concrete may perform at grade 3 in strength and elasticity, grade 2 in shrinkage and scaling resistance, and grade 2 in all other categories.
4 Based on SHRP C/FR-91-103, p. 3.52.
5 Based on SHRP S-360.
6 Based on SHRP C/FR-91-103.
7 Based on PCA Engineering Properties of Commercially Available High-Strength Concretes, RD104
8 Use lower strengths for decks and substructures
9 Based on SHRP C/FR-91-103, p. 3.17. Modulus of elasticity is related to strength and if a lower strength is specified for decks, the MOE should be proportionally lower. See AASHTO LRFD Bridge Design Specifications, Article 5.4.2.4.
10 Based on SHRP C/FR-91-103, p. 3.25.
11 Based on SHRP C/FR-91-103, p. 3.30.

SECTION 4

RESEARCH

by Ben Graybeal, P.E., PSI, Inc.; Joseph Hartmann, P.E., FHWA; and Marcia Simon; FHWA

4.1 Introduction

During the past decade, the FHWA and State DOTs have performed and overseen the completion of many research projects that have contributed to the implementation of HPC. Several areas of the AASHTO bridge design and construction specifications have been updated based on successful research and the advancement of High Performance Concrete (HPC) state-of-the-art technology. Two research areas have the potential for reaping many benefits of HPC technology. These are ultra-high performance concrete (UHPC) and the rapid migration test (RMT) for evaluating chloride penetration resistance.

One form of UHPC is a steel fiber-reinforced concrete consisting of an optimized gradation of fine powders and a very low water/cementitious materials ratio. Compressive strength testing has produced results ranging from 18 ksi to 28 ksi. Tensile strengths have ranged from 0.9 to 1.7 ksi, also depending on the curing procedure. Rapid chloride penetration results have ranged from extremely low to very low, and freeze-thaw and scaling values indicate that UHPC exhibits enhanced durability to resist environmental attack.

The RMT is capable of providing results on HPC cylinders within 3 to 7 days. This new test answers some of the criticisms of the current rapid chloride permeability test (RCPT) method (AASHTO T277/ASTM C1202). It is less affected by the presence of conductive ions than the RCPT, the applied voltage is generally lower so there is no temperature increase during testing, and the depth of chloride ion penetration is measured. Test results can also be used to calculate diffusion coefficients as inputs to service life and life-cycle cost models.

This section of the Guide will explore in more detail the current status of these two research efforts. Also, the Reference subsection contains a partial listing of State DOT websites where additional HPC-related research reports may be accessed. Other State DOTs, the FHWA, or the Transportation Research Board may be contacted for additional reports.

4.2 Ultra High Performance Concrete (UHPC) Research Program at FHWA

The ongoing research into UHPC at FHWA can be divided into four phases as subsequently described.

4.2.1 Phase 1: AASHTO Type II Girder Testing

The first phase of the research into UHPC focused on determining the structural behavior of the AASHTO Type II prestressed girder. The prestressed girders tested contained no mild steel reinforcement and were composed of a fiber reinforced UHPC. The testing focused on determining the flexural and shear behavior from initial loading through failure. The results of these tests indicated that UHPC exhibits some remarkable structural properties.

Photo 4.2.1.1 *– UHPC Flexure Test at Turner-Fairbank Highway Research Center*

Photo 4.2.1.2 *– UHPC Shear Test at FHWA Turner Fairbank Highway Research Center*

4.2.2 Phase 2: UHPC Material Characterization

In order to make efficient use of UHPC in highway bridges, a full suite of material testing was undertaken to fully characterize the behavior of UHPC. Provided below is a listing of the testing that has been completed or is currently underway.

4.2.2.1.1 Material:

- As supplied by Lafarge
- ½" long steel fibers
- 200 MPa compressive strength with steam cure

4.2.2.2 Time Table:

- Tests started Summer 2002, most testing to be completed by Early '04
  - Creep and shrinkage tests are a notable exception as they did not start until mid-2003 and will not be completed until mid-2004

4.2.2.3 Primary variable to be investigated:

- Curing regime applied to the concrete
  - 48 h our steam treatment one day after stripping molds
  - 48 hour steam treatment 2 weeks after stripping molds
  - Air cure (laboratory ambient conditions)
  - 48 hour elevated temp/humidity treatment (140ºF and 95% humidity)

4.2.2.4 Testing to be completed:

- Compressive Strength
  - Cylinder (4x8", 3x6", 2x4") according to ASTM C39
  - Cube (4x4", 2x2") according to ASTM C109
  - Cylinders at various ages after casting

- Elastic Modulus and Poisson's Ratio
  - ASTM C469 on 3x6" cylinders
  - Various ages after casting

- Split Cylinder Tensile
  - ASTM C496 on 4x8" cylinders
  - Various ages after casting

- Notched Cylinder Direct Tension
  - 4x8" cylinders according to modified RILEM specification
  - Primarily focused on post cracking behavior

- Unnotched Cylinder Direct Tension
  - USBR 4914 on 4x8" cylinders
  - Obtain elastic modulus
  - Primarily focused on pre-cracking behavior
- Mortar Briquette
  - AASHTO T132
  - 1x1" cross-section briquette (concrete dogbone)
  - Uniaxial tension test

- Prism Flexure
  - Four-point loading according to ASTM C1018
  - 2x2” cross-sections with 6”, 9”, 12”, 15” spans
  - 3x4” cross-sections with 16” span

- Freeze-Thaw
  - ASTM C666 on a 3x4x16” prism

- Abrasion Resistance
  - ASTM C944 on the molded surface from a steel cylinder mold

- Alkali-Silica Reaction
  - ASTM C1260 on a 1x1x11” prism

- Rapid Chloride Penetration
  - ASTM C1202 on a 4” diameter cylinder slice

- Ponding Chloride Penetration
  - AASHTO T259 on a 4” diameter cylinder cast end

- Scaling Resistance
  - ASTM C672 on cast side of a 3x12x12” slab

- Creep
  - ASTM C512 on 4x8” cylinders
  - Loaded to 40% of Steamed Compressive Strength

- Unrestrained Shrinkage
  - ASTM C157 on both 4x8” cylinders and 3x3x11” prisms

- Early Age Shrinkage
  - ASTM C157 on 3x3x11” prisms with embedded vibrating wire gages

- Thermal Expansion
  - 4x8” cylinders

- Bend Bar Fatigue
  - ASTM E399 adaptation for concrete
  - 2x4” cross-section notched prism with crack mouth opening gage

- Heat of Hydration
  - 3x6” cylinders with thermocouple
  - 6x12” cylinders with thermocouple
  - 6x12” cylinders with adiabatic heat of hydration system

- Air Void
  - ASTM C457 using 4x8” cylinders
  - Computed Tomography using 4x8” cylinders

- Fiber Dispersion and Orientation
  - ASTM C457 using 4x8” cylinders
  - Computed Tomography using 4x8” cylinders

Table 4.2.2-1 *UHPC Test Program Status as of April 2004*

Test	Test Status		
	Preparatory Phase	**In Progress**	**Complete**
Cylinder Compression Strength			x
Cylinder Compressive Modulus			x
Cube Compressive Strength			x
Split Cylinder Tensile Strength			x
Mortar Briquette Tensile Behavior			x
Prism Flexural Behavior (Static)			x
Prism Flexural Behavior (Fatigue)		x	
Creep			x
Unrestrained Shrinkage			x
Freeze-Thaw Resistance			x
Scaling Resistance			x
ASR Resistance			x
Rapid Chloride Penetration			x
Ponding Chloride Penetration			x
Abrasion Resistance			x
Thermal Expansion			x
Heat of Hydration			x
Air Void			x
Load Rate Effect on Compressive Strength/Modulus			x
Strip Time Effect on Compressive Strength			x

4.2.3 Phase 3: Optimization of Girder Cross-Sections for UHPC

An analytical study to determine an efficient highway bridge girder shape has been completed and has yielded a double-T like section. This prestressed girder contains no mild steel, has two 2” thick webs, a 3” thick deck, and is only 33” deep. This cross-section is designed for use with bridges that span between 70 and 100 feet.

4.2.4 Phase 4: Construction of an Optimized UHPC Bridge

The optimized section that resulted from Phase 3 has since been used in the construction of four optimized UHPC girders in the next phase of the program. Between November 2003 and January 2004, four 70 foot long optimized UHPC girders were constructed. The girders are 70 feet long with a depth of 33". The deck is 8 feet wide and only 3" thick. Two of the four girders have been used to construct a demonstration bridge at the Turner-Fairbank Highway Research Center.

The bridge will be periodically tested and monitored for several years. The other two girders will be destructively tested at TFHRC to determine a baseline behavior for this girder shape.

4.3 Rapid Migration Test

4.3.1 Introduction

AASHTO TP64-03, "Prediction of Chloride Penetration in Hydraulic Cement Concrete by the Rapid Migration Procedure," [1] also known as the Rapid Migration Test (RMT), was developed for FHWA by the University of Toronto. The goal in developing this test was to address some concerns and limitations with the AASHTO T-277 [2], commonly referred to as the "Rapid Chloride Permeability Test" (RCPT). The RMT is based on the CTH test[1] developed at Chalmers Technical University in Sweden by Tang and Nilsson.

4.3.2 Summary of RMT procedure

The RMT resembles the RCPT in some respects. Both tests use a 50 mm x 100 mm cylindrical test specimen that is exposed to NaCl solution on one side and NaOH solution on the other side. The NaCl solution concentration in the RMT is 10% by mass (compared with 3% in the RCPT). The NaOH concentration in both tests is 0.3 N. Specimen preparation before testing (epoxy coating, vacuum saturation) is similar as well.

In the RMT, as in the RCPT, an external potential applied across the specimen forces chloride ions to migrate into the specimen. In the RCPT, the applied voltage is always 60V; however, the applied voltage in the RMT varies depending on the initial current measured at 60V. The applied voltage is adjusted depending on the current reading –
specimens with higher initial current readings will have lower applied voltages, to reduce problems associated with heating. The initial current ranges and corresponding voltages are defined in Table 1. The duration of the RMT is 18 hours[2].

[1] The CTH test has been standardized by Nordtest as NT Build 492 (see Reference 4).
[2] In NTBuild 492, a similar table (different and more complex than the RMT table) defines both the applied voltage and test duration based on initial current at 30V.

Table 4.3.2 -1 ***Applied voltage during testing based on initial current at 60 VDC***

Initial current at 60 VDC (mA)	Applied Voltage (V)	Test Duration (h)
<120	60	18
120-240	30	18
240-800	10	18
>800	do not test	do not test

After 18 hours, the RMT specimen is removed from the solutions, split axially, and sprayed with 0.1 N silver nitrate solution (silver nitrate is a colorometric indicator for chloride where the chloride concentration exceeds 0.7 percent). Measurements of the depth of chloride penetration (defined by the extent of the white silver chloride precipitate) at several locations along the exposed surface are averaged to obtain the penetration depth. A rate of chloride penetration is calculated by dividing the measured depth of penetration by the product of the applied voltage and test duration. The concrete's performance in terms of chloride penetration is classified according to this rate.

A schematic of the prototype apparatus is shown in Figure 4.3.2-1. The prototype consists of a platform (cathode) capable of holding 2-3 test specimens. The test specimens are placed in rubber sleeves as indicated in the figure. The platform is placed in a large tub and the specimens are placed on the platform.

NaCl solution (10% by mass) is placed in the tub, and NaOH solution (0.3N) is ponded above the specimen inside the rubber sleeve. An anode is placed in the NaOH solution and voltage is applied.

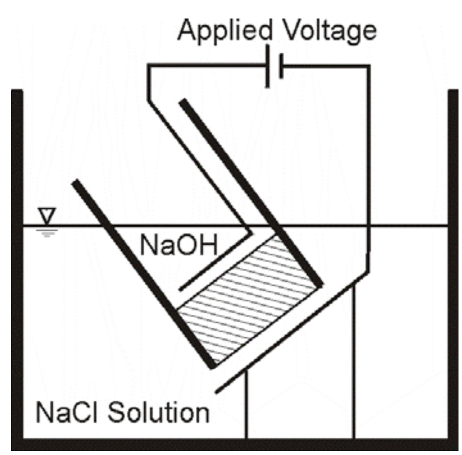


Figure 4.3.2 -1 *Schematic of prototype device (after Reference 3)*

4.3.3 Advantages

(1) Use of variable voltage settings reduces problems associated with heating.

(2) The measurement of chloride penetration depth is specific to chloride ions, whereas in the RCPT measurement of charge passed reflects all ions migrating through the concrete.

(3) The measurement of depth is more intuitive as a measure of chloride penetration than the RCPT measurement of charge passed (which is more correctly a measurement of conductivity or resistivity).

(4) The test shows less variation than AASHTO T277.

4.3.4 Disadvantages

(1) Although the prototype device works well, it requires a large volume of NaCl solution (approximately 10 gallons per test) to fill the tub to the required depth.

(2) The test takes 18 hours to perform.

(3) Many State DOTs and other labs have already invested in AASHTO T277 test cells; therefore, it would be advantageous to be able to use these cells in the RMT, which is functionally very similar to AASHTO T-277.

The FHWA has successfully run the RMT test using RCPT test cells connected to programmable power supplies. Also, at least one commercially available T277 system advertises that it can be used for NTBuild 492 testing as well. It should be possible to run the RMT using such a system as well.

4.4 References

1. AASHTO TP64-03, "Prediction of Chloride Penetration in Hydraulic Cement Concrete by the Rapid Migration Procedure," American Association of State Highway and Transportation Officials, Washington, D.C., 2003.

2. AASHTO T277, "Electrical Indication of Concrete's Ability to Resist Chloride," American Association of State Highway and Transportation Officials, Washington, D.C., 1993.

3. Graybeal, B.A. and Hartmann, J.L., "Strength and Durability of Ultra-High Performance Concrete," October 2003.

4. Hooton, R.D., Thomas, M.D.A, and K. Stanish, *Prediction of Chloride Penetration in Concrete*, Report #FHWA-RD-00-142, Federal Highway Administration, October, 2001.

5. NTBuild 492, "Chloride Migration Coefficient from non-steady state migration experiments", Nordtest, Espoo, Finland, 1999.

State DOT Research Websites:
Florida DOT: www.dot.state.fl.us/research-center/Completed_StateMaterials.htm
www.dot.state.fl.us/research-center/Active_StateMaterials.htm
Virginia DOT: http://www.virginiadot.org/vtrc/main/index_main.htm

SECTION 5

STRUCTURAL DESIGN AND SPECIFICATIONS FOR HIGH STRENGTH CONCRETE

by Shri Bhide, P.E., Portland Cement Association; Tom Saad, P.E., FHWA; and Jeff Smith, P.E., FHWA

5.1 Introduction

Long-term performance benefits can be achieved in highway structures when high performance concrete (HPC) is properly used in the structural system. The main benefits for utilizing high strength concrete (HSC) in bridge elements is to extend span lengths of bridges with commonly fabricated girder types, reduce the depth of superstructures, and eliminate girder lines to offer cost-efficiency.

The vast majority of bridges in the United States are constructed with concretes with compressive strengths less than 10 ksi. From the structural engineer's viewpoint, the major impediment to deploying HPC has been the limited validation of design provisions for HSC. It provides a synthesis of information on the mechanical properties of HSC and offers guidance to structural engineers regarding the use of HSC. The information presented is based on the state-of-the-art of HSC and may not be included in the AASHTO bridge design specifications.

The AASHTO Standard Specifications limit the compressive strength of prestressed concrete to 5 ksi. In the AASHTO Load and Resistance Factor Design (LRFD) Specifications the limit has been increased to 10 ksi. Both specifications allow, at the discretion of the Engineer, the use of higher strengths if tests are conducted to establish various mechanical properties of concrete. Furthermore, the LRFD specification prohibits the use of concrete with strengths below 2.4 ksi at 28 days.

Research is currently underway to validate, and/or improve design provisions in the LRFD specifications with regard to HSC. Three NCHRP research projects in particular are NCHRP Projects 12-56, 12- 64, and 12-60. These projects deal with shear design; flexural and compression members; and transfer, development, and splice length for prestressed and non-prestressed reinforcement, respectively.

5.2 Cost Effective Designs

More cost effective designs are possible with HPC. This is due to the enhanced mechanical properties and the improved durability characteristics of HPC. The performance benefits give the designers greater flexibility in selecting the type and size of a bridge and bridge elements. The designers are able to use less materials, fewer beams and longer spans for their HPC projects. The long-term durability of HPC results in lower maintenance and fewer repairs. All these sum up to lower construction and life-cycle costs. The three basic cost elements of a concrete structure are materials, labor, and markup. Each cost element is affected when HPC is used.

The primary materials in a concrete structure are concrete, prestressing steel, and non-prestressed steel reinforcement. The demand for higher performance naturally leads to higher material costs:

(1) Concrete - An HPC mix is roughly 30 to 40 percent more expensive than a conventional concrete mix. This is primarily due to a higher cementitious material content. It is important for the designers to specify the minimum required concrete strength at each stage of construction, such as at release of prestress, at handling and shipping, form removal, and in service. This allows the contractor and fabricator to select the least expensive mix to achieve the design objectives and reduces the risk associated with achieving high concrete strengths.

(2) Prestressing Steel - More prestressing steel is required to develop the higher prestress levels possible. The use of 0.6-inch diameter strand is often necessary to provide these higher prestress levels. Currently, 0.6-inch diameter strand costs slightly more than a ½-inch strand on a unit weight basis. However, since fewer strands are needed when using 0.6-inch strands, the overall cost may not be significantly different. The designers may consider optimizing the girder sections for greater economy.

(3) Non-prestressed Reinforcement - The use of steel reinforcing bars in prestressed girders is nominal. No significant increase in cost is expected.

The labor required for the construction and fabrication of an HPC structure is not much different than for conventional concrete structures. For fabrication plants that have not utilized HPC, the startup labor cost may be increased due to some changes in standard tooling, such as, changing from ½-inch strand to 0.6-inch strand.

The markup, which covers overhead, profit and risk, is expected to initially be higher for HPC. HPC is perceived to have a higher risk, particularly for contractors and fabricators who are not familiar with it. The designers can help minimize this risk factor by specifying only the minimum concrete strengths required by the design and by communicating with the fabricators early on in the design.

When all three cost elements are summed together, the current cost of an HPC girder will be roughly 10 to 15 percent higher per linear foot than a standard girder. This increase in cost can be easily offset by the need for fewer girders or piers in the structure.

5.3 Material Properties

The primary material properties that impact the structural design of concrete components are compressive strength, modulus of elasticity, unit weight, modulus of rupture, and creep and shrinkage coefficients.

5.3.1 Stress-strain curve

The stress-strain curve for HSC is different than that for normal strength concrete (NSC). This has an effect on the equivalent rectangular stress block parameters, reinforcement limits and strength of composite section. Modifications are necessary for the efficient use of HSC.

Figure 5.3.1-1 shows typical stress-strain curves for a range of concrete strengths. It can be seen that as the concrete strength increases, the concrete stress-strain curves exhibit increased initial stiffness and greater linearity.

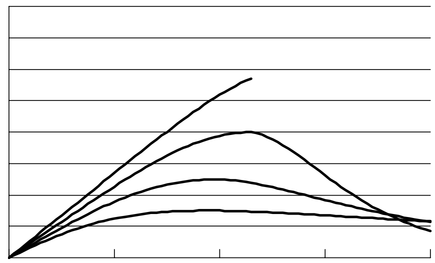


Figure 5.3.1-1 *Concrete Stress-Strain Curves in Compression*

For normal strength concrete, the compressive stress block shape is parabolic. The equivalent stress block is idealized as a rectangular stress block, Figure 5.3.1-2. The maximum compressive strength is multiplied by 0.85 to give the design stress intensity and the neutral axis depth is multiplied by a factor, β_1, which varies from 0.85 for concrete strengths equal to 4 ksi to 0.65 for concrete strengths greater than or equal to 8 ksi, to determine the depth of the rectangular block.

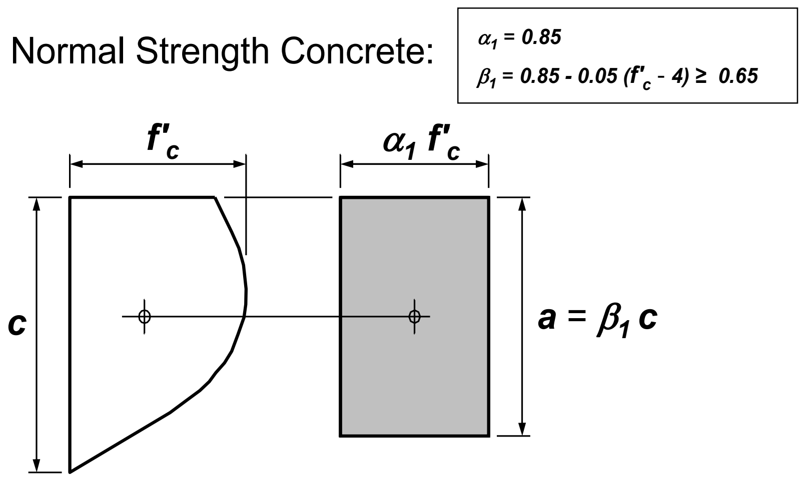


Figure 5.3.1-2 *Equivalent Rectangular Stress Block for Normal Strength Concrete*

For very high strength concrete the idealized stress-strain curve is almost linear up to and beyond a strain of 0.003. As a result the idealized concrete stress block is triangular in shape as shown in Figure 5.3.1-3. The maximum stress occurs at the top fiber and is zero at the neutral axis of the cross section.

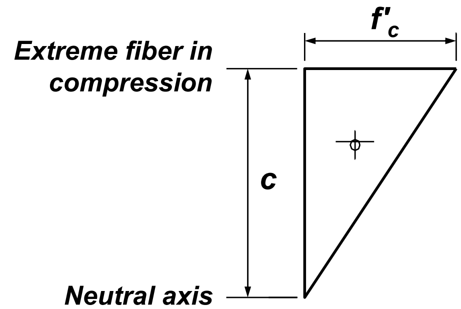


Figure 5.3.1-3 *Idealized Stress-Strain Curve for HSC*

The equivalent stress block is idealized as a rectangular stress block as shown in Figure 5.3.1-4. If the equivalent stress block depth factor, β_1, is set equal to 0.65, the coefficient, α_1, needs to be equal to 0.75 in order to maintain an equivalent force level between the triangular and rectangular stress blocks. To maintain equivalent force level between the triangle and the rectangle, the alpha-1 coefficient should be 0.75 rather than the conventional 0.85.

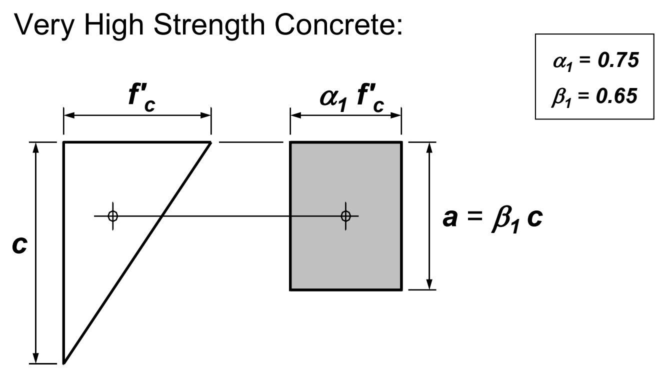


Figure 5.3.1-4 *Equivalent Stress Block for HSC*

5.3.2 Compressive strength

Concrete properties such as modulus of elasticity, tensile strength, shear strength, and bond strength are frequently expressed in terms of compressive strength. Generally, expressions for these quantities have been empirically established based on data for concrete having compressive strengths up to 6 ksi. Modifications to these equations may be necessary for HSC.

When using HPC it is necessary to modify the compression test procedures. The compressive force required for 6-inch by 12-inch cylinders made of HSC may exceed the capacity of existing equipment. To avoid this drawback, 4-inch by 8-inch cylinders with the proper end treatment can be used for compression tests (*HPC Bridge Views* Issue No. 14).

5.3.3 Modulus of elasticity

The modulus of elasticity, E_c, is used to calculate deflections, stresses under service loads, camber, and prestress losses of concrete members. Five options are listed for determining E_c. The AASHTO LRFD Specifications and the ACI 318 Building Code specify the identical equation for calculating the modulus of elasticity:

$$E_c = 33{,}000\ K_1(w)^{1.5}\sqrt{f_c^{'}} \qquad (5.3.3\text{-}1)$$

The correction factor, K_1, is applied to account for the source of the aggregate. It is taken as 1.0 unless physical tests give a different value that is approved by the bridge owner (2005 AASHTO).

A significant amount of research has been performed to determine a standard equation for modulus of elasticity for various concrete strengths. The type of coarse aggregate used has a strong influence on the modulus of elasticity of concrete. Equation 5.3.3-1 may not be suitable for HSC. It is best to experimentally determine the modulus of elasticity of HSC to be used in a specific project.

5.3.4 Unit weight

HSC is typically a dense concrete and as such its unit weight is higher than the unit weight of normal strength concrete. Tests conducted at the University of Nebraska showed (Figure 5.3.4-1) that the unit weight of high strength concrete may be as high as 0.160 kips per cubic foot (kcf). It is important to use the proper value for an accurate evaluation of the modulus of elasticity and for other structural calculations.

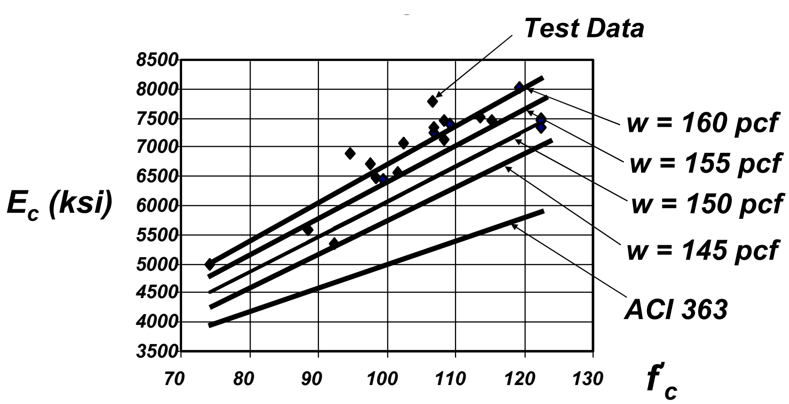


Figure 5.3.4-1 *Unit Weight of HSC*

5.3.5 Modulus of rupture

The AASHTO LRFD Bridge Design Specifications, in Article 5.4.2.6, establish the modulus of rupture, f_r, or flexural tensile strength, of conventional strength concrete to be:

For normal weight concrete:

When calculating the cracking moment of a member for the control of cracking by distribution of reinforcement (LRFD 5.7.3.4) and for deflection and camber (LRFD 5.7.3.6.2)(2005 AASHTO):

$$f_r = 0.24\sqrt{f_c'} \quad (f_c' \text{ in ksi}) \quad (5.3.5\text{-}1)$$

When calculating the cracking moment of a member for the minimum amount of reinforcement (5.7.3.3.2), the modulus of rupture should be less than (2005 AASHTO):

$$f_r = 0.37\sqrt{f_c'} \quad (f_c' \text{ in ksi}) \quad (5.3.5\text{-}2)$$

For sand-lightweight concrete:

$$f_r = 0.20\sqrt{f_c'} \quad (f_c' \text{ in ksi}) \quad (5.3.5\text{-}3)$$

For all-lightweight concrete:

$$f_r = 0.17\sqrt{f_c'} \quad (f_c' \text{ in ksi}) \quad (5.3.5\text{-}4)$$

These equations are thought to underestimate the flexural strength of HSC. For concretes with compressive strengths between 10.0 and 13.0 ksi, the modulus of rupture may be greater than predicted by the equations in the LRFD Specifications.

5.3.7 Creep coefficient

Creep is deformation under sustained load. The measure of creep is used to determine prestress loses, stress redistribution and deflection in concrete members and continuity reinforcement over piers. The magnitude and duration of the applied stress and the maturity of the concrete at the time of load application influence the magnitude of the creep.

There are several methods for calculating the amount of creep. The equation for calculating creep of concrete provided in the LRFD Specifications (LRFD Equation 5.4.2.3.2-1) is an improvement over the one in the AASHTO Standard Specifications and can be used estimate creep for concretes with compressive strengths up to 13 ksi.

Revised provisions for calculating concrete creep were adopted at the 2004 AASHTO Subcommittee on Bridges and Structures annual meeting. They are based on the results of NCHRP Project 19-07, NCHRP Report 496. The new provisions yield approximately the same results as the previous provisions for conventional strength concrete and more accurate results for high strength concretes (2005 AASHTO).

5.3.8 Shrinkage coefficient

Similar to the creep coefficient, the shrinkage coefficient equation in the current version of the LRFD Specifications can be used for HSC with compressive strengths up to 13.0 ksi. Past equations for the calculation of the shrinkage coefficient in the Standard Specifications greatly underestimated shrinkage. The LRFD equations take into account recent research conducted on high strength concrete.

Revised provisions for calculating concrete shrinkage were adopted at the 2004 AASHTO Subcommittee on Bridges and Structures annual meeting. They are based on the results of NCHRP Project 19-07, NCHRP Report 496. The new provisions yield approximately the same results as the previous provisions for conventional strength concrete and more accurate results for high strength concretes (2005 AASHTO).

5.4 Flexure

Flexural members are designed to limit stresses, deformations, and crack width under service conditions. Their design also ensures development of significant and visible inelastic deformations before failure in strength and extreme event limit states.

Factored resistance at the strength limit state is the product of the nominal resistance and an appropriate resistance factor. As discussed in Section 5.3.1, a uniform rectangular stress distribution using stress block parameters, γ and β_1, is frequently used to determine the nominal flexural resistance. The width of the equivalent rectangular stress block is taken as a fraction of the concrete compressive strength:

$$\gamma f_c' \qquad (5.4\text{-}1)$$

For conventional concrete, γ is taken to be 0.85. The depth of the equivalent rectangular stress block is taken as a fraction of the depth to the neutral axis of the section:

$$\beta_1 c \qquad (5.4\text{-}2)$$

The stress block factor, β_1, for conventional concrete is 0.85 for concrete strengths up to 4.0 ksi and is reduced by 0.05 for each 1.0 ksi of strength in excess of 4.0 ksi, but is not reduced below 0.65. The stress block factor will reach this limit at concrete strengths above 8 ksi. Although the rectangular stress distribution may still be valid for the design of flexural members at higher strengths, the values of γ and β_1 for higher strength concrete may need to be adjusted (See section 5.3.1).

When using the equivalent stress block method for flexural design gross section properties are typically used. Transformed section properties can be used and the LRFD Specifications now allow this. However, using transformed section properties in hand calculations may become burdensome, as the transformed section changes with the addition or deletion of reinforcement and multiple iterations are needed to reach a final design.

The resistance factors for conventional concrete analysis and design may not be suitable for use with HSC mixes. Higher strengths are often achieved by using chemical and mineral admixtures. This can lead to an increase in the variability of the concrete properties. However, HSC is usually produced under tighter quality control, which can lead to a lower coefficient of variation.

5.4.1 Tensile and compressive stresses

Although HSC has higher unit weight, it also has improved mechanical properties. As such, the stresses in HSC flexural members may not be significantly different from those of normal strength concrete flexural members. Service level stresses depend on the magnitude of both dead and live loads. For equivalent sized members, a higher unit weight for HSC will result in higher dead load moments and stresses.
HSC may also impact the magnitude of live load stresses in a member through the change in modular ratio that results from a higher modulus of elasticity. Although HSC has a higher unit weight, it also has improved mechanical properties and as a result, higher stress limits may also be possible.

5.4.2 Development length and bond

The development length of non-prestressed reinforcement and of prestressing strand is the length required to mobilize the tensile strength of the reinforcement. For prestressing strand, it consists of two components, the transfer and the flexural bond lengths. The prestress force varies linearly over the transfer length starting at zero at the end of the member. At the end of the transfer length the stress in the strand is the effective prestress. The prestress force then varies in a parabolic manner and reaches the tensile strength of the strand at the end of the development length. Usually, the two-stage stress transfer is approximated as a bilinear relationship.

The current development length provisions in LRFD Specifications for non-prestressed and prestressed reinforcement are valid for concretes up to 10 ksi. The applicability of current basic development length equations and multipliers for tension bars, compression reinforcement, bundled bars, tension splices, and hook development for HSC may need to be verified. Shorter transfer lengths may be possible with HSC. Tests show that transverse reinforcement within the development length of non-prestressed bars improves ductility (Ghosh).

5.4.3 Prestress losses

The loss of prestress force in pretensioned members is due to elastic shortening, shrinkage, creep of concrete, relaxation of steel at transfer, and relaxation of steel after transfer. The loss of prestress due to shortening depends on concrete stresses at the center of gravity of prestressing tendons due to the prestressing force at transfer, the weight of the member, the modulus of elasticity of prestressing steel, and the modulus of elasticity of the beam concrete at transfer. The loss of prestress due to the relaxation of the prestressing steel after transfer depends on the losses due to elastic shortening, shrinkage, and creep of concrete. The current provisions for prestress losses do not reflect the higher prestress levels achievable with high strength concrete.

Since HSC typically has a higher modulus of elasticity and less creep and shrinkage than NSC, it may appear that HSC would have lower prestress losses. However, this is not always the case. Higher strength concrete allows more prestressing force and thus increased member capacity. Consequently the total losses may be lower or higher depending on the level of prestressing force and other factors (See *HPC Bridge Views,* Issue No. 33).

Revised provisions for calculating prestress losses were adopted at the 2004 AASHTO Subcommittee on Bridges and Structures annual meeting. They are based on the results of NCHRP Project 19-07, NCHRP Report 496 (2005 AASHTO). If transformed section properties are used to calculate concrete stresses, losses due to elastic shortening are not calculated. However, the remaining time dependant losses still need to be calculated and gross section properties are still used. The new provisions now allow designers to include the effect of prestress gains due to dead loads applied after release. When these loads apply positive moments to the beams the bottom fibers are stretched thereby increasing the strain and stress in the strands.

5.4.4 Reinforcement limits

The limit on maximum amount of prestressed and non-prestressed reinforcement in flexural members is specified in order to ensure ductile behavior at the strength limit state such that the concrete cracks and the tension reinforcement yields resulting in large deflections before the concrete crushes. The LRFD specifications ensure this by limiting the c/d_e ratio as given in Equation 5.4.4-1. Compressive strength of concrete, amount and strength of reinforcing steel, span and depth of flexural members are interrelated and it is difficult to say what effect Equation 5.4.4-1 may have on HSC designs.

$$\frac{c}{d_e} \le 0.42 \qquad (5.4.4\text{-}1)$$

However, this limit ensures the strain in steel is at least two times the yield strain. The limit can derived from the assumed linear strain diagram and is not dependant on concrete strength.

The minimum amount of prestressed and non-prestressed reinforcement is the amount needed to develop a factored flexural resistance equal to the lesser of:

$$1.2\ M_{cr}$$
$$1.33 \text{ factored moments}$$

The cracking moment depends on the modulus of rupture of the concrete, non-composite and composite section properties, and dead load. For prestressed concrete members it also depends on the compressive stress in concrete due to effective prestress. For the designs controlled by $1.2M_{cr}$, selecting higher strength concrete typically will result in an increase in the minimum amount of reinforcement.

5.5 Shear and torsion

NCHRP research project No. 12-56, currently underway, is focused on extending the LRFD shear design provisions to concretes having strengths higher than 10 ksi. In the ACI 318 Building Code, the minimum reinforcement for shear is directly proportional to the square root of the concrete compressive strength. Thus, HSC members require higher minimum shear reinforcement.

5.6 Deformations, Deflections, and Camber

Deformations of structural concrete members can be shortening of compression members due to axial loads, shortening of prestressed members due to the prestressing force, or deflections of flexural members. Structural materials are considered to behave linearly up to the elastic limit. Stiffness properties of concrete or composite members are based on cracked and/or uncracked sections consistent with anticipated behavior. The deflections and cambers of reinforced and prestressed concrete members depend on: external loads, effective prestressing force, member stiffness, creep, and shrinkage, which can be different for NSC and for HSC.

5.7 References

1. AASHTO (2004). *AASHTO LRFD Bridge Design Specifications, 3rd Edition*, American Association of State Highway and Transportation Officials, Washington, DC, 1450 pp.

2. Tadros, M.K., Huo, X., and Ma, Z. (1999). “Structural Design of High-Performance Concrete Bridges,” *High-Performance Concrete: Research to Practice* (SP-189), American Concrete Institute, Farmington Hills, MI, pp. 9-36.

3. Stanton, J.F., Barr, P., and Eberhard, M.O. (1999). “Behavior of High-Strength High Performance Concrete Bridge Girders,” *High-Performance Concrete: Research to Practice* (SP-189), American Concrete Institute, Farmington Hills, MI, pp. 71-92.

4. Shehata, I.A.E.M., Shehata, L.C.D., and Garcia, S.L.G. (2002). “Minimum Reinforcement in High Strength Concrete Beams,” *High-Performance Concrete: Performance and Quality of Concrete Structures, Proceedings Third International Conference, PE, Brazil* (SP-207), American Concrete Institute, Farmington Hills, MI, pp. 279-295.

5. Serra, G.G., and de Campos, P.E.F. (2002). “Precast High Performance Concrete,” *High-Performance Concrete: Performance and Quality of Concrete Structures, Proceedings Third International Conference, PE, Brazil* (SP-207), American Concrete Institute, Farmington Hills, MI, pp. 327-338.

6. Rangan, B.V. (2002). “Some Australian Code Developments in the Design of Concrete Structures,” *Concrete: Material Science to Application, a Tribute to Surendra P. Shah* (SP-206), American Concrete Institute, Farmington Hills, MI, pp. 123-133.

7. Ibrahim, H.H.H., and MacGregor, J. G. (1997). “Modification of the ACI Rectangular Stress Block for High-Strength Concrete.” *ACI Structural Journal* Vol. 94, No. 1, pp. 40-48.

8. Frosch, R. J. (2001). “Flexural Crack Control in Reinforced Concrete,” *Design and Construction Practices to Mitigate Cracking* (SP-204), American Concrete Institute, Farmington Hills, MI, pp. 135-153.

9. *HPC Bridge Views,* bi-monthly newsletter published by the Federal Highway Administration and the National Concrete Bridge Council *(http://www.cement.org/bridges/br_newsletter.asp)*

10. Ghosh, S.K., Azizinamini, A., Stark, M., and Roller, J.J., "Bond Performance of Reinforcing Bars Embedded in High-Strength Concrete," ACI Structural Journal, September-October 1993

11. 2005 AASHTO: These revisions to the AASHTO LRFD Bridge Design Specifications for the 2005 interims were adopted at the 2004 AASHTO Subcommittee on Bridges and Structures annual meeting. These revisions do not become official until published by AASHTO.

SECTION 6

HIGH PERFORMANCE CONCRETE (HPC) MIX DESIGN AND PROPORTIONING

by Michael Bergin, P.E., Florida Department of Transportation;
Jon Mullarky, P.E, FHWA; Dr. Celik Ozyildirim, P.E.,
Virginia Transportation Research Council

6.1 Introduction

The intent of this section is to provide the guidelines for developing effective high performance concrete mix designs and proportioning of materials.

The topics to be covered in this section include a brief discussion of the advantages of HPC, mixture proportions (both basic and advanced concepts), selection of mixture proportions, a short discussion on specification requirements for strength and durability and a final closing statement. The critical process of material selection and the optimization of these materials will be discussed as well as the role of materials to ensure continued concrete performance.

6.2 Advantages of High Performance Concrete in Highway Bridges

6.2.1 General

High performance concrete allows for the design of taller structures, and in the case of bridges longer, shallower, lighter beams and girders. HPC gives the designer the latitude to design the project with fewer bents, girders and/or ancillary devices so that the time and cost of construction may decrease. HPC can provide increased durability, which effectively increases service life and reduces maintenance.

Shallower girder designs can replace deeper members cast with normal concrete because they maintain similar strength and deflections. In this way, HPC helps to optimize the construction process, but only after optimizing the concrete mixture for specific desirable properties.

The designers, the materials engineers and the suppliers must communicate and coordinate early on in the structural design process to make sure the specifications are reasonable and practical. Through this joint effort, the parties will mutually discover whether the specified strength and durability grades are appropriate and the materials are available locally. Time and money spent in this effort will be rewarded many folds in monetary values and job satisfaction in the long run.

6.2.2 Mixture Design

Mixture design involves identifying the fresh and hardened concrete properties required for a specific application. The FHWA has developed a model based on performance grades that will guide mix designers in developing concrete mixtures to be placed in bridge elements. These performance grades can be found in Section 3 of this HPC guideline, and are determined by the environment and restrictions that are applied to the construction site location.

The designer should take into account performance issues in addition to the normal compressive strength at a particular age. The performance criteria should identify durability issues that the concrete will be exposed during its service life.

HPC is often required to have improved mechanical properties. The mix designer may require higher compressive strength, or specific requirements on shrinkage, creep or modulus of elasticity. Exposure conditions may dictate concrete that has specific levels of resistance to sulfate attack, abrasion resistance, resistance to alkali-silica reaction or frost damage.

In order to meet this approach a clear understanding of the environmental demands must be developed. Tests to define the expected service life should be implemented in order to establish the resistance criteria needed to meet the environmental demands. If sulfate is found to be an attack mechanism then include tests for permeability and specific sulfate resistance criteria, or use sulfate resistance cements to ensure that this protection can be delivered in the final product.

Abrasion resistance on the other hand, can be addressed by including requirements for strength. In addition, identifying material requirements such as hardness of the aggregate in the specification, can also be used in the development of the concrete mixture.

It can be shown by graphical representation, that concrete produced with different sources and percentages of fly ash will be more resistant to abrasion. In this case, abrasion resistance can best be described by compressive strength. In other words, the high cementitious fraction of the mixture is a key factor in the mix to resist abrasion than the actual selection of the specific coarse aggregate.

Resistance to Alkali –Silica reaction in the concrete is best addressed by developing a low permeability concrete as well as the selection of the composition of the materials. A key here is to avoid using reactive aggregates when producing the concrete.

Freeze thaw characteristics can be controlled by including specific acceptance criteria for permeability, as well as a conservative window for the air content of the concrete.

Permeability should be the controlling requirement if the concrete will be placed in an environment subjected to concentrations of chloride ions. The designer should understand that permeability will decrease with concrete age. With that, permeability values can be established at 28 days with an understanding that by the time the element is placed into service it will be considerably lower than when tested at 28 days.

It can be shown that the permeability of plain cement concrete with no fly ash can decrease significantly up to about 60 days after placement. However no significant decrease in permeability will occur after 60 days. On the other hand, concrete containing fly ash continues to decrease in permeability for 100 to 200 days after casting.

6.3 Mixture Proportioning I

6.3.1 Basic Concepts

Mixture proportioning is both an art and a science that involves determining appropriate amounts of various ingredients to produce a mixture fulfilling design requirements economically. To proportion high performance concrete, the designer should follow some general rules. The old rules of thumb no longer apply. Concrete will now be proportioned to be durable and perform its intended function in a long service life. Concrete fundamentals and experience are needed, and the importance of optimizing the materials is essential. The interaction between specific component materials is considerably more important than the individual materials themselves. It is important to be innovative and test ideas in the trial batch process. If it makes sense then give it a try!

6.3.2 Cement and Supplementary Cementitious Materials

Typically use only Type I or II cement, try to avoid Type III unless very high early strengths are needed. Look for medium range fineness in the cement, if the cement is too fine the concrete will produce excess heat during hydration that may develop additional problems. If possible, try to utilize cements with high C2S content for long-term strength.

Understand that cements produced under the same specification, be it ASTM or AASHTO, may not perform the same. Comparatively, the different types of cements will tend to produce higher or lower compressive strengths, will set at different rates, or will be more or less sulfate resistant than others. The mix designer will need to trial batch concrete with a specific cement and then continue to use this cement for the duration of the project. An alternate cement could be used, but the trial batch process should be repeated to ensure that the concrete produced will perform its intended function.

Photo 6.3.2-1 *Cement and supplementary cementitious materials*

Understanding the performance of concrete, especially HPC, requires an understanding of the interaction of Portland cement with fly ash, slag or silica fume. Calcium hydroxide is developed as a result of the reaction of water and cement commingling and reacting. The reaction typically develops heat and initiates the hydration process. Fly ash meeting the requirements of ASTM C-618, on the other hand, may or may not contain a significant amount of calcium in its composition. Some fly ashes are rich in calcium and may initiate the hydration process with the addition of water. However, fly ashes that are low in calcium need cement to generate enough calcium hydroxide to aid in the development of strength. Usually this strength development is a long- term strength gain since the combination of fly ash and cement normally delays the setting time and thus the short- term strength development. This reaction is sometimes referred to as a “pozzolanic reaction” because it requires the production of calcium hydroxide produced as a result of hydration of the Portland cement. This reaction occurs at later times than for cement mixed only with water. In addition, the “pozzolanic reaction” will react differently based on the type and source of the cement.

Another cementitious material that is utilized during the proportioning of high performance concrete is ground granulated blast furnace slag meeting the requirements of ASTM C-989. With slag, the hydroxyl ions released as the cement initiates hydration provides the mechanism for the breaking down of the glassy slag particles into a cohesive paste and ultimately a hardened concrete. Unlike fly ash or silica fume, slag does not require calcium hydroxide to initiate hydration. When utilizing slag as a cementitious material trial batches are necessary since slag will normally require less chemical admixtures to ensure workability than mixes with silica fume or fly ash. Also, the rates of slag substitution are typically in a rage from 25 to 70 % of the cementitious material. This generates a slower rate of strength gain depending on the amount of slag proportioned into the mix. Slag provides an excellent supplementary material for mass concrete placements because overall heat is reduced as the cement content is lowered. Additionally, slag provides improved sulfate resistance, decreased permeability, and increased resistance to freeze thaw conditions.

Photo 6.3.2 -2. *Trial batching concrete mixtures.*

Silica fume is another supplementary cementitious material utilized in the batching of HPC. It is produced to meet the requirements of ASTM C-1240. It has an extremely small surface area with the ability to react with calcium hydroxide to form a very hard and impermeable concrete. Silica fume is a by-product of the silicon and ferro-silicon production industry. The particle size of the raw silica fume is smaller than cigarette smoke and as a result is only sold as a condensed or granular form and normally has a specific gravity of 2.20 to 2.30. The production of silicon and ferro-silicon materials requires extremely tight quality control measures to ensure a consistent end product. This provides a very consistent by-product in the form of silica fume that in turn provides some of the strongest and most durable concrete used in construction. However, one short coming of this material is its propensity for shrinkage cracking. This usually occurs when adequate moist curing is not provided after the finishing of the concrete surface.

The effect of different brands of cement in concrete containing fly ash, slag or silica fume cannot be fully explained on the basis of water to cementitious materials ratio. Different brands of cements achieve similar reductions in water to cementitious materials ratio, however each concrete produced with different brands of cement and different cementitious materials can also produce significantly different flexural and compressive strength which may or may not meet the design requirements. In short, the emphasis on trial batching cannot be overlooked.

6.3.3 Aggregates

Coarse aggregate is one of the most important materials in HPC. The following are some general guidelines to be considered when selecting a coarse aggregate for use in the production of HPC. These include limiting the maximum size of the aggregate to less than 1 inch, which ensures good compactability. The use of coarse aggregate with lower percent voids results in the production of high compressive strength concrete because the mixing water can be reduced and still maintain good workability

Smaller maximum size aggregates are typically needed to ensure a high mortar to aggregate bond. Smaller size aggregate also allows for closer spacing between reinforcing steel. It has been found that the use of a coarser gradation of coarse aggregate often results in the achievement of higher compressive strength concrete as a result of being able to use less mixing water while ensuring the same workability. A general guideline developed by ACI Committee 211 suggests that for concrete less than 9000 psi compressive strength, use ¾ to 1 inch maximum size aggregate. For concrete compressive strength greater than 9000 psi, use 3/8 to ½ inch size aggregate.

In normal concrete mixtures the maximum aggregate size is utilized in order to reduce the aggregate surface area, which in turn reduces the water requirement for the mix. By doing, this the cement content is increased to ensure complete coating of all aggregate particles. At some point the continued addition of cementitious materials will not increase the compressive strength. In these cases the surface area of the aggregate is reduced to ensure that the optimum cement content and aggregate size are combined to deliver the optimum concrete mixture.

Further increases in compressive strength can be realized by changing the type of aggregate in the mix. For instance, crushed aggregates are better than smooth because of the angular surfaces that are formed as a result of the crushing process. The rough angular surface forms a strong bond at the aggregate/cement paste interface. In addition, a good aggregate will usually influence the properties of the hardened concrete, especially strength, modulus of elasticity, creep and shrinkage.

Fine aggregate is also a very important part of the concrete mix which effects the workability during placement. In general, HPC can be produced by using natural or "uncrushed" rounded sand with a fineness modulus between 2.60 and 3.10. When designing for HPC consider reducing the ratio of the fine to coarse aggregate to ensure that the water demand can be reduced. In addition, the workability of the concrete is assisted by the use of high range water reducing admixtures. Always avoid the use of manufactured or crushed fine aggregate because they will typically increase the water demand of the mix.

6.3.4 Admixtures

An important issue to identify in the design of HPC is the control of the set time. Admixtures are specifically designed to help the contractor control the setting time of the mix in certain cases. These admixtures are designed to precondition the concrete and allow for greater placing time. They aid in the control of concrete by slowing the rate of hydration and should be used when the concrete temperature is expected to rise above 75 degrees F. They will normally reduce early strengths at 24 hours but after this initial delay typical strength gains occur.

High range water reducers (HRWR) should be used with good quality concrete. The purpose of the HRWR is to give the contractor additional time to place concrete and in some cases delay the set time. Its purpose is not to make quality concrete out of a poorly designed mix. HRWR should not be used if the design mix was not proportioned correctly or the correct materials were not included in the mix. A mix design targeting a 1 to 2 inch slump would normally indicate a low water to cementitious material ratio which is ideal for HPC. However, this is typically a very difficult mix to place and consolidate. If a HRWR is added at this point, the slump could be increased to 7.5 inches or more, which would allow the concrete to be placed and consolidated with less effort. Again, the drawback of adding a HRWR to the mix is the associated delay in the set time and lower early compressive strengths. Even with the addition of HRWR to the mix, compressive strengths in the 9000 to 12000 psi range are attainable using an aggregate size of 1 inch or smaller.

As a general statement, the production of HPC will require the use of HRWR to ensure workability when a low water to cementitious materials ratio is specified. In addition, the dosage rate of these admixtures will typically be higher than normal concrete mixtures and higher than the manufacturer's recommendation. If a newer generation of HRWR are utilized, (admixtures specifically designed for flowing and self-consolidating concrete) the HRWR usage rates will be defined by trial batching the concrete and identifying the plastic properties of the mixture.

Air entrainment is another admixture that is typically required in the mix to assist in the workability of the concrete. If the concrete will be exposed to cycles of freezing and thawing, air entrainment will be needed to allow the concrete to expand and contract. A typical amount of air entrainment to control expansion and contraction is approximately 3 to 4 %. However, air entrainment will typically reduce the strength of the concrete.
The typical rule of thumb is a mix with 3% air entrainment will have a 5% reduction in compressive strength. Therefore, do not include air entrainment if it is not needed for workability or to resist freeze/thaw conditions.

6.4 Mixture Proportioning - Advanced Concepts

This section will focus on the advanced concepts for mixture proportioning, optimizing materials, and quality control for high performance concrete. Field test results, and temperature effects on the plastic and hardened concrete will also be considered.

AASHTO LRFD Bridge Construction Specifications recommend the use of American Concrete Institute (ACI) 211 as the controlling document for proportioning concrete mix designs, and that the mix designs be prepared in accordance with the absolute volume method (see ACI 211.1). The initial step in this process is to determine the required strength of the concrete, f'cr, to be used in order to meet the specified design strength, f'c. These requirements are typically defined in ACI 318 and are based on experience using normal strength concrete.

ACI has not established suggested strengths for HPC when used in highway applications, but has developed some suggestions for high strength building construction concrete. The production of HPC requires that the suggested equations developed by ACI be slightly modified to include ten percent of the design strength as the allowable under design strength factor. This is compared to 500 psi which is the value for normal strength concrete production. Therefore, for HPC, ACI has suggested the following equations based on experience and research.

$$f'cr = f'c + 1.34\, s \qquad \text{where } s = \text{standard deviation}$$
$$f'cr = 0.90\, f'c + 2.33\, s$$

Additional modifications have been made to these equations as recommended by ACI Committee 211 to select the average field strength of concrete based on the laboratory strength tests. These equations have become the current general recommendation:

$$f'cr = (f'c + 1400)/\, 0.90$$

6.5 Selection of Material Proportions

6.5.1 Control of Materials

Controlling the water to cementitious materials ratio has always been a key factor in efforts to produce durable concrete. Recent information indicates that in order to produce an efficient and durable HPC the designer needs to look closely at all of the material components. As mentioned above, concrete batched with fly ash will produce a durable concrete. When the same mix is prepared with a low water to cementitious materials ratio and high range water reducer the strength and permeability of the concrete can be significantly improved. This is the general concept that will be discussed in this next section.

Photo 6.5.1-1 *- Selection of materials*

As mentioned previously, the HPC mix design approach tosses out the old rules of thumb. Although concrete fundamentals are still maintained, the focus of the design mix will be to optimize all of the materials. This is primarily because the interaction of the individual components is far more important than any one individual component material in the mix. Again, it is important to be innovative and make good use of trail batch. Consider the individual mixture components.

6.5.2 Water

Use the least amount of water possible that ensures hydration of all the cementitious materials and maintains a low water to cementitious materials ratio. The low ratio will produce a dense concrete matrix with a low permeability and more resistance to shrinkage cracking. To assist in the workability during placement, add a high range water reducer to the mix. This will increase workability and still maintain the low water to cementitious materials ratio that is desired in the mix. To select a water to cementitious materials ration, the tables in ACI 211 provide a good starting point. These tables suggest a water to cementitious materials ratio for a cubic yard of concrete based on the size of the coarse aggregate.

In addition ACI also recognizes that the amount of mixing water needs to be modified on the basis of the compactability of the voids content of the fine aggregate. The following equation is used in calculating the voids content of the fine aggregate used in concrete:

$$V = (1 - (DRUW/(BSG\ (OD) \times 62.4)) \times 100$$

Where:

	V	= voids content in percent
	DRUW	= dry rodded unit weight, pcf
	BSG (OD)	= bulk specific gravity (oven dry)

To adjust the mixing water as a function of the voids content of the fine aggregate, use the following equation:

$$\text{Mixing Water adjustment (lbs/yd}^3\text{)} = (V - 35) \times 8$$

Where: V = voids content in percent

As mentioned, optimizing the water to cementitious materials ratio is a function of the required strength, a specific test date and the use of chemical admixtures such as HRWR. to reduce water content and increase strength and durability. This is shown in the tables in ACI 211 and verifies that the selection of water to cementitious materials ratio is based on the size of the aggregate and the specific strength required at a specified test date.

6.5.3 Coarse Aggregate

Optimizing the coarse aggregate means increasing the amount of coarse aggregate with respect to the fine aggregate in the concrete mixture, which is contrary to the typical aggregate ratio in normal concrete. This is primarily because the coarse aggregate plays a major role in the development of modulus of elasticity (MOE) and control of creep and shrinkage of HPC. As shown in ACI 211 the production of HPC will focus on the maximum size of the coarse aggregate. This is the result of using a HRWR which allows the designer to effectively reduce the fines in the mixture. As such, the workability can be controlled with the admixtures. In fact, the fineness modulus of the fine aggregate can all but be ignored as an influential factor when determining the coarse aggregate content for HPC. Previously collected data indicates that the modulus of elasticity is dependant on the compressive strength of the concrete. In addition, both of these calculations are dependant on the type and amount of the coarse aggregate fraction in the mix. For instance, if two different coarse aggregates were used to produce concrete, the harder, denser rock would develop the specified modulus of elasticity at an earlier age than a softer less dense aggregate. To say this another way, it would take longer for the softer aggregate to develop the same modulus of elasticity than the harder more dense aggregate.

If the more dense aggregate yields the desired HPC, but the cost of the material is prohibitive, consider blending the harder, denser aggregate with the softer material. The combination of the aggregates could reduce the overall cost to produce the mix.

Coincidentally, strength and modulus of elasticity development could be very acceptable. This kind of experimentation is always worth a trial batch to determine the effects of combining good empirical and mechanistic engineering.

Another point worth noting is that research has identified that the modulus of elasticity is independent of the curing conditions. The primary component of the mixture that provides the most influence on the modulus of elasticity is the coarse aggregate content. ACI 318 makes a relatively accurate prediction of MOE based on the compressive strength when the coarse aggregate content of the mixture is approximately 44%.

6.5.4 Supplementary Cementitious Materials

The second most important aspect of designing HPC mixtures is the optimization of the cementitious materials. Optimizing the cementitious materials is second only to the water to cementitious materials ratio. This optimization is only achieved through the use of chemical and mineral admixtures. To verify the interaction of these materials, the designer needs to prepare trial batches to ensure material compatibility through the plastic and hardened concrete properties.

Again, ACI 211 has identified approximate percentages of cement to be replaced with fly ash, slag and even silica fume when designing concrete mixtures. Because of the unpredictable nature of batching concrete with various materials, the importance of trial batching should not be overlooked. For example, as the percentage of fly ash to replace the cement portion of a mix is increased, the concrete's compressive strength will increase to a certain point. Then, at some percentage of cement replacement the compressive strength of the concrete will decrease with subsequent increases in fly ash. It should be noted that it is not unusual that the combination of specific cements and certain fly ashes will produce continuous increases in compressive strength. Experience has shown that as the fly ash content increases the early age strength of the concrete will decrease. However, at later ages concrete with fly ash will continue to gain strength as it goes through a slower hydration process. In comparison, pure cement concretes have reached their maximum compressive strength more quickly through a faster hydration process.

Another critical aspect of optimization of cementitious materials is the use of different chemical admixtures in cement and fly ash combinations. When different cements and fly ashes are substituted within a mix design, wide variability in the concrete properties can be seen. This is a result of the interaction of such chemical admixtures as set retarders and the HRWR. The effects of these combinations can change the early compressive strength by as much as 2000 psi.

Again, the need for a trial batch is imperative to ensure that the desired properties are met for the concrete mixture before the mixture goes into production.

In addition to mix components for optimization, the following paragraphs discuss issues of durability and concrete temperature. These are the basis for using HPC and will require specific tests to evaluate the plastic and hardened properties.

6.5.4.1 Durability

Durability of concrete identifies the ability of the concrete to resist degradation due to environmental exposure conditions. There are many factors that indicate the durability of concrete, one being permeability. Permeability is the ability of a concrete to pass pore water through the hardened matrix. In reality, a durable concrete should resist this passage of pore water, or in other words, the concrete should be impermeable. This important factor is affected by water to cementitious materials ratio, mineral admixtures and high range water reducers. In cases where severe exposure of the concrete is expected, such as in splash zones, marine environments and climates where deicing salts are applied, silica fume is another mineral admixture that can be added to the mix to increase density and significantly reduce the permeability of the concrete.

Photo 6.5.4.1-1 *- AASHTO T-259 Diffusion Test to evaluate durability.*

For concrete having compressive strengths in the 6000 to 9000 psi range, permeability is more dependant on the use of mineral admixtures. For HPC with compressive strengths in the range of 9000 to 15000 psi, the use of high range water reducing admixtures results in significant reductions in permeability. However, for these concretes, permeability is still affected by the use of mineral admixtures. Water to cementitious materials ratio is not a good predictor of the permeability of concrete. Use of mineral admixtures has a more significant effect on concrete permeability than reductions in water to cementitious materials ratio. For concrete having very low water to cementitious materials ratios, permeability of the concrete seems to be more dependent on the composition of the cementitious material (cement, fly ash, silica fume, etc) than the water to cementitious materials ratio.

Photo 6.5.4.1-2 *- To evaluate durability, ASTM C-1012 is utilized.*

6.5.4.2 Temperature of Concrete

Another important consideration in the design of concrete mixtures is the temperature rise of the concrete during placement and the effects of this temperature rise on the performance of the concrete. As the temperature of concrete starts to rise, several things start to happen. Conditions such as thermal cracking, formwork removal, and the effects on strength gain are all realities of rapid temperature rise in the concrete. In many instances the results of rapid temperature increase leads to cracking. Thermal cracking occurs as a result of the exterior surface of the concrete setting more rapidly than the internal portions of the concrete. As a result the stress developed at the exterior surface(s) exceeds the stresses that the freshly placed concrete can withstand, until a crack develops.

Additionally, as the temperature of the concrete increases to above 170 degrees its compressive strength decreases. This strength reduction is typical at any test date. The result is that the concrete will reach a compressive strength and then stop gaining strength even if the concrete has pozzolanic materials that would normally allow the concrete to continue to gain strength. This phenomenon occurs even in HPC.

6.6 Specification Requirements for Strength and Durability

Specifications transmit the owner's requirements to the contractor and the concrete producer. They are extremely important in HPC because requirements are needed for both the fresh and hardened concrete properties. Part of the specification should identify the fresh concrete properties and should include air content, w/cm ratio, unit weight, age of concrete when tested and aggregate size. Concrete temperature at placement should be less than 85 degrees Fahrenheit. If the concrete is to be delivered at temperature above 85 degrees then mixes should be trial batched at the delivery temperature to demonstrate and verify properties and the workability of the concrete. Additional tests that would be considered performance tests for HPC are workability, bleeding, segregation, placeability, finishability, responses to vibration, and setting time of concrete. Although these tests give the owner a good indication of the concretes properties they are usually not included in the specification package attached to the contract documents.

Hardened concrete requires the utilization of other test methods to ensure a durable, consistent material. The tests may include but are not limited to compressive strength, modulus of rupture, modulus of elasticity, strength gain, shrinkage, and durability by freeze – thaw, sulfate attack resistance and chloride resistance. An additional parameter to be addressed is the test age for determination of compressive strength and other durability tests. Should the test be accepted at 28 or 56 days? This is a specification requirement that should be included based on cementitious materials. If fly ash is utilized in the mixture, 56 days may need to be considered as the test date for acceptance, since the fly ash will react at a much later date than normal cement mixtures or mixtures with a cement and slag.

Particularly reliable tests for HPC, especially when the concrete will be used in a prestress application, are the modulus of elasticity (MOE) and modulus of rupture (MOR). In addition, what strength is needed for releasing of the prestress forces into the element? These questions can only be addressed by trial batching the design mix and then testing the concrete. The designer needs to use experience based on current data and publications to develop a sound design, batch the mix, and test the concrete for both plastic and hardened properties.

6.7 Conclusion

The design of HPC is met when materials are optimized to produce a strong durable concrete. The water, cementitious materials, aggregates and chemical admixtures all need to be proportioned effectively to deliver the mix with the most desirable properties for placement, finishing, curing, and hardened condition. The designs are not cook book and in most cases require that the mix be trial batched to compare the fresh and hardened properties. As mentioned earlier in this section, the designer needs to be innovative with his materials and the proportioning of these materials. Once the mix has been designed and prepared, ensure that enough material is available to make additional tests for durability. Only with testing will the designer be confident that the concrete will be able to meet its intended function.

6.8 References

1. Carrasquillo, Ramon and Miller, Richard; “Mix Proportioning Parts 1 & 2: SHRP High Performance Concrete Bridge Showcase Notebook; New Hampshire DOT and FHWA; Sept. 1997.

2. Hover, Kenneth; Portland Cement Concrete Mix Design Course Notebook; FHWA 2000

SECTION 7

PRECAST/PRESTRESSED BEAM FABRICATION, TRANSPORTATION AND ERECTION

by Rich Pakhchanian, P.E., FHWA; Jerry Potter, P.E., FHWA; and
Kevin Pruski, P.E., Texas Department of Transportation

7.1 Introduction

This Section includes guidance for the successful fabrication, transportation and erection of precast/prestressed concrete beams using High Performance Concrete (HPC). Many of the basic processes will be the same for beams using HPC as for normal concrete. However, some processes will require special emphasis if the full benefit and objectives of HPC are to be achieved. HPC for beams will normally be related to higher concrete strengths rather than durability. However, both high strength and improved durability requirements may be specified for a beam and the fabrication processes should be developed to assure all requirements are met.

This document includes only provisions which are deemed significant when HPC concrete is used. The fabrication, transportation and erection provisions that are similar for both HPC and normal concrete are not included.

7.2 Fabrication

7.2.2 Plans

Contract documents for HPC projects should clearly convey the concrete requirements and components that will contain HPC. It would be desirable to especially highlight for the contractor's (fabricator's) attention, that HPC will be used to differentiate from routine fabrication.

7.2.2 Specifications

Specifications for the fabrication of precast/prestressed concrete beams vary from state to state but the basic requirements are similar. Concrete mixes and construction requirements may vary considerably depending on the states' objectives, design for durability, environment and local materials. Specific project specifications should be thoroughly reviewed and the fabricator should become familiar with all requirements before fabrication. Fabricators who work with a state routinely become very familiar with the generic provisions included in specifications but when some unusual or specific requirement such as HPC is included in the contract documents, the fabricator may overlook the requirement. High Performance Concrete may necessitate a change in the producer's normal process such as curing. This may be overlooked if the total contract documents are not reviewed.

7.2.3 Quality Control/Quality Assurance

Quality Control and Quality Assurance (QC/QA) are critical for achieving the desired performance using HPC. Quality Control is generally the responsibility of the fabricator and the Quality Assurance is the responsibility of the Owner. A successful Quality Control Program consists of a Quality Control Plan and procedures implemented by qualified production personnel and verified by quality control inspection staff. Periodic communications should exist between production and inspection personnel to discuss needed process improvements when work is not meeting expectations.

The QC/QA should be adequate to assure the final product is built to specified requirements with emphasis on efforts needed to assure the HPC properties are met. The QC/QA processes needed to assure high strengths are achieved may not be significantly different than normal production processes. However, if durability parameters are specified, special effort may be necessary to assure implementation of the procedures and processes established to achieve compliance.

Quality Control operations should be under the direction of the Quality Control Manager. Activities should be included in a Quality Control Plan developed for the specific project and be approved by the respective DOT Representative. Any special actions such as concrete placement, inspection and testing necessary to achieve compliance with HPC requirements should be included in the Quality Control Plan.

7.2.4 Personnel

Applicable Plant Personnel should have appropriate knowledge of High Performance Concrete and its characteristics. These would include the Quality Control Manager and inspectors as well as the Plant Foreman and Concrete Foreman.

Owner personnel at a precast plant should have the responsibility to assure the QC Program is executed and is producing the expected product. Owner personnel should not be involved in the day-to-day operation of the plant nor provide direction to plant personnel. Deficiencies and needed improvements in the QC Program should be directed to the QC Manager for actions. The Owners personnel should follow-up on identified deficiencies until satisfactory actions have been implemented.

7.2.5 Plant Facilities

Plant facilities should be in accordance with the following:

- Concrete mix designs should be prepared to meet the HPC requirements specified. The mix design should be verified through testing. Appropriate consideration of the fabrication process should be included in the mix design.

- Bed setup for precast prestressed beams containing HPC is not significantly different than normal production. If high strength concrete is used, the beam length may be longer and fewer beams can be produced on a bed. The normal requirements for preparing a bed to receive HPC members are applicable.

Types of beds include the self-stressing and fixed abutment concepts. Both are applicable for use with products containing HPC.

Self-stressing beds resist the prestressing force and shorten during stressing. Accordingly, the effect of the bed shortening has to be included when computing the stressing force and elongations. No specific differences exist for products using HPC except possible larger prestress forces to utilize the higher concrete strengths.

- Fixed abutments may have to be strengthened to accommodate the larger prestress forces or larger abutment rotations have to be included in the stressing calculations. No specific differences exist for members using HPC unless a larger prestress force is specified when high strength concrete is used.

- Bed support is usually provided by a substantial concrete base and steel framing. This should be adequate for beams with HPC unless deeper members are specified than normally produced on the bed. If so, some strengthening of the bed support may be necessary.

- Forms normally used for beam fabrication should be adequate for beams using HPC. If the concrete placement sequence is changed to one layer placement, the internal form pressure should be checked.

  Forms should be checked for horizontal and vertical alignment tolerances for each bed set-up. No special action is necessary for HPC unless the mix is more fluid and requires better form fit up to prevent grout leakage.

7.2.6 Draped Strands

The use of draped strands may increase with the use of HPC for beams because of the potential for increased span length and/or increased beam spacing.

7.2.7 Concrete Placement and Finishing

Paying particular attention to ambient air and concrete temperatures at the time of HPC placement can be advantageous to production and ensure good durability of the con-crete. Concrete should not be placed when the temperature of the surrounding air is expected to be below 40ºF [4ºC] within 24 hours after placement, unless a heated enclosure is provided around the beam. The temperature of the plastic concrete as placed should be above 50ºF [13ºC]. When using supplementary cementitious materials, the low placement temperature may need to be adjusted upward to initiate hydration. The concrete temperature after placement should be maintained above 55ºF [13ºC] until the prestressing steel is detensioned. The upper limit for placement of the plastic concrete should be limited to 95ºF [35ºC]. Because HPC may contain higher amounts of cementitious materials than normal concrete, it may be necessary to lower the upper placement temperature to limit the maximum hydration temperature specified in 7.2.8. High hydration temperatures can be detrimental to the concrete.

Except for self-consolidating concrete, the concrete should be placed as near as possible to the final location.. For self-consolidating concrete, the concrete should be placed in accordance with the approved procedure verified to produce the desired results. In any progressive concrete placement operation, the time between successive placements onto previously placed concrete should not exceed 20 minutes, unless the previously placed concrete has not yet stiffened, as evidenced by the continued effective use of vibration.

HPC is dependent on proper consolidation to provide durable members. Poor consolidation may result in concrete with high permeability. This situation can lead to reinforcing steel and prestressing steel corrosion, thereby negating the benefit of HPC.

When using concrete containing silica fume, the member should be finished and screeded with a continuous water fog mist maintained above the concrete. The fog should not be applied directly on the concrete. A monomolecular finishing aid may be used if approved.

7.2.8 Curing

Normal Curing

The concrete should be cured by providing adequate moisture on exposed surfaces and by maintaining the concrete temperature or curing enclosure air temperature at the concrete surface within the limits specified in this Section. Properly curing of HPC is essential to limit cracking, obtain a tight pore structure, and achieve the required strengths.

The curing should begin immediately after the finishing operation, and before the formation of plastic shrinkage cracks, to prevent damage to the surface. A fog spray or a monomolecular finishing aid should be used if needed to prevent plastic shrinkage cracks after finishing and before curing. Exposed concrete surfaces should be kept continuously wet for the duration of the specified curing period.

The following curing requirements apply to prestressed members:

- The concrete should be cured continuously except as allowed during form removal until the compressive strength of the concrete has reached the specified release strength and de-tensioning has been performed as specified.

- The concrete temperatures should be maintained between 50°F [10ºC] and 150°F [66ºC] during the curing period. The maximum allowable concrete temperature may be increased to 170°F [76ºC] if an approved mix design incorporating supplementary cementitious materials is used.

- Prestressed piling should be cured an additional 3 days after attaining the specified release strength. The curing should not be interrupted for more than 4 hours if piling is moved to storage. The concrete temperature for piling should be maintained at 50°F [10ºC] or above during this additional curing period.

The following curing requirements apply to non-prestressed members:

- The concrete should be cured continuously, except as allowed during form removal, for a period of 4 days or until the compressive strength of the concrete has reached the design strength.

- The concrete temperatures should be maintained between 50°F [10ºC] and 150°F [66ºC] during the curing period. The maximum allowable concrete temperature may be increased to 170°F [76ºC] if an approved mix design incorporating supplementary cementious materials is used.

The members should be cured immediately for an additional 24 hours if they are out of cure at any time other than during the allowable 30 minutes for form removal or during the allowable 4 hours for moving piling to storage.

<u>Accelerated Curing</u>

Accelerated curing is defined as curing with artificial heat provided to the curing enclosure or forms. Accelerated curing may be necessary to initiate or speed hydration to obtain the required release strength.

Accelerated curing facilities should be tested for a minimum of 48 hours to demonstrate that temperature variations do not exceed 20°F [11ºC] between any points in the curing enclosure. It is acceptable to perform the test on the entire casting line with either freshly cast concrete inside the forms or with empty forms. One curing enclosure air temperature probe should be provided for each 100 ft. (30 m) of casting line when accelerated curing facilities are being tested. The probe should be located at the center of gravity of I-beam sections and at the center of gravity of the thickest section for other members. Layout drawings and test results for the accelerated curing facility should be provided to the Engineer.

The air temperature in the curing enclosure should be maintained between 50°F [10ºC] and 85°F [30ºC] until initial set of the concrete has occurred and for at least 3 hours after concrete placement. After initial set of concrete has occurred, the concrete temperature may be raised uniformly at a maximum rate of 36°F [20ºC] per hour. An unobstructed air space of at least 6 in. between surfaces of the concrete and the curing jacket shall be provided.

The curing enclosure air temperature should be monitored and maintained between 50°F [10ºC] and 160°F [71ºC] after initial set of concrete has occurred for prestressed and non-prestressed concrete members. The air temperature should not exceed 160°F [71ºC] for more than 1 cumulative hour during the entire curing period. The air temperature should not exceed 170°F [76ºC] at any time during the curing period. The heat discharge into the curing enclosure should be arranged so that temperature variations do not exceed 20°F [11ºC] between any points in the curing enclosure.

Enough moisture should be provided inside the curing enclosure to keep exposed concrete surfaces continuously wet for the specified curing period.

If accelerated curing is terminated before the specified curing period has elapsed, other acceptable curing methods should be provided for the remaining curing period.

Accelerated curing must be stopped as soon as the minimum release compressive strength is obtained.

The concrete element should be cooled gradually at the maximum cooling rate of 10°F [6ºC] per hour.

Figure 7.2.8-1 *Temperature Match Curing for Prestressed Concrete Beams*

7.2.9 Detensioning

Detensioning of the prestressing force should not be allowed prior to concrete reaching the specified minimum compressive release strength.

A minimum of four compressive strength test cylinders should be made for every line of beams cast to determine when release strength is obtained. The release strength test, representing the line of beams, is the average compressive strength of two test cylinders. These cylinders are cured under the condition similar to the product or match-cured test specimens, which are match cured until the time of release.

The exposed strands between beams and at the ends of the bed should be protected from temperature drop until the bed is released. The prestressing force should be released as soon as possible after the specified concrete release strength is obtained.

In all detensioning operations, the prestressing forces should be kept nearly symmetrical about the vertical axis of the product and apply them in a manner that will minimize sudden shock or loading. The prestressing forces should be transferred to the concrete by either single strand detensioning or multiple stand detensioning. To utilize the high strengths attainable with HPC, higher (approximately 35%) than normal prestressing force is often used. It may be necessary to require multiple strand (all) detensioning to prevent cracking of the beam.

7.2.10 Defects

When using HPC for more durable concrete, it is critical to have good consolidation with a minimal number of defects. Defects can allow direct access of moisture and harmful substances to the prestressing and reinforcing steel regardless of how impermeable the concrete is.

The Engineer will determine the kind, type and extent of cracks and surface defects such as honeycombing and chipped edges or corners that will be tolerated. For members that are subjected to moderate to severe exposure conditions, it is advisable to repair all defects identified.

7.2.11 Design Strength

Acceptance of the member is based on meeting the required design strength at the specified time and meeting the dimensional and physical condition requirements as discussed above. Often with HPC, design strengths are specified at 56 days instead of the standard 28 days. The additional time allows the supplementary cementitious materials to activate.

7.2.12 Storage

This Section includes guidance on the storage processes of beams that are constructed with High Performance Concrete materials.

1. Support

Firm support should be provided at the bearing locations during storage. The storage area must have sufficient bearing capacity and stability to prevent differential settlement or twisting of the beam during the entire period of storage.

The beams should not be removed from the casting area until curing, strength, and appropriate stressing requirements have been attained.

2. Repair

Each beam should be inspected thoroughly for damage prior to shipment. If repairs to precast products are initiated in advance of the Engineer's approval, it is recommended that the affected product be considered for use only when the following conditions have been satisfied:

a. Before beginning repairs, the Contractor prepares and delivers to the Engineer a repair proposal for approval.
b. All repair materials must meet the HPC requirements used on the project and be approved by the Engineer.
c. The repairs should be made under the observation of a Quality Control Manager or inspector.

The Contractor is responsible for actions taken without approval. It is intended that repairs be made only after the proposed methods have been accepted to ensure that the proposal will not be modified or rejected.

3. Camber Check

The sweep and camber of beams should be measured and recorded monthly. The measurement records should be kept on file for review at anytime by the Engineer, and upon request, transmit a copy of these measurements to the Engineer. If the camber is exceeded by 1 inch from the design camber shown in the plans, the Contractor should determine an appropriate action subject to approval of the Engineer.

4. Reports to Field

When characteristics of the beams are outside the specified values, a report to field personnel should be prepared in order that appropriate action may be taken. For example if the camber is more than provided for in the plans, the profile may be adjusted to accommodate the excess camber. Conversely, if the camber is lower than plan value, the profile may be adjusted to reduce the haunch over the beam and save concrete volume.

5. Age

Beams should not be shipped until tests of representative concrete cylinders indicate that the concrete in the members has attained the minimum required design strength acceptable for transportation. I-shaped beams should be at least 7 days old, while bulb-T and other wide top flange sections should be at least 10 days old.

7.3 Transportation And Erection

This Section includes guidance for the transportation and handling of beams with High Performance Concrete materials.

The use of HPC may provide for longer beams than normally used and the logistics (weight, length, permits, etc) of transporting longer beams to the job site should be evaluated.

All beams are to be handled and transported only after transfer of the prestressing force. For products that are prestressed by a combination of pre-tensioning and post-tensioning; the member should not be handled before sufficient prestress has been applied to sustain all forces and bending moments that may occur during handling and transporting. Care should be used in handling to prevent damage to products.

Beams should be transported in the upright position with points of support and directions of reactions, with respect to the member, approximately the same as in its final position. Units damaged by improper handling should be replaced.

Pick up points should generally be located a maximum distance of 3 feet from the beam end, unless shown otherwise in the Contract plans. The capacity of lifting devices and handling products should be verified, taking into account various positions during handling. Multiple component lifting devices should be matched to avoid non-compatible use.

Products should be lifted and moved carefully to minimize stresses due to sudden changes in momentum. Appropriate action should be taken to increase the stability of products during handling when the factor of safety against lateral buckling instability is below 2.0. The analysis should include the expected fabrication tolerance for sweep. The analysis procedure provided by the Precast Prestressed Concrete Institute or similar procedures may be used for the stability evaluation.

When a product has multiple lifting devices, the lifting equipment should be capable of distributing the load at each device uniformly to maintain the stability of the product. Lifting devices grouped in multiples at one location should be aligned for equal lifting.

The size limitations for beams should be considered due to geometric constraints of the roadway leading to the project. In addition, the weight limitations of existing structures en route to the project should not be exceeded. Size and weight regulations vary from one state to another. Local regulations should be referenced when large beams are moved. Loads may be further restricted on some secondary roads during spring thaws.

Precast concrete members may need to be re-oriented from the transport position to its final construction position. This "tipping" (rotating) action is to be taken into account during an analysis similar to one used during handling.

Erection of beams should not proceed until the required shop drawings and construction loading and sequencing calculations have been reviewed and accepted by the Engineer. The Contractor is responsible for any cost incurred in modifying the permanent structure due to temporary loadings induced by handling and erection equipment or erection scheme. Beams should not be erected until the concrete has reached the minimum strength (f'c) specified in the contract documents or, if specified, the required minimum concrete strength (fc) at the time of erection.

Products should be adequately braced during all stages of erection to resist wind forces, weight of forms and other temporary construction loads, especially those eccentric to the vertical axis of the products. The horizontal alignment of prestressed concrete beams should not be allowed to deviate from the design alignment prior to placement of the diaphragms and deck.

Figure 7.3-1 *Long-span Prestressed Concrete Beam Erection*

7.4 Bearings

Prior to erection of the beams, it is important to set movable bearings at the appropriate temperature settings. The superstructure should have full and free movement at the movable bearings.

The bearings for structures that may be post-tensioned in the field should be capable of accommodating the movement due to elastic shortening.

7.5 References

1. Applicable State DOT Specifications for Precast/Prestressed Concrete Construction. The following state specifications are considered to be good examples for reference:

    - Texas DOT
    - Florida DOT
    - Tennessee DOT

SECTION 8

CAST-IN-PLACE CONSTRUCTION

by Donald Streeter, P.E., New York State Department of Transportation; Michael F. Praul, P.E., FHWA; and Matthew Greer, P.E., FHWA

8.1 Introduction

The Cast-In-Place Construction (CIP) section will discuss methods and recommendations regarding bridges, including substructures and decks, requiring High Performance Concrete (HPC). HPC for CIP applications requires attention to details during design and construction for this product to exhibit excellent durability characteristics and performance. For substructure applications, most CIP operations are the same as those for conventional concrete. However, additional requirements are needed for bridge decks.

In developing this Section, reference was made to *The Design and Control of Concrete Mixtures*, 13th edition by PCA, for general information regarding quality practices for Portland cement concrete. A number of topics covered herein are to highlight differences with emphasis areas within the PCA manual, including: Preparation for CIP Construction, Batching and Mixing, Handling and Placement, Finishing, and Curing.

HPC in bridge decks will usually contain some type(s) of pozzolan and require the use of certain admixtures. Cast-in-place HPC is usually a different product compared to HPC precast applications as well as traditional concrete mixes. An HPC bridge deck should exhibit less permeability and cracking overall, and thereby exhibit greater durability. In the area of precast HPC, the emphasis is generally on rapid and higher strength gain. In most instances, a higher strength bridge deck is not necessary, nor is it appropriate.

8.2 Preparation for Cast-In-Place Construction

8.2.1 Pre-Placement Meeting

While many construction activities for HPC are the same as for conventional concretes, attention to details results in quality HPC construction. Proper planning by both the contractor and the inspection force is essential before any concrete is placed. Such planning should include a job meeting to discuss, in detail, the equipment and procedures that will be employed by the contractor. A major point of discussion should be adequate delivery of concrete and sufficient placing equipment to insure that the placement can be accomplished properly. In addition, an agreement should be reached on contingency plans to handle unanticipated equipment breakdowns or interruptions in concrete supply.

The Engineer and inspectors should be completely familiar with the specifications for the work, including any special provisions, plan notes, appropriate Materials Methods, and all related information.

A Pre- placement Meeting should be required between the Contractor, subcontractors, materials supplier, and the Engineer at least one week prior to the start of any concrete placement for cast-in-place HPC construction. The Contractor and the Engineer should review all aspects of the proposed placement including, but not limited to, the following:

- Equipment proposed for use and for back-up.
- Planned workforce, assigned tasks of each designated position, and experience and expertise.
- Proposed construction techniques.
- Safety Considerations.
- Concrete mixture design.
- Admixture use and technical data.
- Proposed or required placement rate, curing plans, and loading schedule.
- Curing application plans and workforce assigned to the curing process.
- Delivery and conveyance equipment, finishing equipment information.

For late season or cold weather placements, additional information should also be discussed:

- Expected environmental conditions at time of placement and during curing
- Proposed curing methods to maintain acceptable curing temperature
- Engineers permission to proceed with cold weather concreting
- Actions required if temperature drops below specified limit

For hot weather placements, the following should be discussed:

- Expected environmental conditions at time of placement and during curing
- Proposed methods to reduce temperature effects on fresh concrete
- Response plan to protect fresh concrete in the event of equipment breakdown or delivery problems
- Engineer's permission to proceed with hot weather concreting
- Actions required if temperature rises above specified limit

Concrete should not be placed until all aspects of the proposed placement are approved by the Engineer. Modifications to the established placement plans should be submitted in writing to the Engineer for approval. The timing of this submittal should account for the complexity of the proposed change and the review procedures of, and time required by, the state agency.

Consideration for the type of placement needs to be addressed in the pre-placement meeting. Whether HPC is to be used for new construction or placed in contact with existing concrete, protection from moisture loss should be addressed. When placed against forms where moisture loss is not expected, no special considerations are necessary, except possibly the shading or otherwise cooling of steel forms prior to initiating the placement. If placing HPC in contact with any existing or previously placed concrete, it is important that the existing substrate be in a saturated surface dry condition so drying and shrinkage cracking of the HPC does not occur at the interface.

For large, multi-span continuous structural deck applications, a major consideration should be the placement sequence. Regardless of the use of phased placements of positive and negative moment areas, or a continuous placement, HPC retardation needs to be considered. It is important that all concrete remain plastic for the duration of an entire day's placement. If concrete begins to set before all loading of a deck is complete, cracking will result.

8.2.2 Mixture Design and Development

HPC mix design and development should follow the recommendations from Section 7 of this guide. Differences in precast and field applications must be considered. While precast applications will normally require higher strengths, faster strength gain, and possibly improved performance characteristics, HPC for cast-in-place applications does not usually need higher strength. These applications will typically be for decks and other members in severe environments, where durability is more important, although somewhat higher strengths can be expected.

For cast-in-place HPC, a mixture needs to be easily batched and transported. Longer mixing durations are needed in order that pozzolans, particularly silica fume, properly disperse throughout the HPC mix. The cast-in-place HPC mixture also needs to be easy to handle, place and finish.

To assure the appropriate mixture characteristics are developed, a trial batch and handling simulation are highly recommended. This should include a trial batch produced, transported, and discharged in the same manner as expected in the field. If, for example, a haul time of 35 minutes is expected, then the trial batch should be agitated for 35 minutes before evaluating the concrete or performing a trial placement. This will help ensure that similar HPC characteristics are achieved on the day of placement. Trial placements should be performed using the same materials, equipment, and labor as planned for the actual placement. This will provide laborers with a better understanding of HPC characteristics and how HPC can be expected to perform.

8.2.3 Placement preparation

In general, placement preparation for HPC is very similar to that of conventional concrete: forming evaluations need to address set times and strength gain rates; reinforcing materials and placement procedures will be the same; handling and placement procedures will differ only to accommodate different mixture characteristics; and curing materials and operations are similar.

For bridge decks, a dry run of the finishing equipment should be performed to assure proper operation. This should include not only set up of the machine, but checking that mechanical functions are operating properly. Vibration should be checked during the dry run to assure it is operational, however, actual inspection must be done with a reed tachometer when the equipment is actually finishing concrete. The dry run should also address concrete cover, machine clearances with surrounding obstructions, and operation practices.

For overlay applications, existing surface preparation and expansion joints must be addressed. Joints used for thin overlays should be saw cut at the surface for a clean appearance. The depth of cut should not be more than 1 inch (25mm). The remaining joint face should be a chipped surface, at approximately a 45-degree angle into the area of placement; this detail has proven successful over many years of use by the NYSDOT. All existing concrete which bonds to new HPC needs to be prepared to a saturated surface dry condition. To achieve this, a minimum of 12 hours of continuous wetting is recommended, with any excess, standing water, removed with oil free compressed air. This should be done immediately prior to concrete placement. The surface should not be left exposed to the environment for extended periods of time and allowed to dry prior to HPC placement.

8.3 Batching and Mixing

There are many similarities between HPC mixes and conventional concrete mixes regarding batching and mixing. The primary differences usually occur with HPC mixes including various pozzolans, the need for certain admixtures, and the use of higher quality materials. An HPC mixture containing silica fume for reduced permeability and/or higher strength will usually require a water reducer(s), and also additional mixing. HPC bridge deck mixtures do require several material and admixture considerations.

The batching process for HPC is generally similar to that of conventional concrete. However, performance of an HPC mixture may be affected by the batching operations, the order of batching materials, and the mixing equipment and procedures. If a new HPC mixture will be utilized, it is advisable for the Materials or Project Engineer to meet with the ready mix company. This should be accomplished as part of the pre-placement meeting. Also, it may be beneficial for the Engineer to become familiar with the batching facility and to review the batching operations. Many of the issues involved with batching can be addressed through the trial batching process and a pre-placement meeting.

All materials should be from approved sources. Actual mixture designs should follow the guidance from Section 6 of this Guide. Consideration must be given to the intended application of HPC in establishing required mix characteristics. Admixture use and dosages will need to be determined from trial batching. Although some HPC mixes utilize high range water reducers to achieve a desired higher than normal slump (6-8 inches), such mixes are generally not desirable for bridge deck placements. Normal and mid-range water reducers have instead been shown to provide suitable workability for bridge deck placements. As noted previously for bridge decks, it is important that the concrete remain plastic for the duration of the placement. In most cases, a set retarding and water-reducing admixture is needed in HPC deck concrete. All admixtures and pozzolans must be compatible and provide consistent results, as verified through the trial mix process.

HPC mixtures using silica fume and admixtures will require sufficient mixing at the batching facility to achieve homogeneity. For a central mix plant a full 90-second mix cycle is generally necessary. For facilities using truck mixers, 100 mixing revolutions at the batching facility before the truck leaves for the project is recommended. Once on the project site, and before any testing of the HPC mixture, an additional 30 mixing revolutions should take place to reactivate the admixtures. After this mixing is complete then it is appropriate to test the concrete for air and slump. An additional 70 mixing revolutions may be ordered if needed for the addition of admixtures on site. A combined total of 200 mixing revolutions should not be exceeded.

Driver operations, including wash down after batching and mixing can have a significant impact on HPC. The mixing speed for truck mixed HPC may need to be slowed to provide better mixing. HPC mixtures can be “sticky” as a result of the particulars of a given mix. This “stickiness” makes the mix adhere more readily to metals and can result in the concrete adhering to the drum of the mixer. Also, the maximum load size may need to be reduced to approximately 70% of the mixing capacity of the truck, i.e. a 10-yard truck would be permitted to haul 7 cubic yards. Slowing the mixing speed and reducing load size will allow the HPC to more efficiently mix.

Inspectors should look for a uniform mixture discharged from the truck. If this is not the case, alterations in the batching and mixing process must be considered. Trial batching will usually address issues affecting the HPC handling and workability characteristics resulting from the batching and mixing process. HPC with poor workability or handling properties should not be used since finishing operations will become more difficult and the performance of the HPC may be reduced.

8.4 Handling and Placement

Generally, conveyance and vibration of HPC is very similar to ordinary Portland cement concrete. Good concreting practices should be called for in the specifications and adhered to by the contractor.

8.4.1 Conveyance

Concrete should be deposited as close to its final position as possible. There are several methods of conveying the concrete from the truck to its proper location in the forms. These include concrete buckets, buggies, conveyor belts, pumping, or simply from the chute of the truck. With any of these methods, the concrete should not be allowed to free-fall more than two feet nor should it be allowed to segregate by striking the reinforcing steel. Good practice calls for the concrete to be discharged against previously placed concrete already in the forms.

Figure 8.4.1-1 *- Concrete being discharged into previously placed concrete to avoid segregation and damage to the epoxy coating.*

Many HPC mixes lend themselves quite readily to pumping. Where pumps are to be used, an appropriate, flowable HPC mixture should be supplied along with the grout required to prime the pump. It is advantageous to coordinate concrete delivery when using a pump to expedite placement. In many cases when pumps are used, two concrete trucks can be discharged simultaneously and the discharge should be as horizontal as possible to avoid segregation and promote proper air retention. Constant communication between the plant and the jobsite is a must in order to effectively coordinate delivery and to deal with unforeseen delays or difficulties. With HPC, these delays can be even more critical for bridge deck placements. Concrete left exposed for excessive periods of time could result in plastic shrinkage cracking. While the use of set retarders will be beneficial in keeping the concrete workable for additional time, they will not reduce the water loss that comes from fresh concrete being exposed to the air, a particularly critical item for flatwork placements.

When conveyor belts are used, transfer points between conveyors should be equipped with discharge hoods to prevent segregation. Also, the transfer time should not exceed 15 minutes. For HPC deck placements, this transfer time is even more significant as the concrete will be exposed and losing water prior to being placed on the deck. This makes timely placement and curing even more important.

8.4.2 Consolidation and Vibration

Consolidation is the process of compacting fresh concrete to assure a uniform product and to remove the entrapped air. The most common process for consolidation is by using vibrators.

Once the concrete is placed it should be vibrated thoroughly so that it completely surrounds the reinforcing steel and fills any voids. Due to the materials used in HPC mixtures, the use of internal vibration, i.e. spud vibrators, is of paramount importance. While many HPC mixtures are resistant to segregation, care should be taken not to move the concrete with the vibrators and not to over-vibrate the concrete. Over-vibration may cause segregation or remove the desirable entrained air. Good practice for HPC is the same as for ordinary cement concrete; more detailed discussions are available in a wide range of references.

8.5 Finishing

Like any conventional concrete, HPC finishing needs to be performed properly to assure durability. There are few differences in the finishing operations of conventional concrete and HPC however those few differences can have significant results. The most noteworthy of these differences is the need for comparatively rapid placement and finishing of the concrete accompanied by timely commencement of curing operations. Following good concrete construction practices is essential to obtaining the benefits in service that HPC can provide.

For substructure applications, proper consolidation is needed to achieve the desired finish, to avoid significant bug holes (surface voids), on formed faces. Since HPC mixtures contain pozzolans, these mixtures tend to be "stickier" and may need more vibration. The exposed surfaces require only minimal hand finishing. Too much finishing could result in scaling or freeze-thaw damage.

For superstructure applications, regardless of new construction or overlays, finishing is similar to that used with conventional concretes. The same finishing machines can be used as long as they are set up and operated according to manufacturer's recommendations, and checked for proper operation. Use of reed tachometers to assure proper vibration is necessary. Finishing operations should follow good concreting practices. HPC must be protected from evaporation and the environment by covering with wet burlap or plastic if any delays in finishing operations occur. The impact of any delays is dependent on the weather conditions during placement and its relationship to rate of water loss from the fresh concrete; a delay on a day with low temperature, high humidity, and low wind would be less critical than a delay on a day with high temperature, low humidity, and high wind. No excess water or "blessing" (spraying the surface to improve workability) should occur. The stickier consistency of HPC often leads finishers to add water to the surface to help close-up the finish. This will result in poor performance of the surface.

Finishing operations need to be completed in a timely manner. It is preferable to place HPC during periods of low evaporation. Regardless, HPC needs to be deposited, finished and cured in no more than 30 minutes. Field personnel should understand that the use of a set retarder will maintain workability in the concrete, it will not protect the concrete from water loss and the resulting onset of plastic shrinkage cracks. HPC placement should not progress further than 5 to 8 feet ahead of the finishing machine. If concrete is placed beyond this then drying or setting of the material can occur and it becomes difficult to finish the surface without excessive floating. Once the finishing machine has passed, hand-work must progress within 5 to 8 feet behind the machine to complete operations before the surface begins to dry.

Minimal hand finishing should be performed. For decks, the need to completely close up the surface is not necessary since a texture will be applied for friction/safety concerns. Excessive effort to completely finish the surface could result in poor freeze/thaw performance, increased scaling potential, and a delay in placement of curing. Where hand work is necessary, it should be kept to a minimum to assure HPC durability. Overworking the surface will cause aggregate to sink into the mix and paste to rise.

Figure 8.5-1 - *Scaling along the curbline. This is due to excessive hand-finishing and over-working the concrete prior to curing.*

Figure 8.5-2 - *A more severe example of scaling*

Application of any desired texturing to the HPC surface needs to be completed immediately behind any floating operation. Delay in applying texture to plastic concrete will result in tearing of the surface, with an excessively roughened finish.

Immediately after texturing, curing needs to be applied to prevent shrinkage cracking. The minor imprint of curing materials is less important than delayed application of curing. To facilitate deck drainage and reduce the likelihood of vehicles hydroplaning on the deck once it is in service, many agencies have called for tining the fresh concrete to provide grooves for the water to run off the deck. With HPC, the use of saw-cut grooves is a better choice. Tining requires the concrete to harden to the point where the groove that is cut into the fresh concrete will not collapse on itself. The need for timely placement of wet curing for HPC, dictates that the concrete be covered before it will be able to be properly grooved with a tining rake. Saw cutting can be done at a future time, allowing the concrete to be cured immediately, and provides better, more consistent grooves.

8.6 Curing

Durability of concrete is dependent on many things: materials; batching; handling; placing; finishing, and curing. Curing is the protection provided to new concrete to assure the desirable characteristics of the concrete are maximized. Proper curing provides an environment for the concrete; this means keeping the concrete at the proper temperature and moisture conditions to maximize the hydration of the cementitious materials.

Thorough hydration provides many enhancements to concrete properties including improved strength gain, reduced permeability, improved freeze-thaw resistance, and reduced plastic shrinkage cracking. Yet, knowing all this, curing is often treated as a secondary operation. As noted in the Introduction to this Section, the success of HPC is dependent on increased attention to detail. Nowhere is this more evident than with curing. As HPC technology and the increased use of supplementary cementitious materials moves forward, proper specifications for curing HPC, followed by improved curing practices during construction, must be implemented. The techniques used to provide proper curing for HPC are the same as those used for ordinary PCC. The real difference comes from the increased diligence on the part of the contractor and inspection personnel in assuring that curing specifications are adhered to during and after HPC placement. The importance of timely and proper curing cannot be overemphasized.

8.6.1 Curing Environment

Hydration of cement requires proper temperature and moisture conditions. It is commonly accepted that the best curing method for concrete is continuous wet curing. When curing is interrupted, hydration will eventually stop as the internal relative humidity of the concrete drops below about 80%. The resulting concrete properties include lower-strength, freeze thaw resistance and increased permeability. Re-establishing the proper curing environment can reactivate the hydration process to some extent, however, it is very difficult to re-saturate the concrete. The hydration process is even more critical with HPC, given the likelihood that the mix contains supplementary cementitious materials which rely on completion of the hydration reaction and the continued presence of moisture to fully develop the desired properties of the concrete. Concrete in which the hydration process has stopped will never achieve the same properties it would have if proper curing was maintained as specified. Methods for providing a moist curing environment are discussed in section 8.6.2..

Concrete that cures at the proper temperature will have superior performance properties when compared to concrete cured at very high or very low temperatures. At high temperatures, HPC behaves similarly to ordinary PCC. The consequences of curing at too high a temperature include reduction in ultimate strength and the increased chance of shrinkage cracking. Ways to mitigate the effects of high temperatures include the use of retarding admixtures, cooling the constituent materials in the concrete, and curing with continuous wetting. Methods used for HPC are the same as for ordinary PCC and further discussion of hot-weather concreting issues is available in many concrete reference manuals.

Curing HPC in cold weather may present some difficulties above and beyond those experienced with ordinary PCC. Due to the presence of supplementary cementitious materials, including pozzolans, concrete set may be delayed and strength gain can be significantly reduced in cold weather. Depending upon the particular element being placed, this can cause problems for initiation of curing. Some agencies have reported delays as long as several days for HPC to set in very cold weather. If project schedule dictates HPC placement in very cold weather, all parties should be made aware of the possible consequences and ways to mitigate them should be investigated. Many of the ways that cold weather is mitigated for PCC also work for HPC. These include the use of insulating forms, heated blankets, heated enclosures, and accelerating admixtures.

8.6.2 Methods of Curing

The duration of curing and the method applied are dependent on the concrete being cured. Vertical applications are treated differently than horizontal. Consideration of the element being cured is always necessary when planning the curing. The goal is to cure as effectively as possible and for the greatest duration possible. Applications vary but concrete is generally cured for 7 or more days. The more common methods of curing include:

- Continuous wet curing
    - Ponding
    - Sprinklers/soaker hoses
    - Fogging (initial curing)
- Curing covers
    - Plastics and papers
    - Forms
- Curing compounds

Combinations of the above methods are often used for the desired or specified duration. Ponding of concrete is very desirable for flatwork that does not have any grade to it and where dams can be erected around the perimeter. Few concrete placements are conducive to ponding so the use of sprinklers or soaker hoses, usually in conjunction with blankets or burlap, provide for continuous wetting that can be easily established. This process requires a continuous supply of water and runoff may need to be controlled.

When continuous wetting is not viable, curing covers are often used. Covers may consist of plastic sheeting, impervious paper, or plastic coated fiber blankets. These materials prevent moisture from escaping. Care in applying these materials is necessary since they can cause damage to the surface of plastic concrete. In most cases it is more important to commence curing than to be concerned with any aesthetic imperfections on the concrete surface. Covers need to be protected from displacement to remain effective during the curing period. Overlapping the edges of covers and anchoring in place is necessary. Forms left in place are effectively curing covers for vertical applications. Forms act as insulation and care is needed in hot conditions to prevent thermal stresses from developing. Loosening forms and providing wet curing between the forms and the fresh concrete is beneficial. Care needs to be exercised to ensure the concrete has set sufficiently prior to loosening the forms.

Liquid membrane curing compounds provide another means of curing. These products are effective when properly applied. It is imperative that curing compounds be applied immediately after finishing and before the concrete surface dries. Application rates, generally of 3.5 m^2 per liter, must be maintained for acceptable curing. Often application rates are not maintained or material is not uniformly applied in one coat and a second coating, applied at right angles to the first coat is necessary. A major limitation on the use of curing compounds is that if a subsequent placement will occur, the cure applied surface will need to be cleaned because curing compounds act as bond breakers.

8.6.3 Special Considerations for Bridge Decks

Due to their very nature, bridge decks present the most challenges when it comes to properly curing HPC. HPC decks invariably contain a supplementary cementitious material, they are often placed in hot or cold weather, and they are prone to cracking for a multitude of reasons including the large amount of fresh concrete area exposed to heat, direct sun, wind, and potential delays during placement. These factors, in combination, demand the best curing practices in order to get the most benefit from the concrete.

The single most important action thing that can be taken for bridge deck curing is to place wet burlap over the concrete as soon as possible after placement and finishing. This should take place no later than 30 minutes after the concrete is discharged from the truck and no later than 10 minutes after finishing. Many field personnel are concerned about leaving impressions in the fresh concrete. While these can be minimized with proper burlap placement techniques, the benefits of this approach to curing far outweigh the presence of any surface blemishes. Concrete that has curing initiated in this fashion is far less likely to develop plastic shrinkage cracking and is more likely to fully hydrate and perform better in service.

Figure 8.6.3-1 *Note the proximity of the burlap to the finishing machine*

The curing process for bridge decks begins as soon as the concrete is placed. Many agencies have made use of the ACI evaporation rate chart to guide placement schedules and/or to trigger specific actions to mitigate the loss of water from the deck surface. These mitigating measures often include requirements for fogging and/or the use of wind screens. The purpose of fogging is to retain high humidity levels over the fresh concrete in order to avoid water loss until the wet burlap can be placed. Fogging should not be perceived as a “safety factor” that allows for delaying the placement of burlap. Fogging may be impractical on windy days.

***Figure 8.6.3-2**- A good example of fogging. Note the mist above the deck, maintaining high humidity above the fresh concrete. Also note that no moisture is accumulating on the fresh concrete.*

While specifications may call for the use of wind screens, they are often not practical for bridge decks. A proper wind screen takes some planning and cannot be placed at the last minute. In fact, a poorly designed or installed wind screen can actually worsen the situation by increasing the wind vortices at the deck level above the concrete. Contractors will not want to incur the expense of installing a wind screen that may not be necessary. For these reasons, wind screens should generally not be relied upon for bridge decks unless the deck is in a perpetually windy location in which case the contract documents should require use of a windscreen and include some details of an acceptable installation.

Another unique challenge presented by bridge decks is cold weather curing. Depending on the temperature, the use of insulating blankets may be enough to retain the heat necessary for a proper curing temperature. For very cold weather, the contractor may need to build a heated enclosure. Such structures can be a costly, but necessary, addition to a project. If an enclosure is called for, the entire deck should be contained, not just the top. A great deal of heat can be lost from the bottom if it is not contained within the heated enclosure. It is important to provide a uniform temperature for all the concrete as it cures. Neither HPC nor ordinary PCC should be exposed to freezing temperatures. Once concrete has frozen, it can never reach its intended level of performance. This is particularly significant with an HPC bridge deck being the most critical element in the bridge.

In concluding this section on bridge decks, one rule of thumb is offered as a guide: The best thing for curing bridge decks is to start the cure as soon as possible, and keep them as wet as possible for as long as possible. Several States require wet curing of all HPC bridge decks for 14 days.

8.7 Conclusion

Many of the practices and procedures used for HPC vary little from ordinary Portland cement concrete. Concrete is a durable and forgiving material, but in order to fully achieve the benefits of HPC, use of appropriate materials and adherence to proper construction practices must be improved.

Innovations are constantly being made to improve concrete curing. Some innovations are in the curing materials, such as the use of cotton mats and improved curing compounds or combinations of curing and sealing products. Other innovations are related to the concrete itself. The use of absorptive aggregates to provide internal curing is progressing. Also, the use of high early strength mixtures and maturity methods are increasing, resulting in the need for shortened cure durations.

Regardless of the curing methods used or the innovations that are progressing, the importance of curing must be recognized. Applying what is known to be important in a timely manner can ensure the best, most durable concrete possible.

8.8 References

1. *ACI Manual of Concrete Practices, Part 4*

2. *Design and Control of Concrete Mixes,* 13th edition, by Portland Cement Association

3. State Department of Transportation Construction and Materials Specifications –

* Colorado, contained in Section 601, under Class H and Class HT –
http://www.dot.state.co.us/Bridge/ProjectSpecials/Project_Specials.htm

*Florida DOT 346 Concrete —
http://www.dot.state.fl.us/specificationsoffice/2004BK/D346.doc.pdf

*Maine –
http://www.maine.gov/mdot/contractor-consultant-information/ss_division_500.pdf

*New York, contained in Section 501 for materials requirements and Section 557 for placement operations –
www.dot.state.ny.us/specs/2002specbook.html

SECTION 9

BRIDGE INSTRUMENTATION

by Myint Lwin, P.E., FHWA and Joey Hartmann, P.E., FHWA

9.1 Introduction

The purpose of this section is to provide general information on bridge instrumentation for measuring strain, deflection, rotation, acceleration and temperature.

The most common instruments used to monitor structural behavior are capable of measuring strain, load, deflection, rotation, acceleration and temperature. These devices have been successfully used for field instrumentation of segmental concrete bridges to study short- and long-term performance. Properly selected and strategically located instruments will collect valuable data that can be used to determine:

- Material behavior, e.g. concrete creep and shrinkage
- Sectional behavior, e.g. neutral axis location
- Component behavior, e.g. deflections and rotations
- Systematic behavior, e.g. reactions, translations, vibration frequencies
- Environmental loading, e.g. thermal gradient, wind speed, ground motion

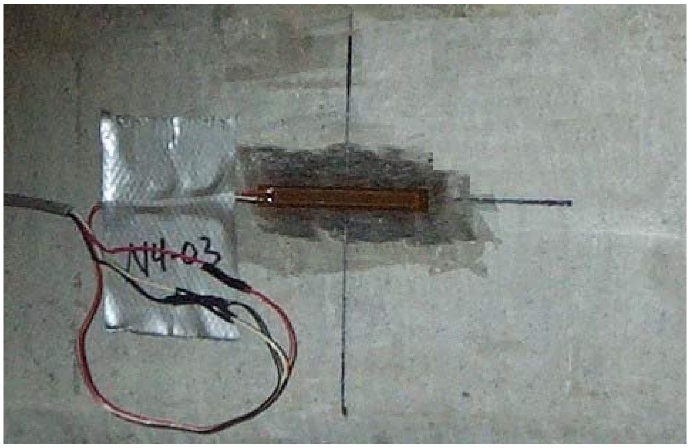


Figure 9.1-1 *Strain Gage*

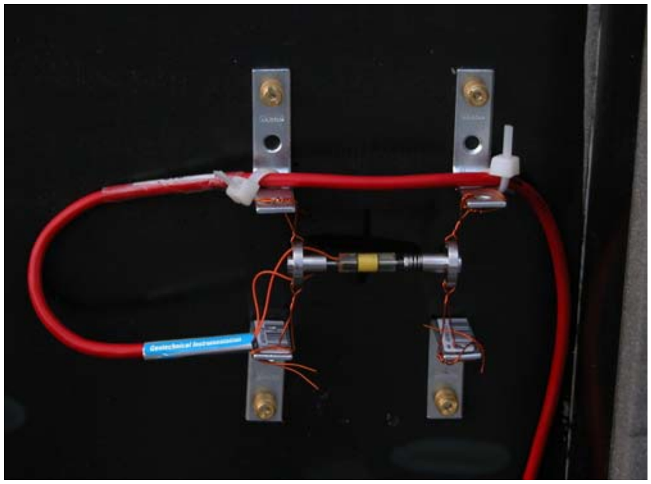


Figure 9.1-2 *Vibrating Wire Strain Gage*

Bridge engineers use the collected data to:
- Verify design assumptions and parameters
- Make modifications to the bridge
- Improve on future designs and specifications
- Assess the general health of specific bridges
- Provide information for bridge management systems

9.2 Instrumentation Program

An instrumentation program is best planned and developed concurrently with the design process. The development of the instrumentation program should be the joint effort of the designers, researchers (universities), instrument suppliers and contractors involved on the project and should consider the following factors:

- Parameters to be measured
- Type of measurement needed, static or dynamic
- Accuracy needed for measurements to have value
- How and from what location the instrument will be interrogated (manually or automatically, onsite or remotely)
- Environmental conditions instrument will operate in
- Period of time instrument will be needed
- Budget

To date, experimental researchers have generally been in the best position to lead in the development of an instrumentation program, provide resources for collecting and analyzing the data for the duration desired, and to prepare interim and final reports.

9.3 Examples

9.3.1 The North Halawa Valley Viaduct

An example of an effective instrumentation program on a concrete bridge is the North Halawa Valley Viaduct located on Oahu, Hawaii.

The North Halawa Valley Viaduct is better known as the Interstate Route H-3 Project. It is a 2-km (6,500 ft.) long prestressed concrete box girder bridge. It was constructed by the cast-in-place cantilever segmental method using an overhead erection gantry. The project consists of two parallel viaducts, one carrying two lanes of traffic inbound to Honolulu and one carrying two lanes of outbound traffic from Honolulu. The inbound viaduct is 1897 m (6,225 ft) long and the outbound viaduct is 1667 m (5,470 ft) long. Both viaducts are on horizontal alignment with a minimum radius of approximately 1800 m (5,906 ft.). The spans vary from 61.5 m (200 ft) to 109.7 m (360 ft) in length.

The instrumentation program, started in 1995, was sponsored by the Hawaii Department of Transportation and the Federal Highway Administration, and is being conducted by the University of Hawaii at Manoa. The primary objective of the program is to monitor the creep and shrinkage strains of this structure in order to produce sufficient data to improve the predictor models for this bridge type employed by existing analysis and design softwares.

To effectively determine the creep and shrinkage effects on this structure, seven sections of the inbound viaduct were instrumented to provide an adequate representation of the overall structure's behavior. The information gathered at each section includes concrete strain, tendon force, concrete temperature gradient, ambient temperature, and relative deflection, rotation and translation. The instrumentation program was designed by the University of Hawaii at Manoa and the Construction Technology Laboratories (CTL) of Skokie, IL. In order to achieve the project goals, the instrumentation teams concluded that the necessary measurements would be made statically and include the following devices: strain gages, load cells, thermocouples, tiltmeters, extensometers and a base-line level referencing system. The budget also permitted use of an automated data acquisition system, which was necessary given the remote location and inaccessibility of the viaducts.

Data will be collected automatically onsite and analyzed by researchers from University of Hawaii at Manao for a period of 10 years.

9.4 References

1. Russell, Henry, "Implementation Program on High Performance Cocnrete – Guidelines for Instrumentation of Bridges", Report No. FHWA-SA-96-075.

2. Shushkewich, Kenneth; Vo, Nhan T; and Robertson, Ian, "Instrumentation of the North Halawa Valley Viaduct, Oahu, Hawaii", funded by Hawaii Department of Transportation and Federal Highway Administration, September 1998.

3. Arrellaga, J.A., "Instrumentation Systems for Post-Tensioned Segmental Box Girder Bridges", Masters Thesis, The University of Texas at Austin, December 1991.

4. Main, J.A. and Jones, N.P., "Full-scale measurements of stay cable vibration.", *Proc., 10th Int. Conf. on Wind Engrg.*, Balkema, Rotterdam, The Netherlands, 1999.

5. Roberts-Wollman, C.L., Breen, J.E., and Cawrse, J., "Measurements of Thermal Gradients and their Effects on Segmental Concrete Bridge", ASCE Journal of Bridge Engineering-- May/June 2002 Volume 7, Issue 3.

6. Chajes, M.J., Mertz, D.R., Edberg, W., and Schulz, J.L., "Bridge Evaluation Through Experimental Field Testing," *Proceedings of the Structural Stability Research Council's 1995 Annual Technical Session*, Kansas City, Missouri, 1995.

SECTION 10

COSTS

by Myint Lwin, P.E., FHWA

10.1 Introduction

The purpose of this section is to provide cost information and methods for assessing the cost-effectiveness of high performance concrete (HPC) in highway bridges. The section covers cost estimating for the initial construction cost, and life-cycle cost analysis (LCCA). Additional information on LCCA may be found in Appendix B.

10.2 Types of Cost Estimates

10.2.1 Project Cost Estimate

Cost estimates are prepared during the planning and development of a bridge project to determine the relative costs of various bridge types and to determine the probable cost of the project. They help in making a decision to proceed with the project and in selecting the most cost-effective type of bridge for the project in meeting the needs of the owner. Once the bridge type has been selected, the Engineer refines these cost estimates as the project progresses and until the construction plans and specifications are completed.

These cost estimates are approximate estimates. They are either generally based on historical cost data kept in a design office or are generated by researching current costs of the planned bid items. In the preliminary planning stage, the Engineer reduces a bridge to square feet of deck area and then multiplies the square feet by the estimated cost per square foot for the type of structure under consideration. The Engineer may improve the cost estimate by using the unit square foot cost for the superstructure and a different unit square foot cost for the substructure, depending on the type of foundations being proposed. When plans and quantities are available, the Engineer will use unit prices to improve the cost estimate for the bridge.

Following are some examples of national average square foot cost ranges (2002) of concrete bridges based on deck area:

Prestressed Concrete Girder Bridges - 80 to 140 feet spans

Water Crossing with Pile Footings:	\$55 - \$100 /SF
Water Crossing with Spread Footings:	\$50 - \$90 /SF
Dry Crossing with Pile Footings:	\$50 - \$90 /SF
Dry Crossing with Spread Footings:	\$45 - \$80 /SF

Post-tensioned Concrete Box Girder Bridge - 100 to 200 feet spans

Water Crossing with Pile Footings:	\$70 - \$120 /SF
Water Crossing with Spread Footings:	\$70 - \$110 /SF
Dry Crossing with Pile Footings:	\$70 - \$110 /SF
Dry Crossing with Spread Footings:	\$60 - \$100 /SF

Following are some examples of national average unit price ranges (2002) for preparing the Engineer's estimate based on quantities:

Steel reinforcing bars	$0.35 - $0.50 /lb.
Epoxy coated steel reinforcing bars	$0.45 - $0.70 /lb.
Concrete for bridge deck	$350 - $500 /CY
Concrete for substructure	$250 - $350 /CY
PS Concrete Girder Spanning 60' to 100'	$100 - $115 /LF
PS Concrete Girder Spanning 100' to 150'	$110 - $135 /LF

10.2.2 Contract Cost Estimate

Contractors prepare a detailed cost estimate to include the costs of materials, construction equipment, labor, overhead, and profit. The contractor uses the detailed cost estimate for preparing bids. However, the actual competitive bids submitted for a contract may not represent the true detailed estimates computed by the contractor. A contractor uses experience and judgment to adjust the estimates before submitting the final bids. This is done to optimize the possibility of winning the contract and making a reasonable profit. This is one of the reasons for the variations of bid costs tendered by the prospective bidders.

Following is an example showing the differences between the low bid (May 1996) and the Engineer's estimate for the SR18 over SR516 HPC bridge project in King County, Washington. It is a three-span continuous prestressed concrete bridge. The center span has a length of 137 feet and the end spans are 80 feet each. The roadway deck is 40 feet out to out.

Bid Analysis

Item	Quantity	Engineer's Est.		Low Bid	
		Unit	Total	Unit	Total
HPC items only:		Cost	Cost	Cost	Cost
P/S Conc. Girder W-74G	1,430 LF	$115	$164,450	$153	$218,790
Test Girder	1	L.S.	$ 9,000	L.S.	$ 9,000
Conc. Class 4000D Deck	1	L.S.	$271,000	L.S.	$250,000
Total Project Cost			$696,079		$737,540
Square Foot Cost					
Superstructure			$27.06/SF		$24.64/SF
Total Bridge			$58.59/SF		$62.08/SF

Note: The higher bid in the prestressed girder is attributable to the instrumentation for the demonstration project and unbalancing of some bid items. The contractor has to coordinate with the researchers to install and protect the instruments during fabrication, transportation and erection. Without instrumentation, the HPC girder would have cost only $4.00 per linear foot more than the non-HPC girders.

Following is an example on the contractor's cost breakdown for a project with 770 cubic yards of cast-in-place concrete walls:

Concrete Cost:

Normal Mix (5.5 sack)

5.0 sack	$55.00
0.50 sack	$ 2.30
Total	$57.30 x 770 CY = $44,121

HPC Mix (5.0 sack, fly ash, mid-range plasticizer)

5.0 sack	$55.00
80 lbs. Fly ash	$ 2.00
Mid-range plasticizer	$ 3.76
Total	$60.76 x 770 CY= $46,785

Pump Cost:

Normal Mix

30 CY/hour @ 4" slump
770 CY/30 CY per hour = 25.7 hours
25.7 hours x $95.00/hour = $2,442
770 CY x $2.00/CY pump cost = $1,540
8 pours x $50/pour mobilization = $400
Total = $ 4,382

HPC Mix

34.5 CY/hour at 5" to 7" slump
770 CY/34.5 CY per hour = 22.3 hours
22.3 hours x $95.00/hour = $2,119
770 CY x $2.00/CY pump cost = $1,540
8 pours x $50/pour mobilization = $400
Total = $ 4,059

Placing Labor Cost:

Normal Mix

Concrete placed at a 4" slump will require 3 persons
(1 on the hose and 2 on the vibrators)
3 persons x $40/hour = $120/hour
$120/hour x 25.7 hours = $3,084

HPC Mix

Concrete placed at 5" to 7" slump will require 2 persons
(1 on the hose and 1 on the vibrator)
2 persons x $40/hour = $80/hour
$80/hour x 22.3 hours = $1,784

Sacking Cost (Using 20,738 sq. ft. finished wall):

Normal Mix

20,738 sq. ft. x $1.00/sq. ft. = $20,738

HPC Mix

20,738 sq. ft. x $0.50/sq. ft. = $10,369

Summary:

	Normal Mix	HPC Mix
Concrete Cost	$44,121	$46,785
Pump Cost	$ 4,382	$ 4,059
Placing Labor	$ 3,084	$ 1,784
Sacking	$20,738	$10,369
Total	$72,325	$62,997

Savings = $9,328 = $ 12.11/CY

Notes: There are savings in pump cost, placing labor and sacking. HPC mix is more pumpable, requires less time and labor to place, and costs less to sack as there are fewer bug holes and visual defects. The unit costs are based on January 2000 prices.

Here is another example illustrating the cost between normal concrete and HPC concrete in a prestressed concrete plant based on January 2000 prices:

Ingredients	Normal Mix Cost/Cu. Yd.		HPC Mix Cost/Cu. Yd.
Fine Sand	$ 3.08	Blended	$ 3.40
Coarse Sand	$ 1.65	Sand	
Coarse Aggregate	$ 4.99		$ 5.61
Cement	$33.28		$32.21
Fly Ash	-----		$ 4.56
Silica Fume	-----		$15.00
Superplasticizer	$ 4.85		$ 9.92
Water Reducer	$ 0.54		$ 0.54
Operating Expenses	$ 7.14		$ 7.14
Total	$55.53/CY		$78.38/CY

Difference in cost = $22.85/CY

10.3 Life-Cycle Cost

After the constructed project is put into service, the owner of the facility continues to incur costs for maintenance, repair and rehabilitation throughout the service life of the facility.

A bridge represents a significant long-term, multi-year investment. After the bridge is opened to traffic, it requires periodic maintenance, repair and rehabilitation actions to assure continued uninterrupted service and safety. A cost effective bridge design must consider the initial cost for construction and the long-term costs for maintenance, repair and rehabilitation. It is no longer sufficient to just consider the first costs in making investment decisions on bridge projects. Life-cycle cost analysis (LCCA) must be made to help managers and engineers make better investment decisions. LCCA can be used to assess the cost effectiveness of HPC and other new construction materials. However, there is a lack of historical bridge cost data for LCCA.

Recognizing that historical cost data is not readily available, transportation agencies and bridge owners are beginning to collect construction, maintenance, repair and rehabilitation data for performing LCCA. The former HPC Lead State Team developed a condition state report form for the collection of cost data for LCCA. An example of a Condition Report as prepared by a bridge inspector of Washington State Department of Transportation (WSDOT) is shown below:

HIGH PERFORMANCE CONCRETE CONDITION REPORT

Bridge No. *18/25S* Location *7.2 mi. E. Jt. SR164*
Bridge Name *SR516 Overcrossing* Intersection *SR 516*
Structure Type *PCB* Signatures *Jason Kikuta/MyintLwin*
Structure ID No. *0014919B* Date *10/7/97*

Elem No.	HPC Element Description	Quantity	Units	Env.	State 1	State 2	State 3	State 4	Bid Price
115	*P/S Concrete Stringer*	*1430*	*LF*	*3*	*1430*				*$153/LF*
367	*High Performance Concrete*	*1*	*Each*	*3*	*1*				

Remarks:

115, 367 - High performance concrete used in stringers. A few diagonal hairline cracks in webs of stringers 1A,
1C, 1D, 3A and 3E near both abutments. A couple vertical hairline cracks in webs of stringers 1A and 3E near mid-
Span. All cracks open < 0.015".

Note 1: The element number and element description are based on the Pontis Bridge Management System. Each bridge element has an assigned element number and condition description. The total element quantity is broken down into condition states that best describes the state of the element being inspected. In addition, a smart flag element number is assigned to the High Performance Concrete. This is used to identify bridges with High Performance Concrete Elements. The bid price at the time of construction is entered in the unit price category.

Note 2: Attention all designers, inspectors and supervisors - The WSDOT is a leading State in a High Performance Concrete (HPC) program. The WSDOT's role is to track cost and performance of HPC components, including bridge condition, repair, rehabilitation and maintenance data. The objective is to provide data on the deterioration of HPC under service conditions. Deterioration, if any, should be noted on the HPC Condition Report form. Damage due to impact or conditions not resulting from deterioration of the concrete should not be listed in this form.

When deterioration has occurred and repair is needed, the cost of repair will need to be documented. Place a note in the "Repair List" instructing the Region Maintenance Personnel that the repair cost should be submitted when a repair confirmation is forwarded. The designers are advised to designate the HPC members on the construction plans as "HPC" in a way similar to the designation of fracture critical members, FCM.

Note 3: The data in the Condition Report can be input into PONTIS, BRIDGIT, or other systems. For states which have not adopted any bridge management systems at this time, the data can be used in the future.

Different methods and procedures have been used to perform LCCA. Currently, there is no commonly accepted methodology for performing LCCA. Computer programs are available to help conduct LCCA. Some methods and computer programs for LCCA are discussed in Appendix C Life-Cycle Cost Analysis.

10.4 Initial and Long-Term Cost Savings

The structural and durability properties of HPC offer opportunities for innovative solutions to achieve initial and long-term cost savings. The structural properties, such as early high strength, allow the designers to meet site specific design requirements cost effectively. The durability properties, such as freeze-thaw resistance, abrasion resistance, low permeability, lead to durable structures and lower maintenance needs.

Initial cost savings may be accomplished in many ways. The three most common applications of HPC to save initial construction cost are:

(1) Use fewer beams or lines of girders. All states using HPC are able to take advantage of this benefit to lower the initial costs. For example, using HPC in the South Orient Railroad Overpass in San Angelo, Texas, the designers were able to reduce the number of lines of beams from 7 to 4 for a span length of 131 feet and from 8 to 5 for a span of 140 feet. Using HPC in the Route 140 Bridge, Bristol, New Hampshire, the designers reduced the number of girders from 7 to 5 for the 60-foot single span bridge.

(2) Use longer beams or girders. Many states are taking advantage of the high strength of HPC to design beams or girders to expand the applicability of prestressed girders to longer span bridges. Longer girders help solve horizontal clearance problem or accommodate late minute design changes. For example, using HPC in the Type III AASHTO Girders, North Carolina DOT was able to use the Type III AASHTO Girders of the original design for a longer span rather than changing the design to Type IV AASHTO girders. The longer span resulted from a last minute raising of the grade. A last minute change in bridge design would have caused delay in construction in addition to more money in design. Longer spans also mean few piers for the substructure. This will alleviate environmental concerns and result in significant construction cost savings for river crossings. For example, using HPC in an adjacent box beam bridge located on US Route 22, Cambridge, Ohio, the designers changed a conventional three-span bridge of 38’ 6” each) to a single, monolithic span bridge of 115’ 6” c/c of bearings, eliminating two piers in the water. Another example, using HPC in the Twisp River Bridge, Twisp, Washington State, the designers are able to span across the Twisp River without any pier in the river by using 200-foot girder lengths. Nebraska DOT is working on precast prestressed girders to span up to 300 feet.

(3) Use shallower members. Some projects encounter vertical clearance problems in crossing over an existing roadway, path, railroad or the 100-year flood. Raising the grade can be an expensive preposition. Using shallower members, the designers can solve the problems speedily and inexpensively. For example, using HPC in the I25/Yale, Denver, replacement project, Colorado DOT was able to use shallow adjacent box beams to maintain existing roadway geometry while providing the required vertical clearance to the existing roadway below.

In many cases, more than one of the above beneficial features of HPC can be mobilized to meet design requirements and save costs. For example, in the rehabilitation or widening of existing bridges, HPC may be used advantageously to lengthen the spans, to eliminate some existing piers, to reduce the weight of superstructure by using less materials, or by eliminating lines girders, and to keep the new girders at the same or shallower depth as the existing girders.

Long-term cost savings come from the durability characteristics of HPC - freeze-thaw resistance, low permeability, high abrasion resistance and scaling resistance. Research efforts and experimental projects have confirmed the durability of HPC and the positive impact on longer service life and lower maintenance cost. However, the use of HPC in highway bridges has not gained wide acceptance until recently. Currently, over 17 states are using HPC for bridge projects. Many of these bridges are instrumented to collect long-term performance data to help with estimating life-cycle cost. The former AASHTO/SHRP Lead State Team for HPC Implementation developed guidelines on condition, maintenance and repair data for use independently or in conjunction with bridge management systems in tracking life-cycle cost of HPC bridges. Much effort has been devoted to collect long-term performance data of HPC and to develop computer software to use the performance data for performing life-cycle cost analysis. In due course, tools will be available to quantify the long-term cost savings of HPC.

10.5 Preliminary Design and Cost Estimate

Preliminary design is an important step in the design process for quality and economy. The purpose of the preliminary design is to select the proper type, size and location of a bridge. In addition to the functional requirements, the main factors outlined in the previous chapter must be fully considered. Any items that have cost impact in the design and construction should be addressed at this time. For example, environmental issues, permit requirements, foundation information, material availability, etc. should be resolved or have an understanding of the solutions. Once the size and location of the bridge are decided, the designers will study feasible alternatives that will suite the bridge site and do cost comparisons for the alternatives. With all facts considered, the designers will select the best type of bridge for the site.

The type, size and location (TS&L) of the proposed bridge are generally documented in a TS&L Report. The TS&L Report will give a description of the project, the alternatives considered, the advantages and disadvantages, the cost associated with each alternative, and justification for the selected design. The TS&L Report may be as simple as a one- to two-page document with a set of preliminary plans. The TS&L may be as comprehensive as necessary to comply with FHWA policy for major or unusual bridges.

Following is an example of alternative study done by Colorado DOT during preliminary design in 1998:

> The existing Yale/I25 Bridge in Denver carried 6 lanes of traffic with a roadway width of 110'. It was a four-span bridge with a total length of 214' 5". It carried over 150,000 vehicles a day. Because of the deteriorated condition of the existing bridge, it was necessary to replace the bridge. The replacement bridge would use two spans instead of the existing four. It was necessary to improve the vertical clearance over Yale Avenue without changing the existing vertical alignments. Construction must be phased and accelerated to minimize impact to traffic. The designers had a handful of challenges in hand.

The designers carried out three preliminary designs: structural steel, cast-in-place post-tensioned concrete box girder and precast, prestressed side-by-side concrete box girder. The cost estimates for the preliminary designs are as follows:

Structure Type	Cost
C.I.P. Post-Tensioned Box Girder	$1,810,000
Rolled Steel I-Girder	$2,370,000
Precast Pretensioned Side-by-Side Box Girder	$1,850,000

The designers selected the high strength precast concrete side-by-side box girder as the optimum solution for the site. Although this option was slightly higher in cost than the C.I.P. alternative, it was selected to eliminate the need for falsework. The use of falsework would cause unacceptable interruption of traffic on the roadway below.

The preliminary design stage is a good time to start communication with local suppliers fabricators and contractors. They can provide some invaluable ideas for improving constructability and reducing cost.

One idea that is often overlooked is the consideration for future widening. When bridges are constructed in urban or suburban areas, it is very probable that these bridges will need to be widened to accommodate more lanes. Money will be saved if the designers make provisions to facilitate future widening.

10.6 Final Plans, Specifications, and Cost Estimates

Once the TS&L Report is accepted by the approving authority, the designers can proceed to final design. During preparation of the final plans and specifications, the designers need to continue dialogue with the suppliers, fabricators, and contractors. The specifications must be consistent with the plans. For HPC, the specifications must address the construction issues outlined in the previous chapter.

The cost estimates can be refined by using the quantities taken from the final plans and the unit prices from the current database. The cost data given in Section 5 will be very helpful in finding the appropriate unit prices.

10.7 Cost Data from State HPC Projects

One of the goals of the current HPC Technology Delivery Team is to collect and disseminate the latest information on HPC research, design, construction and cost. The following Table shows the strength and cost data on HPC from the States which responded to a survey questionnaire sent out by the former AASHTO/SHRP Lead State Team in 1997.

***Table 10.7-1** Cost Data on HPC Bridges*

State	Bridge			Deck		Beam		Substructure		Square Foot Cost	
	Length (Feet)	Main Span (Ft.)	Width (Ft.)	Strength psi.	Cost $/Unit	Strength psi.	Cost $/Unit	Strength psi.	Cost $/Unit	Superstr $/SF	Total $/SF
AL	>600	81-120	33-40								
DE	81-120	81-120	49-56		10/sf		35/sf		630/cy	64	197
FL										26	49
GA					427/cy		88-140/ft		--		49
KY	301-400	81-120	49-56		363/cy						
NE	181-240	80	>77		511/cy		720/cy				67
NH.	80	80	57-68		545/cy		255/lf			59	
NY	80-600	80-180	26-77		21/sf						
NC											
SC											
TN											39
TX					9/sf		115/lf		413/cy	15-32	24-47
VA											50
WA	241-300	121-180	33-40			10,000	153/lf			26	62
WI											

HPC costs and benefits have been difficult to assess, due to the different methods States use to bid their structural concrete items. As a specific example, the Georgia DOT provided cost comparisons for their first HPC bridge, noting a savings over conventional concrete of $137 per cubic yard for HPC deck concrete, a savings of almost $4 per linear foot for Type II HPC beams, and an increase of $37 per foot for a Type IV HPC beam. On a unit cost basis, the HPC bridge saved $2.19 per square foot.

Since January 2000, the Ohio DOT has bid 65 projects with a warranty clause and 7-year maintenance bond on bridge decks. Prior to instituting the warranty provision, HPC deck concrete was being bid at $514 to $521 per cubic yard. The first HPC deck project with warranty provisions was bid at $553, and have dropped to $514 since that time.

The Virginia DOT has noted a drop in statewide square foot costs for Federal-aid bridges from $58 for conventional concrete to $50 for HPC bridges. The in-place costs of HPC in Vermont is approximately $150 to $250 per cubic yard higher than conventional concrete, however, the State feels the increased cost is justified because of the better product that results.

10.8 References

1. SHRP High Performance Concrete Bridge Showcase, Houston, Texas, March 25-27, 1996.

2. SHRP High Performance Concrete Regional Bridge Showcase, Omaha, Nebraska, November 18-20, 1996.

3. SHRP High Performance Concrete Bridge Mini-Showcase, Atlanta, Georgia, March 1997

4. SHRP High Performance Concrete Bridge Showcase, Richmond, Virginia, June 24-26, 1997.

5. SHRP High Performance Concrete Bridge Showcase, Bellevue, Washington, August 18-20, 1997.

6. SHRP High Performance Concrete Bridge Showcase, Waterville Valley, New Hampshire, September 22-23, 1997.

7. SHRP High Performance Concrete Bridge Showcase, Denver, Colorado, February 18-20, 1998.

8. SHRP High Performance Concrete Bridge Showcase, Cincinnati, Ohio, February 23-24, 1999.

9. SHRP High Performance Concrete Regional Bridge Showcase, Auburn, Alabama, June 29-July1, 1999.

10. Goodspeed, C.H., Vanikar, S., and Cook, R.A., "High Performance Concrete Defined for Highway Structures", Concrete International, V. 18, No. 2, February 1996.

11. Jobse, H.J., "Application of High Strength Concrete for Highway Bridges", Report No. FHWA/RD-87/079, Federal Highway Administration, Washington, D.C. October 1987.

12. Russell, H.G., Volz, J.S., and Bruce, R.N., "Optimized Sections for High-Strength Concrete Bridge Girders", FHWA-RD-95-180, Federal Highway Administration, Washington, D.C., 1995.

SECTION 11

CASE STUDIES & LESSONS LEARNED

by Claude Napier, P.E., FHWA; Edward Parker, FHWA and Madhwesh Raghavendrachar, P.E., Caltrans

11.1 Introduction

Information about the HPC bridges is located in numerous published and unpublished technical reports, papers in technical journals, symposia proceedings and student theses. The purpose of this chapter is to present brief case study summaries on HPC bridges that have been built, to demonstrate the practical applications of HPC, and to provide available references at the end of each summary for more detailed information if desired.

11.2 Outline for Case Studies

The following is the outline for the Summaries.

- Description: Identifies if the bridge is open to traffic and provides a summary of the bridge features, including location, bridge length, span lengths, girder spacing, HPC elements, the type environment.
- Benefits: Highlights why HPC was used.
- Costs: Provides the available cost (total cost, cost per square foot, or cost per foot) of the structure.
- Admixtures specified: Includes items that were required by the HPC special provisions.
- Admixtures used: Identifies the actual admixtures used by the contractor to meet the requirements of the specifications.
- HPC Requirements: Identifies such HPC parameters as permeability, design strength (greater than 8 ksi), high early strength, SCC/placement, lightweight aggregate, modulus of elasticity, and others.
- Lessons Learned (when available): Builds on the benefits of using HPC and highlights any problems experienced and how solved.
- References: Provides references or web links for additional information.

The initial case study summaries are for the 18 bridges included in the national program initiated in 1993 by FHWA to implement the use of HPC. The program included the construction of demonstration bridges in each of the former FHWA regions and dissemination of the technology and results at showcase workshops. The 18 bridges were located in 13 States. Other States have implemented the use of HPC in various bridge elements. Case study summaries will be completed on the best innovations, with the objective of trying to have at least one case study summary for every state that implements HPC.

11.3 Lessons Learned from HPC Projects

A number of lessons have been learned from the HPC projects. Research and FHWA demonstration HPC projects have affirmed that HPC is constructable and can be used cost effectively in highway bridge construction. HPC has enhanced durability characteristics and strength parameters not normally attainable by using conventional design mixes.

The use of HPC in bridges has shown that HPC saves initial construction cost and long-term maintenance cost by:

- Reducing weight and using fewer lines of beams.
- Using shallow beams to solve vertical clearance problem.
- Increasing span lengths.
- Increasing short- and long-term durability.

An increasingly large number of manufacturers and contractors are involved in HPC production, fabrication, and construction. They have learned valuable lessons for improving productivity and quality and achieving a consistently good product. Precast prestressed concrete beams using concrete strength of 10,000 psi and release strength in the range of 7,000 to 8,000 psi can be cast on a normal daily cycle with only slightly higher costs over conventional concrete. For example, HPC concrete typically costs around 20% more than conventional concrete per cubic yard, . In a number of instances, HPC concrete has cost the same as conventional concrete because the fly ash and slag being used cost the same or less than the cement being replaced.

A good quality product saves cost in the long run. The steps taken by the owner and the manufacturer to assure success and quality in production are vital to cost savings and profitability. Trial mix designs have proved to be invaluable in developing the project construction specifications, and in helping the manufacturers minimize surprises and delay in production.

For the first time users of HPC, a test section constructed prior to the start of fabrication has proved to be very beneficial. The lessons learned from the test section can be put into practice to assure quality and production.

HPC possesses all the essential elements for structural applications to extend service life and reduce life-cycle cost. The successful application depends greatly on the designers working closely with the local fabricators and contractors to arrive at cost-effective solutions.

11.4 Project Listing

The following are the case study summaries developed to date:

Table 11.4-1 Case Study Summaries

Alabama	Highway 199 over Uphapee Creek, Macon County
California	New San Francisco – Oakland Bay Bridge: Skyway
Colorado	Interstate 25 over Yale Avenue, Denver
Georgia	State Route 920 (Jonesboro Road) over I-75
Louisiana	LA 87 over Charenton Canal in St. Mary Parish
Nebraska	120th Street and Giles Road Bridge, Sarpy County
New Hampshire	Route 104 over Newfound River, Bristol Route 3A over Newfound River, Bristol
New Mexico	Old Route 66 over Rio Puerco
North Carolina	U.S. 401 over Neuse River, Wake County
Ohio	U.S. route 22 over Crooked River at Mile Post 6.57 near Cambridge in Guernsey County
South Dakota	I-29 Northbound over Railroad in Minnehana County, Structure No. 50-181-155 I-29 Northbound over Railroad in Minnehana County, Structure No. 50-180-155
Tennessee	Hickman Road over State Route 840, Dickson County Porter Road over State Route 840, Dickson
Texas	Louetta Road Overpass, SH 249, Houston US Route 67 over North Concho River, US Route 87, and South Orient Railroad, San Angelo
Virginia	Route 40 over Falling River, Brookneal in Lynchburg District Virginia Avenue over Clinch River, Richlands
Washington	Eastbound lanes of State Route 18 over State Route 516 in King County

11.4.1 Alabama
Highway 199 over Uphapee Creek, Macon County

Opened to traffic in April 2000, Highway 199 over Uphapee Creek is a 798 ft long bridge. It consists of seven 114 ft spans with 54 in bulb tee girders spaced at 10 ft. The environment is considered a normal bridge over water. The HPC elements are the substructures, girders, and deck. The bulb tee girders cost \$120/linear ft, the substructure cost \$24.72/ft^2 of deck surface area, and the superstructure cost \$16.93/ft^2 of deck surface area.

Design of the girders for a concrete compressive strength of 10000 psi at 28 days allowed the use of five lines of girders. Six lines would have been required with conventional strength concrete. Eliminating a line of girders resulted in an estimated cost savings of \$100,000. Furthermore, the use of HPC resulted in one less pier at an estimated savings of \$100,000. The anticipated benefits with the use of HPC are twofold. One is a savings on initial construction costs from the use of one less girder line and one less pier. The second is the anticipation of a more durable concrete structure contributing to less maintenance costs and a longer service life.

For the girders, the owner allowed the use of Class C fly ash at less than 35% and silica fume at less than 15% of cementitious material weight. For the deck, Class C fly ash at less than 30% and silica fume at less than 15% of cementitious material weight was specified. Chloride permeability thresholds were not required. The deck was required to be wet cured for seven days.

For the girders, the contractor used 753 lb/yd^3 of cement and 133 lb/yd^3 of fly ash. At 28 days, compressive strength quality control tests on the girders averaged 9920 psi. For the decks, compressive strength quality control tests averaged 7370 psi at 28 days using 658 lb/yd^3 of cement and 165 lb/yd^3 of fly ash.

For more detailed information, click <here> to view files from the HPC Bridges – HPC Data Compilation or click the link below to read an HPC Views article on the project:

http://www.cement.org/pdf_files/hpc-9mayjun00.pdf

11.4.2 California

Oakland Bay bridge: Skyway, San Francisco

The new San Francisco-Oakland bay bridge is comprised of the Signature span, the Skyway spans, and the approach spans. This bridge, currently under construction, is located in a marine environment over the San Francisco Bay on Interstate 80. The design life of this bridge is 150 years.

The Skyway is a multi-span precast segmental box-girder with a typical span length of 540 feet. The box girder is comprised of two “main” girders spaced approximately at 26 ft. While all the components of the new bay bridge have stringent performance requirements, this synopsis addresses the superstructure elements of the Skyway. HPC, combined with the segmental construction, resulted in a significant increase in design span lengths.

The strength used in designing the precast girders of the Skyway was 8000 psi. However, the owners required a minimum modulus of 5,200,000 psi. In addition, a shrinkage limit of 0.045 % at 180 days per ASTM C 157 (7 days water cure) and a creep limit of 0.5 mils/psi @ 20% - 40% f'_c (28 days) were also specified. The use of admixtures (flyash/ silica fume/ metakaolin) was also required.

The contractors used 25% fly ash, shrinkage reducing admixtures, retarder (Type B) and high range water reducer (Type G). Concrete strengths in excess of 10,000 psi were achieved.

I-25 over Yale Avenue, Denver

Opened to traffic in June 1998, I-25 over Yale Avenue is a 214 ft long bridge with a 38 ft wide roadway. The two span structure utilizes 1700 mm (67 in) wide by 750 mm (29.5 in) deep box girders spaced at 1720 mm (67.7 in). The environment is considered a normal bridge over a road. The HPC elements are the piers, girders, and deck.

High performance concrete was specified for the box girders to meet the requirements for long spans with a shallow superstructure depth. To provide additional room for a turning lane, the bridge was reduced from an original four span design to two spans. The new bridge has one pier with four columns as compared with the previous bridge that had three piers and a total of 45 columns. The reduction in the number of spans and columns subsequently improved aesthetics and sight distances. The girders cost $188.29 per linear ft.

The owner specified a maximum fly ash percentage of 25% for the girders and 10% for the deck. Compressive strengths were specified as 10,000 psi at 56 days for the girders and 5076 psi at 28 days for the deck. There were no chloride permeability requirements specified. Dependent upon the time of the year, membrane curing followed by five days of water curing or membrane curing followed by five days of insulating blankets was utilized for the deck.

For the girders, the contractor used 730 lb/yd^3 of cement and 35lb/yd^3 of silica fume and obtained compressive strengths ranging from 7800 to 14000 psi at 56 days. For the decks, the contractor used 712 lb/ft^3 of cement and no fly ash or silica fume and obtained a compressive strength averaging 5945 psi at 56 days.

For more detailed information, click <here> to view files from the HPC Bridges – HPC Data Compilation or click the link below to read an HPC Views article on the project:

http://www.cement.org/pdf_files/hpc-3mayjun99.pdf

11.4.4 Georgia
SR 920 (Jonesboro Road) over I-75, Henry County

Opened to traffic in February 2003, the SR 920 over I-75 bridge is 352 ft long and 90 ft wide. The structure has a four span arrangement of 53 ft, 127 ft, 127 ft, and 45 ft. The 127 ft spans use AASHTO Type IV girders spaced at 7.6 ft. The shorter spans used AASHTO Type II girders also spaced at 7.6 ft. The environment is considered a normal bridge over water. The HPC elements are the girders and deck.

The use of HPC was essential to the project because it allowed the Georgia Department of Transportation to use 127 ft long AASHTO Type IV beams. This minimized the overall depth of the superstructure and avoided raising the grade of the bridge and requiring the purchase of additional right-of-way. HPC was also used to optimize beam spacing. Conventional strength concrete girders would have required the use of five additional girders. The lower permeability of HPC is expected to result in a more durable structure with a longer service life and minimal maintenance. The total cost of the bridge was \$58.77/ft^2 of deck surface area.

For the girders and deck, the owner allowed the use of Class F fly ash ranging from 0-15% and silica fume ranging from 5-10%. The chloride permeability requirements were less than 3000 coulombs at 56 days for the girders and less than 2000 coulombs at 56 days for the deck. The deck was required to be continually moist cured for seven days utilizing burlap-polyethylene cover and soaker hoses.

For the girders, the contractor used 810 lb/yd^3 of cement, 75 lb/yd^3 of silica fume, and 100.6 lb/yd^3 of Class F fly ash. The mixture resulted in a chloride permeability less than 500 coulombs at 56 days. For the deck, the concrete used 617 lb/yd^3 of cement, 26 lb/yd^3 of silica fume, and 104 lb/yd^3 of Class F fly ash. However, because of mix design changes made by the contractor in the field, a much higher than specified chloride permeability of 3963 coulombs at 56 days was obtained.

Lessons Learned: In conjunction with the Georgia's HPC project, research was conducted by Georgia Tech to evaluate the time-dependent behavior of HPC in prestressed girders. Two grades of HPC were investigated for more than a year to determine their time-dependent mechanical properties and structural responses. A large number of compressive strength, modulus of elasticity, creep, and shrinkage tests were conducted. The results unveiled an inaccuracy of creep and shrinkage predictions when modeling HPC. A better model, which more favorably compares to tested data, is being proposed. Time-dependent prestress loss and camber were also monitored on four 33 ft 4.4 in long AASHTO Type II girders and six 9 in X 18 in X 14 ft beams. The results indicate that the current AASHTO LRFD Specifications and the PCI Design Handbook significantly overestimates the time-dependent prestress loss and deflection of HPC precast beams. The research proposes modifications to the current specification to account for the characteristics of HPC.

For more detailed information, click on <here> to view files from the HPC Bridges – HPC Data Compilation or click the link below to read an HPC Views article on the project: http://www.cement.org/pdf_files/hpc-28julaug03.pdf

11.4.5 Louisiana

LA 87 over Charenton Canal in St. Mary Parish

Opened to traffic in November 1999, LA 87 over Charenton Canal is a 365 ft long and 46 ft 10 in wide bridge. There are five-71 ft spans of AASHTO Type III girders spaced at 10 ft. The environment is considered a normal bridge over water. The HPC elements are the pile, pile caps, girders, deck, approach slabs, and barrier rails. The Type III Girders cost $82/linear ft, the 30-in piles cost $105/linear ft, and the class AA (HPC) concrete cost $540/yd^3.

Design of the girders with a concrete compressive strength of 9000 psi allowed the use of five lines of girders. Six lines would have been required with a routine concrete compressive strength of 6000 psi. The use of high-strength concrete in the piles increased their resistance to compressive and tensile driving stresses and allowed the casting and shipping of longer lengths. Because of the added durability of the HPC members, the Louisiana DOT expects a 75 to 100-year service life for the bridge instead of the normal 50-year service life for concrete structures.

For the girders, the owner allowed the use fly ash ($\leq$ 35%) and silica fume ($\leq$ 10%). For the deck, fly ash ($\leq$30%), silica fume ($\leq$10%), and ground granulated blast furnace slag (GGBFS) ($\leq$50%) was allowed. The chloride permeability required for the girders and the deck was 2000 coulombs or less based on AASHTO T 277 at 56 days. The deck was required to be cured 7 days wet under burlap.

For the girders and piles, the contractor used 691 lb/yd^3 of cement and 296 lb/yd^3 of fly ash and obtained a chloride permeability of around 1100 coulombs. For the pile caps and decks, permeability of around 1020 coulombs was obtained using 306 lb/yd^3 of cement and 305 lb/yd^3 of GGBFS.

Lessons Learned: *The scope of the project included a literature search, a survey of regional fabrication plants, a study of mix designs in the laboratory and in the field, fabrication and testing of full-size concrete specimens, and analysis of the test results. The test program included flexural tests of three 24-in (610-mm)-square concrete piles, flexural and shear tests of three 54-in (1.37-m)-deep prestressed concrete bulb-tee girders, field driving of a 130-ft (39.6-m)-long prestressed concrete pile, and fatigue testing of a 54-in (1.37-m) deep prestressed concrete bulb-tee girder. The report concluded that the provisions of the AASHTO Standard Specifications for Highway Bridges are conservatively applicable to members with concrete compressive strengths up to 10,000 psi (69 MPa).*

For more detailed information, click <here> to view files from the HPC Bridges – HPC Data Compilation or click the link below to read an HPC Views article on the project:

http://www.cement.org/pdf_files/hpc-8marapr00.pdf

11.5.6 Nebraska
120^{th} Street and Giles Road Bridge, Sarpy County

Opened to traffic in July 1996, 120^{th} Street and Giles Road Bridge is a 225 ft long bridge. It consists of three 75 ft spans, each with NU1000 girders spaced at 12.4 ft. The environment is considered a normal bridge over water. The HPC elements are the girders and deck. The prestressed concrete girders cost $741/yd^3 and the cast-in-place concrete deck cost $313/yd^3.

Design of the girders required a 56-day concrete compressive strength of 12,000 psi. Deck strength criteria included a concrete compressive strength of 8,000 psi and a rapid chloride permeability of less than 1800 coulombs at 56 days. This resulted in seven lines of girders compared to eleven that would have been required with conventional strength concrete girders. Building the HPC bridge less than a half mile away from an identical conventional concrete bridge allowed Nebraska Department of Roads to evaluate service life and establish incremental costs for design and construction of the HPC bridge in relation to a conventional bridge.

For the girders, the contractor used 750 lb/yd^3 of cement, 200 lb/yd^3 of Class C fly ash, and 50 lb/yd^3 of silica fume. At 56 days, compressive strength quality control tests on the girders averaged 13,944 psi. For the deck, the contractor used 750 lb/yd^3 of cement and 75 lb/yd^3 of Class C fly ash. Compressive strength quality control tests averaged 10,433 psi at 56 days. The rapid chloride permeability measurement for the deck was 589 coulombs.

Lessons Learned: Several cracks were observed on the bottom surface of the HPC deck. These cracks were generally perpendicular to the axis of the girders and were well distributed over the entire surface. The orientation, distribution and narrow widths of these cracks indicate that they are not caused by structural overstress, rather that they are shrinkage cracks.

The cracks may have occurred due to the following reasons:

- Strict curing requirements were applied to the top surface of the deck, but the bottom surface received no special attention. Therefore, the bottom surface of the deck was deprived of moisture needed for curing.

- The high temperature variation between the top surface and the bottom surface of the deck.

For more detailed information, click <here> to view files from the HPC Bridges – HPC Data Compilation or click the link below to read an HPC Views article on the project:

http://www.cement.org/pdf_files/hpc-3mayjun99.pdf

11.4.7a New Hampshire
Route 104 over the Newfound River, Bristol

Opened to traffic in 1996, Route 104 over the Newfound River is a single span, 65 ft long, and 57 ft 6 in wide bridge. The superstructure consists of five AASHTO Type III girders spaced at 12 ft 6 in. The environment is considered a normal bridge over water. The HPC elements are the girders, deck, and approach slabs.

As their first HPC bridge, New Hampshire had three objectives 1) evaluate a structure which was representative of the majority of structures built in the State; 2) choose a practical concrete mix which would have an easily attainable strength and durability; and 3) test and evaluate the performance of the HPC and the in-situ performance of the bridge itself. The girders were designed for a concrete compressive strength of 8000 psi at 28 days while the deck was designed for a strength of 6000 psi. The cost of the project was $59/ft^2 of deck surface area as compared to a conventional strength bridge which costs $48/ft^2.

For the deck and approach slabs, 50 lb/yd^3 of silica fume was specified. At 56 days, less than 1000 coulombs of chloride permeability was required for the girders and deck. The deck was wet cured for four days with cotton mats and sprinklers.

For the girders, the contractor used 777 lb/yd^3 of cement and 50 lb/yd^3 of silica fume. At 56 days, compressive strength quality control tests on the girders averaged 9000 psi. For the decks, compressive strength quality control tests averaged 9600 psi at 56 days using 660 lb/yd^3 of cement and 53 lb/yd^3 of silica fume. The chloride permeability at 56 days averaged 753 coulombs for the decks.

Lessons Learned: Trial batching and trial pour played an important role in optimizing the development and placement of the HPC deck. Modifications were made throughout the pre-pour process to improve the mix proportions and eliminate unforeseeable problems. The final product exceeded expectations. No visible cracks in the deck were found during subsequent post-construction inspections. Excellent results were obtained for the 28-day concrete strength, freeze-thaw durability, chloride ion permeability, and scaling tests.

For more detailed information, click <here> to view files from the HPC Bridges – HPC Data Compilation or click the link below to read an HPC Views article on the project:

http://www.cement.org/pdf_files/hpc-4julaug99.pdf

11.4.7b New Hampshire
Route 3A over Newfound River, Bristol

Opened to traffic in January 1999, Route 3A over the Newfound River is a 65 ft long and 39 ft 6 in wide bridge. The structure has a single span and uses New England 1000 Bulb Tee girders spaced at 11 ft 6 in. The environment is considered a normal bridge over water. The HPC elements are the girders, precast panels, and cast-in-place deck.

The use of HPC allowed the New Hampshire DOT to use a shallower girder and wider spacing than if conventional strength concrete had been used. In addition, one line of girders was eliminated. The superstructure consisted of precast prestressed concrete panels as formwork and a composite cast-in-place 5.5 in deck. The concrete in the girders and deck had to meet low permeability requirements, which are expected to result in a longer service life with minimal maintenance.

For deck, the owner allowed the use of silica fume at 7.5% of cementious material weight. The chloride permeability specifications were less than 1000 coulombs at 56 days for the deck and less than 1500 coulombs at 56 days for the girders. The deck was required to be continually moist cured for seven days utilizing cotton mats and soaker hoses.

For the girders, the contractor used 550 lb/yd^3 of cement and 50 lb/yd^3 of silica fume. For the deck, the contractor used 660 lb/yd^3 of cement and 52 lb/yd^3 of silica fume. The girders achieve an average compressive strength at 28 days of 11,200 psi. The deck achieved an average compressive strength at 28 days of 9004 psi. Both the deck and girders obtained permeability well below the specified limit of 1000 coulombs.

For more detailed information, click <here> to view files from the HPC Bridges – HPC Data Compilation or click the link below to read an HPC Views article on the project:

http://www.cement.org/pdf_files/hpc-17sepoct01.pdf

11.4.8 New Mexico
Old Route 66 over Rio Puerco, Albuquerque

Opened to traffic in December 2000, Old Route 66 over Rio Puerco is a 293.3 ft long bridge. It consists of three spans of 96.1, 101.1 and 96.1 ft with four 63 in bulb tee girders spaced at 12.6 ft. The environment is considered a normal bridge over water. The HPC elements are the girders and deck. The overall cost of the bridge was $2,896,701.

New Mexico DOT's objective was to establish the viability of HPC in their State. As a result of the success of HPC, prestressed concrete beams are now produced with design concrete strengths of 7000 psi as compared to previous concrete design strengths of 6000 psi. More HPC projects are planned. Cost comparisons show that six lines of BT63 girders utilizing HPC would provide a superstructure cost savings of 7-10 percent, compared to conventional strength concrete using eight lines of BT63 girders. The use of BT63 girders with HPC was estimated to provide a cost savings of 6 to 9 percent compared to the use of Type IV girders with HPC.

For the girders, the contractor used 846 lb/yd^3 of cement and 127 lb/yd^3 of Class F fly ash. The precast beams had specified concrete compressive strengths of 7000 psi at release and 10,000 psi at 56 days. A 3-day curing period was required to achieve the concrete strengths. Actual average strengths at release and 56 days were 7500 and 10,340 psi, respectively.

For more detailed information, click <here> to view files from the HPC Bridges – HPC Data Compilation or click the link below to read an HPC Views article on the project:

http://www.cement.org/pdf_files/hpc-25novdec02.pdf

11.4.9 North Carolina
US 401 over Neuse River, Wake County

Fully opened to traffic in September 2002, the US 401 over the Neuse River bridge is 299 ft long and 42 ft wide. There are four spans, two at 92 ft and two at 57 ft. The 92 ft spans use AASHTO Type IV girders while the 57 ft spans use AASHTO Type III girders. The spacing of both girder types is 10 ft. The environment is considered a normal bridge over water. The HPC elements are the girders and deck.

The original design for the replacement bridge utilized six lines of girders. HPC made it possible to use five lines. In addition, HPC is expected to improve the durability of the bridge deck resulting in lower maintenance and life cycle costs. The total cost of the bridge was $59.61/ft^2 of deck surface.

For the deck, the owner allowed the use of Class F fly ash at 20% of cementitious material weight. The girders used up to 5% silica fume. No chloride permeability thresholds were specified for the project. The specifications for both the deck and the girders were performance based. The contractor was responsible for submitting a mix design that satisfies the performance parameters. The deck was required to be cured with burlap and polyethylene sheets kept moist for seven days.

For the girder, the contractor used 911 lb/ft^3 of cement and 51.2 lb/ft^3 of silica. For the decks, 593 lb/ft^3 of cement and 177 lb/ft^3 of Class F fly ash were used.

Lessons Learned: North Carolina is especially interested in using HPC as a way to reduce the chloride permeability of its bridge decks. The specifications for both the deck and the girders are performance based and the contractor is responsible for submitting a mix design that satisfies the performance parameters. It is anticipated that his approach will result in the lowest cost.

Concrete curing temperatures were measured at five cross sections in Girders A4, B4, C4, and D4. Ten thermocouples were placed at midspan, three at each quarter span, and three at a distance of span/50 from each end. Maximum measured concrete temperature was 162 ^{o}F.

For more detailed information, click <here> to view files from the HPC Bridges – HPC Data Compilation.

11.4.10 Ohio
U.S. Route 22 near Cambridge

Opened to traffic in November 1998, U.S. Route 22 over Crooked Creek at milepost 6.57 near Cambridge in Guernsey County is a 116.5 ft long bridge. The single span structure utilizes 42 in ODOT box beams spaced adjacent to each other. The environment is considered a normal bridge over a road. The HPC elements are the beams and abutments.

High performance concrete was used to replace a three-span non-composite bridge (using 21-in deep box beams) with a single span structure. Benefits included reduced substructure costs as well as better flow characteristics through having an unobstructed channel. The box beams used 10,000 psi compressive strength concrete and 0.6 in-diameter prestressing strands. HPC also reduced the permeability of the girders.

For the girders, the owner specified 100 lb/yd^3 of silica fume and a water cement ratio of 0.28. For the abutments, 80 lb/yd^3 of either Class F(5) fly ash or GGBFS and 88 lb/yd^3 of silica fume were specified. Compressive strengths were specified as 10,000 psi at 56 days for the girders and 8000 psi at 56 days for the abutments. For both the girders and abutments, chloride permeability of less than 1000 coulombs was required. Abutment curing consisted of an application of wet burlap and soaker hoses for seven days. The girders were steam cured for 18 hours.

For the girders, the contractor used 946 lb/yd^3 of cement and 100 lb/yd^3 of silica fume. The compressive strengths averaged 11,810 psi at 56 days. Permeability was as low as 358 coulombs at 56 days. For the abutments, the contractor used 803 lb/yd^3 of cement, 75 lb/yd^3 of GGBFS, 68.6 lb/yd^3 of fly ash, and 87.5 lb/yd^3 of silica fume. An average compressive strength of 8689 psi was obtained at 28 days.

Lessons Learned: Prior to the construction of the bridge, two prototype beams were fabricated to determine whether the HPC mix would perform satisfactory under actual fabrication and service conditions; allow the fabricator to gain experience with using a HPC mix; and predict the structural behavior.

The following conclusions were noted from tests conducted on the beams:

1. Camber at release was 0.25 in which was well below the calculated camber of 1 in.
2. The measured prestress losses, at the time the girders were loaded to crack, were 17 and 18 percent, which compares with 20 percent calculated using the AASHTO Standard Specifications.
3. The cracking moment was 21 percent higher than the cracking moment calculated using AASHTO Standard Specifications.
4. Measured flexure strengths were four percent greater than the values calculated using the AASHTO Standard Specifications.

For more detailed information, click <here> to view files from the HPC Bridges – HPC Data Compilation.

11.4.11a South Dakota I-29 Northbound Bridge

This is a three span precast I-girder bridge on Interstate 29 over a railroad located in a normal environment. Each span consists of four AASHTO Type II girders spaced at 11.5 feet. The HPC elements of this bridge are the precast girders, bent diaphragm and deck.

The use of HPC resulted in reducing the number of girder lines from five to four and increased deck durability. The average cost for this bridge was $48.03 per sq. ft. of deck. This bridge was opened to traffic in 1999.

The girder design was based on a 28-day concrete strength of 9,900 psi and a concrete strength of 8,520 psi at release. In addition, an air entrainment of 6.5% was specified for the girders and the deck. The specified minimum total cementitious material content for the deck was 684 lb./cu.yd. For the girders, the contractors used 680 lb./cu.yd of Type II cement and 84 lb./cu.yd. of silica fume with a w/c ratio of 0.25. For the deck, the contractors used 511 lb./cu.yd. of cement, 118 lb./cu.yd. of fly ash, and 55 lb./cu. yd. of silica fume. High range water reducer (A and F) and water reducer (A and F) was used in the girders and the deck. The w/c ratio for the deck was 0.39.

The girder concrete mix design resulted in an average concrete strength of 15,900 psi.

For more detailed information, click <here> to view files from the HPC Bridges – HPC Data Compilation or click the link below to read an HPC Views article on the project:

http://www.cement.org/pdf_files/hpc-16julaug01.pdf

11.4.11b South Dakota
I 29 Southbound Bridge

This is a three span precast I-girder bridge on Interstate 29 over a railroad located in a normal environment. Each span consists of four AASHTO Type II girders spaced at 11.5 feet. The HPC elements of this bridge are the precast girders, bent diaphragm, and deck.

The use of HPC resulted in reducing the number of girder lines from five to four and increased deck durability. The average cost for this bridge was \$56.70 /ft^2 of deck. This bridge was opened to traffic in 2000.

The girder design was based on a 28-day concrete strength of 9,900 psi and a concrete strength of 8,520 psi at release. The owners required that 10% of the cement used in the deck concrete be replaced with Class F fly ash at a ratio of fly ash to cement of 1.9:1.0 by weight. For the girders, the contractors used 680 lb/yd^3 of Type II cement and 84 lb/yd^3 of silica fume with a w/c ratio of 0.25. For the deck, the contractors used 590 lb/yd^3 of cement and 124 lb/ yd^3 of Class F fly ash. High range water reducer (A and F) was used for the girders and the deck. Water reducer (A) was used in the girders. The girders and the deck had air entrainment. The w/c ratio for the deck was 0.39.

The girder concrete mix design resulted in an average concrete strength of 13,250 psi.

Lessons Learned: In August 2001, an underside crack survey was conducted from the ground. Transverse cracks, fairly uniformly distributed in all the three spans, were observed. Nearly all the cracks exhibited calcium carbonate precipitation.

For more detailed information, click <here> to view files from the HPC Bridges – HPC Data Compilation or click the link below to read an HPC Views article on the project:

http://www.cement.org/pdf_files/hpc-16julaug01.pdf

11.4.12a Tennessee Hickman Road over State Route 840

Opened to traffic in September 2000, the Hickman over S.R. 840 bridge in Dickson County is a 290 ft 8 in long bridge. It consists of two spans and utilizes 72 in bulb tee girders spaced at 8 ft 4 in. The environment is considered a normal bridge over a roadway. The HPC elements are the retaining walls, abutments, bent, girders, and deck.

HPC was used to provide longer span lengths and a more durable structure. Without the use of HPC, steel girders would have been used at an additional cost of $825,000. The cost of the beams, deck concrete, and substructure concrete was $160/ft, $315 lb/yd^3, and 240 lb/yd^3 respectively. The total cost of the bridge, excluding pavement at bridge ends, was $59/ft^2.

For the girders, deck, and substructure, the owner allowed the use of silica fume up to 8% of cementitious material weight. The specified compressive strengths at 28 days for the girders, deck, and substructure were 10,000, 5000, and 4000 psi, respectively. The girders used 0.6 in prestressing strands. Specified chloride permeability at 28 days for the girders, deck, and substructure was less than 2500, 1500, and 3000 coulombs respectively. The deck was cured with fogging followed by membrane curing, wet burlap, and vapor barrier.

For the girders, the contractor used 747 lb/yd^3 of cement and 249 lb/yd^3 of Class C fly ash. The compressive strengths averaged 10,259 psi at 28 days and 11,249 psi at 56 days. The chloride permeability averaged 496 coulombs at 56 days. For the deck, the contractor used 496 lb/yd^3 of cement, 194 lb/yd^3 of Class C fly ash, and 50.7 lb/yd^3 of silica fume. An average compressive strength of 7197 psi at 56 days was obtained. Chloride permeability at 56 days averaged 707 coulombs.

Lessons Learned: Strain and temperatures were measured at 14 locations in each of two girders. The deflection values due to individual effects of temperature change and truck load were found to be significantly smaller than the AASHTO limitation of Span/800. This test clearly demonstrated the role of pile foundation under the abutments in abutment movement due to temperature change and live load. It was, however, noted that temperature change has more influence than live load. The live load distribution coefficients based on the test data were found to be significantly different from the predicted values, say, by AASHTO recommended lever rule.

For more detailed information, click <here> to view files from the HPC Bridges – HPC Data Compilation or click the link below to read an HPC Views article on the project:

http://www.cement.org/pdf_files/hpc-10julaug00.pdf

11.4.12b Tennessee
Porter Road over State Route 840

Opened to traffic in May 2000, the Porter Road over S.R. 840 bridge in Dickson County is 318 ft long and 32 ft wide. It consists of two 159 ft spans and utilizes 72 in bulb tee girders spaced at 8 ft 4 in. The environment is considered a normal bridge over a roadway. The HPC elements are the retaining walls, abutments, bent, girders, and deck.

HPC was used to provide longer span lengths and a more durable structure. Without the use of HPC, steel girders would have been used at an additional cost of $825,000. The cost of the beams, deck concrete, and substructure concrete was $160/ft, $315 lb/yd^3, and 240 lb/yd^3 respectively. The total cost of the bridge, excluding pavement at bridge ends, was $56/ft^2.

For the girders, deck, and substructure, the owner allowed the use of silica fume up to 8% of cementitious material weight. The specified compressive strengths at 28 days for the girders, deck, and substructure were 10,000, 5000, and 4000 psi, respectively. The girders used 0.6 in prestressing strands. Specified chloride permeability at 28 days for the girders, deck, and substructure was less than 2500, 1500, and 3000 coulombs respectively. The deck was cured with fogging followed by membrane curing, wet burlap, and vapor barrier.

For the girders, the contractor used 747 lb/yd^3 of cement and 249 lb/yd^3 of Class C fly ash. The compressive strengths averaged 9651 psi at 28 days and 10,090 psi at 56 days. The chloride permeability averaged 390 coulombs at 56 days. For the deck, the contractor used 496 lb/yd^3 of cement, 194 lb/yd^3 of Class C fly ash, and 50.7 lb/yd^3 of silica fume. An average compressive strength of 8265 psi was obtained at 28 days and 8713 psi at 56 days. Chloride permeability at 56 days averaged 1297 coulombs.

Lessons Learned: Strain and temperatures were measured at 14 locations in each of two girders. The deflection values due to individual effects of temperature change and truck load were found to be significantly smaller than the AASHTO limitation of Span/800. This test clearly demonstrated the role of pile foundation under the abutments in abutment movement due to temperature change and live load. It was, however, noted that temperature change has more influence than live load. The live load distribution coefficients based on the test data were found to be significantly different from the predicted values, say, by AASHTO recommended lever rule.

For more detailed information, click <here> to view files from the HPC Bridges – HPC Data Compilation or click the link below to read an HPC Views article on the project:

http://www.cement.org/pdf_files/hpc-10julaug00.pdf

11.4.13a Texas
Louetta Road Overpass, Houston

The Louetta Road Overpass is a multiple span bridge on State Highway 249 and is located in normal environment. Its spans are simply supported (maximum span length is 135.5 feet) and comprise of six precast Texas US 54 (bath tub) girders, with the girder spacing varying from 11 feet to over 16 feet. The HPC elements of this bridge include the precast piers, precast girders and deck panels and cast-in-place (CIP) deck.

The use of HPC increased deck durability and permitted contractors to utilize preferred methods for bridge construction. The unit cost was $24.09/ft^2 of deck (conventional concrete was $23.61/ft^2). This bridge was opened to traffic in 1998.

The two main requirements were high design strength (9,800 psi to 13,100 psi @ 56 days) and high early strength (minimum 6,000 psi to 8800 psi @ release) for the girders. Neither a permeability requirement for the deck or any admixture was explicitly specified.

For the HPC girders, the contractors used type III cement (671 lb/yd^3) along with 315 lb/yd^3 of Type C fly ash, at a w/c ratio of 0.25. Type C fly ash was also used in the deck panels and CIP deck. High range reducer (Type F) was used in the girders and deck panels and in the CIP deck (Type F). Type B and D retarders were used in the girders, deck panels, and CIP deck. The girder concrete strengths (56 day) achieved varied from 12,170 psi to 14,440 psi.

For more detailed information, click on <here> to view files from the HPC Bridges – HPC Data Compilation or click the link below to read an HPC Views article on evolution of HPC in Texas:

http://www.cement.org/pdf_files/hpc-30novdec03.pdf

11.4.13b Texas
U.S. Route 67 over North Concho River, U.S. Route 87, and South Orient Railroad, San Angelo

The San Angelo Bridge on US Rte 67/87 spans over a river and railroad. This bridge is a multi-span precast I-girder bridge located in normal environment. The eastbound (EB) bridge has eight spans (max. 157 ft) and four girders per span at 11 feet. The westbound (WB) bridge has nine spans (max. 131 ft) and seven girders per span at 5 ft 8 in. The girders are predominantly AASHTO Type IV (a few are Texas Type B). The HPC elements include precast girders and deck panels (EB bridge) and cast-in-place (CIP) deck (EB and WB bridge).

HPC resulted in a reduced number of spans and girders/span on the EB bridge and increased deck durability. Average costs were $42.03 /ft^2 of deck (EB bridge) and $45.38 /ft^2 of deck (WB bridge). The bridge was opened to traffic in 1998

The owners required a permeability corresponding to a maximum of 1500 coulombs for the decks. The specified concrete strength for the girders (EB) varied from 12,500 psi to 14,000 psi (56 days). A high early strength of 8,000 to 8,100 psi (at release) was specified for the EB bridge. In addition, a minimum of 20% flyash was required if reactive aggregates were used (maximum flyash was 35%). The concrete mix also required a 50% ground granulated blast furnace slag (0 % with type IP cement), and a maximum w/c ratio of 0.49 for girders and 0.44 for deck.

The contractors used 671 lb/yd^3 of cement (min. specified was 564 lb/ yd^3) for the HPC. Type C fly ash for the girders and CIP deck was used. In addition, Type F high range reducers for the girders and EB CIP deck, and Type D for the deck panels, Type B for the girders, and Type B and D for the CIP deck were used. The w/c ratio was 0.25 for the girders and 0.27 for the deck. Girder concrete strengths that were achieved ranged from 13,830 psi to 15,240 psi.

For more detailed information, click on <here> to view files from the HPC Bridges – HPC Data Compilation or click the link below to read an HPC Views article on the evolution of HPC in Texas:

http://www.cement.org/pdf_files/hpc-30novdec03.pdf

11.4.14a Virginia
Route 40 over Falling River, Brookneal in Lynchburg District

Opened to traffic in May 1996, Route 40 over Falling River is a 320 ft long bridge with a 44 ft roadway. There are four 80 ft spans of AASHTO Type IV girders spaced at 10 ft. The environment is considered a normal bridge over water. The HPC elements are the substructure, girders, and deck.

The 8000-psi concrete compressive strength in the girders enabled the use of five lines of girders rather than the seven lines that would have been required with a compressive strength of 6000 psi. The total cost of the bridge was $49.32/ft^2 of deck surface. This was lower than the average Federal-aid cost of $58/ft^2 for bridges built that year. Concrete had to meet low permeability requirements, which are expected to result in longer service life with minimal maintenance costs.

For the girders, substructure, and deck, the owner allowed the use of fly ash (20-35%), silica fume (7-10%), and GGBFS (35-50%). The chloride permeability requirements were 1500 coulombs for the girders, 3500 coulombs for the substructure, and 2500 coulombs for the deck based on modified (uses a curing procedure of one week at 73 ^{o}F and three weeks at 100 ^{o}F) AASHTO T 277 at 28 days. The deck was required to be cured with wet burlap covered with plastic sheeting for seven days and then curing compound applied after removal of plastic sheeting and burlap.

For the girder, the contractor used 752 lb/yd^3 of cement and 55 lb/yd^3 of silica fume and obtained a chloride permeability of less than 500 coulombs based on trial batches. For the substructure, trial batches had permeability of less than 1850 coulombs based on using 353 lb/yd^3 of cement and 235 lb/yd^3 of ground granulated blast furnace slag (GGBFS). For the decks, permeability of around 1100 coulombs at 28 days was obtained using 329 lb/yd^3 of cement and 329 lb/yd^3 of GGBFS.

Lessons Learned: A load test was conducted on one production beam. The purpose of the test was to determine the amount of residual deflection in the beam after removal of a maximum load equal to 95 percent of the calculated load to cause a flexural crack. The beam was considered to be adequate if a recovery of 90 percent or greater was achieved. The instantaneous recovery was 97.9 percent. At 30 minutes after removing the load, the recovery was 100 percent. No cracks were seen in the beam.

Before initiation of the Route 40 project, an experimental project was conducted to support the design of high-strength, low permeability beams with 0.6-in-diameter strands. Four prestressed concrete AASHTO Type II beams each containing ten 0.6-in-diameter strands at 2-in center-to-center spacing were fabricated at a prestressing plant and tested to failure at the FHWA Structures Laboratory.

Trial batches were prepared at the plant and in the laboratory before the preparation of actual field concretes for the test beams. Concrete tests showed that high-strength air-entrained concretes with 28-day strengths exceeding 10,000 psi and a minimum release strength of 70 percent of the 28-day strengths could be produced with a water-cementitious materials ratio (w/cm) of about 0.30 or less. To achieve such low w/cm required high amounts of cementitious material, proper selection of aggregates, and high dosages of HRWRA. To achieve high early strengths, proper temperature management was needed. With low curing temperatures, it is difficult to achieve high early strengths, but higher ultimate strengths can be achieved.

Optimum temperature for both the early and ultimate strengths could only be determined by trial batching and testing.

In two of the beams, the temperature within the beam was continuously monitored by thermocouples embedded prior to casting. During steam curing, a temperature of 160 ^{o}F was planned within the enclosure. However, the enclosure temperature inadvertently approached 185 ^{o}F, which resulted in concrete temperatures of 219 ^{o}F and 208 ^{o}F in two beams. Some of the test specimens stored in the recesses of the forms exhibited visual cracks attributed to high heat. These specimens exhibited high variability in strength and some did not meet the strength requirements due to heat-related damage. However, some of the specimens had strengths above the requirements.

The beam with high internal temperatures exceeding boiling performed satisfactory during load testing. All flexural failures were due to concrete crushing in the outermost fibers of the top flange. The prestressed beams with 0.6-in-diameter strands had satisfactory concrete strengths (exceeding 10,000 psi), and performed as intended under the loading condition and did not reflect the high variability and lower-than-desired strengths observed in the test specimens damaged by heat.

For more detailed information, click <here> to view files from the HPC Bridges – HPC Data Compilation.

11.4.14b Virginia
Virginia Avenue over Clinch River, Richlands

Opened to traffic in December 1997, Virginia Avenue over Clinch River is a 148 ft long and 42 ft wide bridge. There are two 74 ft spans of AASHTO Type III girders spaced at 8 ft 9 in and 9 ft 3 in. The environment is considered a normal bridge over water. The HPC elements were the girders and deck.

The original design for the replacement bridge used conventional concrete and had three spans with seven beams per span. The replacement bridge uses two spans with five beams per span. The total cost of the bridge was $60.43/ft^2 of deck surface. This was lower that the average cost of $69/ft^2 for similar bridges. Concrete had to meet low permeability requirements, which are expected to result in longer service life with minimal maintenance costs.

For the girders and deck, the owner allowed the use of fly ash (20-35%), silica fume (7-10%), and GGBFS (35-50%). The chloride permeability requirements were 1500 coulombs for the girders and 2500 coulombs for the deck based on modified (uses a curing procedure of one week at 73 ^{o}F and three weeks at 100 ^{o}F) AASHTO T 277 at 28 days. The deck was required to be cured with wet burlap covered with plastic sheeting for 7 days and then curing compound applied after removal of plastic sheeting and burlap.

For the girder, the contractor used 752 lb/yd^3 of cement and 75 lb/yd^3 of silica fume and obtained a chloride permeability of less than 200 coulombs based on trial batches. For the decks, permeability of less than 1500 coulombs at 28 days was obtained using 560 lb/yd^3 of cement and 140 lb/yd^3 of Class F fly ash.

Lessons Learned: Prior to construction, a test program was initiated to develop the desired concrete and to determine the structural feasibility of using HPC beams with 0.6-in-diameter strands spaced at 2 in. Two full-scale prestressed concrete beams with composite deck slabs were fabricated and tested. The beams were tested using a concentrated load located at various distances from the beam end. The measured cracking load and ultimate load exceed the predicted values in all cases. Both measured transfer and development lengths were much lower than those predicted by AASHTO equation 9.32

A load test was conducted on one production beam. The purpose of the test was to determine the amount of residual deflection in the beam after removal of a maximum load equal to 95 percent of the calculated load to cause a flexural crack. The beam was considered to be adequate if a recovery of 90 percent or greater was achieved. The instantaneous recovery was 98.5 percent. At 30 minutes after removing the load, recovery was 100 percent. No cracks were seen in the beam.

For more detailed information, click <here> to view files from the HPC Bridges – HPC Data Compilation.

11.4.15a Washington
Eastbound lanes of State Route 18 over State Route 516 (King County)

Opened to traffic in March 1998, State Route 18 over State Route 516 is a 297 ft long 38 ft wide bridge. There are three spans of 80, 137, and 80 ft of WSDOT W74G girders spaced at 8 ft. The environment is considered a normal bridge over a road. The HPC elements were the girders and deck.

High performance concrete was specified for the prestressed concrete girders to improve durability and strength of the concrete. The use of high-strength concrete allowed the number of lines of girders to be reduced from seven to five. This resulted in a savings of 594 ft of girders. The girders cost $153/linear ft and the deck was built for $250,000.

For the girders, the owner specified the use of fly ash (25%) and for the deck specified 735 lb/yd^3 of cement and 75 lb/yd^3 of fly ash. The chloride permeability requirements were 1000 coulombs for the girders based on AASHTO T 277 at 56 days and not specified for the deck. The deck was to be cured with two coats of curing compound and wet cured using quilted blankets or burlap for 14 days.

For the girder, the contractor used 728 lb/yd^3 of cement, 222 lb/yd^3 of fly ash, and 50 lb/yd^3 of silica fume and obtained a chloride permeability of around 1000 coulombs. For the decks, permeability of less than 2800 coulombs at 28 days was obtained using 660 lb/yd^3 of cement and 75 lb/yd^3 of fly ash.

Lessons Learned: Prior to fabricating the production girders, a 20-ft-long test girder was fabricated for the precaster to gain experience with using the HPC mix and for the researchers to gain experience installing the instrumentation under field conditions. Some measurements from the test girders are included in HPC Bridges – HPC Data Compilation. Others are available in the "High Performance Concrete in Washington State SR 18/SR 516 Overcrossing: Final Report on Girder Monitoring," Washington State Transportation Center, December 2000, by Barr, P..

For more detailed information, click <here> to view files from the HPC Bridges – HPC Data Compilation.

11.5 References

1. Symposium Proceedings, PCI/FHWA International Symposium on High Performance Concrete, New Orleans, LA,, Precast/Prestressed Concrete Institute, Chicago, IL, 1997.

2. Symposium Proceedings, PCI/ FHWA/fib International Symposium on High Performance Concrete, Orlando, FL, Precast/Prestressed Concrete Institute, Chicago, IL, 2000.

3. SHRP High Performance Concrete Bridge Showcase Notebook, Houston, TX, March 25-27, 1996.

4. SRP High Performance Concrete Regional Showcase Notebook, Omaha, NE, November 18-20, 1996.

5. SHRP High Performance Concrete Bridge Showcase Notebook, Richmond, VA, June 24-26, 1997.

6. SHRP High Performance Concrete Bridge Showcase Notebook, Belleview, WA, August 18-20, 1997.

7. SHRP High Performance Concrete Bridge Showcase Notebook, Waterville Valley, New Hampshire, September 22-23, 1997.

8. Colorado High Performance Concrete Showcase Notebook, Denver, CO, February 18-20, 1998.

9. SHRP High Performance Concrete Bridge Showcase Notebook, Cincinnati, OH, February 23-24, 1999.

10. Southeast Regional High Performance Concrete Showcase Notebook, Auburn, AL, June 29-July 1, 1999.

SECTION 12

ACKNOWLEDGEMENTS

The extensive amount of time and effort contributed by each Section author to compile this comprehensive Guide is sincerely appreciated. This includes Ms. Marcia Simon, retired from the FHWA Office of Infrastructure – Research and Development, Mr. Jerry Potter, retired from the FHWA Office of Bridge Technology, and Mr. Benjamin Graybeal of PSI, Inc., who is currently working under contract with the FHWA. We also wish to thank each of the reviewers for their diligent overview to ensure consistency and continuity of the many drafts of each Section. This includes Messers. Myint Lwin, Jerry Potter, Douglas Edwards and Frank Rich, all of the FHWA. Finally, we thank Ms. Deborah Vocke, Marketing Specialist with the FHWA Resource Center, for her outstanding editorial review and comments in the preparation of this document and its CD reproduction.

APPENDIX A
SAMPLE RESEARCH PROBLEM STATEMENTS

I. PROBLEM NUMBER 5

II. PROBLEM TITLE Curing of High Performance Concrete

III. RESEARCH PROBLEM STATEMENT

The three curing methods for concrete bridge decks are water curing, curing compound, or the waterproof cover method. Curing is necessary to maintain favorable moisture and temperature conditions in the concrete.

Water curing is favored especially when the water-cementitious materials ratio (w/cm) is below 0.40. In these concretes, autogenous curing is considered to cause shrinkage, which is additive to other shrinkage factors that may lead to cracking of the concrete. Whether water curing prevents autogenous shrinkage is not clear from the research. In water curing, it is not clear how long water supplied to the concrete can penetrate in helping with autogenous shrinkage or further curing. This is particularly true for high performance concrete (HPC).

AASHTO M 148 for liquid membrane-forming compounds requires a moisture loss of no more than 0.55 kg/m2 (0.11 1b/ft2) in 72 hours and a daylight reflectance of not less than 60 percent. In summer months, these compounds contain white pigments to reflect solar energy and minimize heating of the concrete.

AASHTO M 171 for sheet materials requires moisture loss of no more than 0.55 kg/m2 (0.11 lb/ft2) in 72 hours and a daylight reflectance of at least 50 percent for white curing paper and 70 percent for white polyethylene film.

All three curing methods need to be evaluated for HPC used in bridge decks. In addition to testing laboratory specimens, cores shall be taken from the field projects to determine the effectiveness of the curing methods.

IV. RESEARCH OBJECTIVE(S)

The objective of the proposed research is to establish effective curing methods for HPC. It will evaluate if a curing compound or waterproof covers with certain reflectance and moisture retention can be successfully used in HPC with different cementitious materials and w/cms, and if water curing reduces drying and autogenous shrinkage. To accomplish the objectives, the following tasks will be performed:

Task 1: Review the literature and identify the curing methods for HPC. The review shall include the following documents:

- AASHTO M 148 (ASTM C 309) - Liquid Membrane-Fonning Compounds for Curing Concrete
- AASHTO M 171 (ASTM C 171) - Sheet Materials for Curing Concrete
- AASHTO T 23 (ASTM C 31) - Making and Curing Concrete Field Test Specimens
- AASHTO T 24 (ASTM C 42) - Obtaining and Testing Drilled Cores and Sawed Beams of Concrete

- AASHTO T 126 (ASTM C 192) - Making and Curing Concrete Test Specimens in the Laboratory
- AASHTO LRFD Bridge Construction Specifications
- ASTM C IISI - Evaluating the Effectiveness of Materials for Curing (This test method was withdrawn in June 2000 since it was not updated in eight years as required by ASTM).
- ICI, TC 003 Technical Committee Report on the Autogenous Shrinkage of Concrete (Japan Concrete Institute Technical Committee on Autogenous Shrinkage of Concrete), November 1996.

Task 2: Develop a detailed work plan and test program to obtain the necessary data. Describe how the work plan will provide the necessary data. It is anticipated that water curing using burlap, cotton mats, ponding, fogging, or soaker hoses; curing compounds: and various sheet materials including curing paper, polyethylene film, and burlappolyethylene sheet will be evaluated for moisture retention and temperature control. Concretes cured with these materials will be tested to determine the effectiveness of curing. Laboratory specimens and cores with varying dimensions shall be tested. A statistical analysis shall be conducted to determine the variability of each test and the differences in the methods used.

Task 3: Conduct field tests under a variety of typical outdoor environments.

Task 4: Prepare specifications for curing compounds, waterproof covers, and water curing for concrete with different strengths and penne abilities.

Task 5. Submit a final report documenting the entire work effort including recommendeds to the specifications and test methods.

V. ESTIMATE OF PROBLEM FUNDING AND RESEARCH PERIOD

Recommended Funding:

Research Period:

VI. URGENCY/PRIORITY

The current curing requirements of the AASHTO specification may not be appropriate for HPC. Improper curing results in poor quality concrete with undesirable cracking. As the industry moves towards the greater use of high performance concrete, the need to revise the curing methods becomes more urgent.

User Community

Results of the research will be directly applicable to the AASHTO Highway Subcommittees on Materials and Bridges and Structures and will benefit the whole bridge community.

Implementation

Research results will be implemented with proposed revisions to the *AASTHO Material Specifications*, the *AASTHO Test Methods*, and the *AASHTO LRFD Bridge Construction Specifications*.

Effectiveness

The benefits of this research include more effective curing methods and will result in longer lasting concretes, requiring less maintenance.

Thrust Areas/Business Needs

The proposed research addresses the AASHTO Highway Subcommittee on Bridges and Structures (HSCOBS) thrust area of Enhanced Specifications for Improved Structural Performance and/or Enhanced Materials, Structural Systems, and Technologies.

The associated building blocks are specifications for high performance materials, performance based specifications, high performance concrete, and performance based acceptance criteria.

APPENDIX B –
LIFE CYCLE COST ANALYSIS

B1 INTRODUCTION

Some Roman bridges are over 2,000 years old and still in service. Many modern bridges are in distress after less than 30 years of service. Are the Roman bridge builders better than the modern bridge engineers? This question challenges the bridge engineers to use modern technology and economic principles to arrive at sound and economical decisions to assure that modern highway bridges will provide the services for which they are intended. It is a challenge to invest wisely, to make efficient use of transportation resources and to provide a safer highway environment. Life-cycle cost analysis (LCCA) is a technique for meeting the challenge.

Section 303 of the National Highway System Designation Act (1995) defines LCCA as "a process for evaluating the total economic worth of a usable project segment by analyzing initial costs and discounted future cost, such as maintenance, reconstruction, rehabilitation, restoring, and resurfacing costs, over the life of the project segment." The Act defines a usable project segment as a portion of the highway that could be opened to traffic independent of some larger project of which the segment is a part. The Act requires LCCA for projects with an estimated total cost of $25,000,000 or more. LCCA may also be applicable to smaller projects given the potential longevity for HPC structures and possible similar costs to non-HPC structures.

LCCA has been used in management and engineering decisions for a long time. For example, the classic application of LCCA is in comparing the cost effectiveness of corrosion protection systems for the service life of steel bridges. In a similar way LCCA can be used to determine which type of bridge deck overlays, including a "no overlay" option, has the lowest life-cycle cost. Another example is in the use of LCCA in comparing alternative construction materials and types of structures for a bridge. LCCA often reveals that the lowest initial cost is not necessarily the best choice in the long term. One of the basic purposes of performing an LCCA is to determine when it is more advantageous to repair or rehabilitate a bridge rather than replace it, and alternatively, when replacement is more cost-effective than rehabilitation, etc.

B2 LIFE-CYCLE COSTS

The cost to the owner of a bridge is the sum of all costs involved in initial construction, operating, maintaining, repairing, rehabilitating, disposing and other incidental costs to the owner, users and third parties. The costs for these activities are interdependent. The actions taken or not taken during initial construction affect the frequency and cost of maintenance. The actions taken or not taken in maintenance will affect the needs for repair and other long-term costs. Bridge designers play a very influential role in the initial construction cost and subsequent operation, maintenance, repair and rehabilitation (OM&R) costs and also in the impact to users. The designers must choose construction materials carefully, pay attention to structural details for durability and long-term performance, and make provisions for future improvement and widening as appropriate. The quality control and quality assurance (QC/QA) personnel also play a very important role in making sure that the materials and workmanship in fabrication and construction meet plans and specifications. LCCA is tool that can be used by bridge designers to systematically estimate the life-cycle costs of bridges using alternative construction materials and designs.

The initial construction (IC) costs, including planning, design and construction, are generally the most readily available cost data. The IC cost data may be based on the engineer's estimate, the low bid, the average of three lowest bids or some cost estimate that is reasonably reflective of project cost. The OM&R costs are more difficult to estimate for owners who have limited or no records. For states which have implemented a bridge management system (BMS), such as PONTIS or BRIDGIT, the OM&R costs may be obtained from the BMS. The user and third party (U&TP) costs are the most difficult for bridge designers to assess. U&TP costs are caused by temporary closures of bridge lanes for maintenance, repair and rehabilitation. Traffic congestion delays cause increased fuel consumption and time lost to users and affect third party businesses.

The IC costs are present costs. The OM&R and U&TP costs are expected to be incurred and accrued in the future. The basic formulas used in LCCA are as follows:

1. Formula for computing future cost (FC) of an activity, such as maintenance, repair, rehabilitation:

$$FC = C (1 + i)^t \qquad (1)$$

Where C = present cost
i = inflation or deflation rate
t = number of years

2. Formula for converting future costs to present value (PV) for cost comparison:

$$PV = FC / (1 + d)^t \qquad (2)$$

Where d = discounted rate
t = number of years

Note: When future costs are expressed in constant or real dollars, the real discount rate can be used in Formula (2). This means that uhe inflation or deflation rate of Formula (1) will not apply. The discount rate adjusts costs for the real earning opportunities of money over time. Government agencies tend to use discount rates and constant dollars in their analyses.

Life-cycle costs are of great interest to state departments of transportation and other agencies tasked with bridge-management decisions in stretching the limited resources in construction, maintenance, repair, rehabilitation or replacement of highway bridges. Currently, there is no commonly accepted methodology for performing LCCA. Old methods have been used and new methods are being explored. Computer programs have been developed to help conduct LCCA. Two of the computer programs for LCCA will be discussed in the following sections.

B3 NIST BRIDGELCC

B3.1.1 Introduction

BridgeLCC was developed by Dr. Mark Ehlen, an economist and civil engineer with the National Institute of Standard and Technology (NIST). It is a user-friendly Windows™ based software program developed to help bridge designers determine the cost effectiveness of alternative construction materials and bridge designs. The software uses a life-cycle costing methodology based on the ASTM Standard for Life-Cycle Costing (ASTM E917), and a cost classification scheme developed by NIST. It is developed with the first-time user in mind. It can compare the life-cycle costs of two to four alternative construction materials or designs.

B3.1.2 Computer System Requirements

BridgeLCC is designed to run on Windows™ 3.x, Windows™ 95/98, and Windows™ NT 4.0. The computer should have at least a 486-66 MHz processor, 8 megabytes of RAM, 15 megabytes of available hard disk space, and a video card that supports 800x600 resolution or better.

BridgeLCC can be installed either from a compact disk (CD) or by downloading the software from the BridgeLCC web site, http://www.bfrl.nist.gov/BridgeLCC/welcome.html.

B3.1.3 Applications of BridgeLCC

BridgeLCC has multiple applications in project evaluations. Although it is specifically tailored to highway bridges, it can also be applied to pavements, piers, and other civil infrastructure. Three most common applications to bridge construction are outlined below:

Accept/Reject Decision

Choosing whether or not to do a project is an accept/reject decision. One example is deciding whether to overlay an existing bridge deck with latex modified concrete or leave the deck "as is". The decision rule is to choose the alternative with minimum life-cycle cost.

Material/Design Decision

This application occurs when choosing the most cost-effective of multiple material/design alternatives to satisfy an objective. The decision rule is to choose the material/design with minimum life-cycle cost. For example, given a particular material, what fabrication and construction method minimizes life-cycle cost? In this application, the decision has already been made to replace the deck with a particular material; the life-cycle cost analysis is needed to decide which design is most cost effective.

Efficiency Level or Size Decision

Choosing how much of something to invest in is the efficiency level or size decision. An example is choosing the thickness of polymer-concrete overlay to apply to a bridge deck. The decision rule is to choose the thickness of the overlay that minimizes the life-cycle cost of the polymer-concrete road surface.

B3.1.4 Users Manual

The Users Manual describes the methodology used, the basic functions, and options of BridgeLCC. The steps involved in performing an LCCA are clearly explained and illustrated with an example comparing conventional and high-performance concrete bridges. The Manual is written with first-time users in mind and guides users through the analysis to making the right choice in selecting the life-cycle cost-effective alternative.

Information about BridgeLCC, and the Users Manual are posted on the Office of Applied Economics web site, http://www.bfrl.nist.gov/oae.html and may be obtained by writing to Office of Applied Economics, Building and Fire Research Laboratory, National Institute of Standards and Technology, 100 Bureau Drive, Stop 8603, Gaithersburg, MD 20878-9950.

B3.2 NCHRP 12-43 Life-Cycle Cost Analysis For Bridges (BLCCA)

Transportation officials consider life-cycle cost analysis an important technique for assisting with investment decisions. Several recent legislative and regulatory requirements recognize the potential benefits of life-cycle cost analysis and call for consideration of such analyses for infrastructure investments, including investments in highway bridge program. However, a commonly accepted, comprehensive methodology for bridge life-cycle cost analysis (BLCCA) currently does not exist. Through the National Cooperative Highway Research Program, NCHRP 12-43, was initiated in May 1996 to develop a methodology for BLCCA for use by transportation agencies. The research was completed and a report titled "NCHRP Report 483 Bridge Life-Cycle Cost Analysis" was published in 2003. The report can be ordered from TRB at www4.trb.org.

NCHRP Report 483 contains the findings of NCHRP 12-43 and describes the research effort leading to the recommended methodology and includes a guidance manual for carrying out BLCCA and software that automates the methodology. The proposed methodology is fully described in the guidance manual. The CD-ROM that accompanies NCHRP Report 483 contains the BLCCA software package and the User's Manual. The User's Manual presents four examples of the application of the methodology. The Guidance Manual and the appendixes to the report are accessible from the software.

B3.3 Life-365

Life-365 is a computer program for predicting the service life and life-cycle costs of reinforced concrete exposed to chlorides. The program was written by Even Bentz and Michael Thomas and funded by Master Builders Technologies, Grace Construction Products and the Silica Fume Association.

Life-365 uses a four-step approach to estimate life-cycle costs. The four steps are:

- Step 1: Predict the time to onset of corrosion (initiation period, t_i).
- Step 2: Predict the time for corrosion to reach an unacceptable level (propagation period, t_p). It is assumed the time to first repair, $t_r = t_i + t_p$.
- Step 3: Determine the repair schedule after the first repair.
- Step 4: Estimate the life-cycle costs based on the initial concrete costs and future repair costs.

Life-365 includes models for predicting the initiation period, t_i, and the propagation period, t. Life-365 predicts the time to the first repair, t_r, by considering the properties of concrete, corrosion protection and the environment. The owner will decide on the cost and extent of the first repair and future repairs. The owner is also responsible for providing cost data for life-cycle cost analysis. Life-365 has default values for deterioration rates and cost information, which can be changed by the owner.

Life-365 Version 1.0 makes assumptions and simplifications in dealing with the complex issues of corrosion and in areas with limited knowledge. Future versions of Life-365 will address the limitations of Version 1.0 to allow for more rigorous analysis.

Life-365 may be downloaded from the Silica Fume Association's website: www.silicafume.org.

B4 REFERENCES

1. Ehlen, M.A., *"BridgeLCC 1.0 Users Manual - Life-Cycle Costing Software for Preliminary Bridge Design"*, National Institute of Standards and Technology, Gaithersburg, Maryland, April 1999.

2. NCHRP Project 12-43 Draft Report, *"Guidance Manual on Bridge Life-Cycle Cost Analysis (BLCCA)"*, Transportation Research Board, Washington, D.C., May 1998.

3. Lwin, M.M., *"Zinc-Plus-Paint System for Corrosion Protection of a Steel Bridge"*, ASTM STP 841, Proceedings of Symposium on New Concepts for Coating Protection of Steel Structures, Lake Buena Vista, Florida, January 26, 1983.

4. FHWA, 1994, *"Life-Cycle Cost Analysis: Summary of Proceedings"*, FHWA Life-Cycle Costs Symposium, Washington, D.C. December 15-16, 1993.

5. Mohammadai, J., Guralnick, S.A., Yan, Li, (1995) *"Incorporating Life-Cycle Costs in Highway-Bridge Planning and Design"*, ASCEJournal of Transportation Engineering, Vol. 121(3), pp. 417-424.

6. Thomas, M.D.A. and Bentz, E.C. *"Life-365 Computer Program for Predicting the Service Life and Life-Cycle Costs of Reinforced Concrete Exposed to Chlorides",* October 2, 2000.

FHWA Publication No.
FHWA-RCBAL-05-007