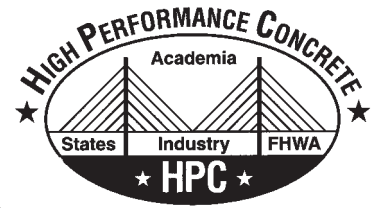




Bridge Views



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HPC FOR I-235 BRIDGE RECONSTRUCTION IN IOWA

Kenneth F. Dunker, Iowa Department of Transportation*

In 1999, a committee from the Iowa Department of Transportation (DOT) and the Iowa FHWA office began exploring the use of high performance concrete (HPC) for bridge projects in the reconstruction of I-235—an urban interstate loop around Des Moines, IA.

The initial expectations for HPC were reduced permeability, increased durability, increased strength when needed, and reduced cracking. To achieve these expectations, the committee evaluated concrete mix proportions, materials selection, timing and duration of curing, winter placement practices, and placement size and sequence. The committee also identified specific performance criteria for the substructure components; precast, prestressed concrete beams; deck; and barrier rails.

In 2000, the Office of Materials began evaluating the standard Iowa DOT mix designs for cast-in-place concrete and precast, prestressed concrete. The standard mixes were improved by reducing the water-cementitious materials ratio, adding supplementary cementing materials, and improving aggregate gradation, while testing for strength, permeability, and cracking tendency.

An early bridge deck experiment off the I-235 corridor included the use of silica fume and a high-range water-reducing admixture (HRWR). Although trial mix testing gave favorable results, the on-site slump and air content were inconsistent, and pumping reduced the air content.

After this experience, the Office of Materials revised the HPC deck mix to eliminate silica fume and the HRWR, to make ground granulated blast-furnace slag (GGBFS) the primary supplementary cementing material, and to use a water-reducing and retarding admixture. By late 2002, the HPC mix design and construction specifications had matured, and test data had stabilized to the values shown in the table.

Curing of deck concrete takes place immediately after finishing, with longitudinal grooving for velvet ride and skid resistance delayed until the concrete has hardened. The curing specifications include prohibition of curing compound, two layers

of pre-wetted burlap to be placed less than 10 minutes after final finishing, and continuous wet sprinkling for 7 days. Deck cracking has not been a problem, although the 42nd Street Bridge did develop a transverse crack above the central pier shortly after construction.

Use of GGBFS and Class C or F fly ash in the HPC mix for substructure components has reduced the heat of hydration and slowed strength gain. For hot weather concreting, the reduced heat has been an advantage, but it is a disadvantage for cold weather concreting.

The Iowa DOT found an opportunity in a major corridor project to adjust standard concrete mixes for improved performance. The agency worked with the local ready-mix concrete suppliers and precast, prestressing plants; developed changes; and tested trial mixes to meet the expectations for HPC. The final results are three durable concretes for all bridge components above the foundations.

Specified and Measured Properties

Component	Specified Value	Average Test Value
28-Day Compressive Strength, psi		
Substructure	5000	6212
Beams	To 9000	—
Deck	5000*	6658
28-Day Chloride Permeability,** coulombs		
Substructure	2500	1557
Beams	2500	—
Deck	1500	1287

* Selected to be the same value as the substructure
** Using AASHTO T 277 after curing for 7 days at 73°F (23°C) and 21 days at 100°F (38°C)

* Contributors to the article were Ahmad Abu-Hawash, Todd Hanson, Kimball Olson, and Wayne Sunday.

HPC FOR COOPER RIVER BRIDGES

Michael J. Abrahams, Parsons Brinckerhoff Quade & Douglas, Inc.

The Cooper River Bridge project involves replacement of the adjacent Grace Memorial and Pearman Bridges that cross the Cooper River between Charleston and Mount Pleasant, SC. The new crossing will have an overall length of approximately 3 miles (4.8 km). It includes interchanges on both sides of the river and a main span of 1546 ft (471 m) that will be the longest cable-stayed span constructed in North America, when opened to traffic in 2005. The new crossing will provide eight lanes of traffic and a 12-ft (3.7-m) wide sidewalk, replacing the now inadequate five lanes on the existing Grace and Pearman Bridges. The design was challenging due to the need to develop a cost effective concept that accommodated the high seismicity of the Charleston area, exposure to hurricanes, and the potential for ship collision. Charleston is the second busiest port on the East Coast.

The project design criteria called for a 100-year service life and required the design-build team to develop an appropriate solution. Although the design criteria indicated minimum concrete cover requirements, the type of reinforcing steel—coated or uncoated—and the type of concrete and mixture proportions were left to the design-build team. The decision to allow the team to develop their own solution for the project's corrosion protection system provided a good opportunity for the engineers, material suppliers, and contractors to work together in determining a cost-effective solution for the 100-year service life—a unique advantage of the design-build process.

While the superstructure of the cable-stayed main span, high-level approach spans, and curved interchange ramps use structural steel with a composite concrete deck, all other portions of the superstructure use precast, prestressed concrete bulb-tee girders with a composite concrete deck. The substructure uses cast-in-place reinforced concrete piers, with reinforced or post-tensioned pier caps, supported on large diameter drilled shafts. Except for the cable-stayed main span piers and abutments, there are no footings as the pier columns are supported directly by the drilled shafts.

Epoxy-coated reinforcement was not used because of its cost and concerns

about its effectiveness in a marine environment. While a service life analysis suggested some improvement if epoxy-coated reinforcement was used, the 100-year service life was obtained without the need for the epoxy coating. There was also a concern about bond of the epoxy-coated reinforcement to the concrete in the plastic hinge zones of the structure. Charleston is an area of high seismicity and the structure has several plastic hinge zones. The longer lap splices required with epoxy-coated reinforcement would have made the reinforcing bar cages in each tower lift heavier and more difficult to handle. A further concern was additional time that might have been spent in the field if disputes arose about the acceptable level of damage to the epoxy coating.

The use of high performance concrete (HPC) provided an opportunity to achieve the 100-year service life. However, silica fume was not utilized in the HPC mixes because of concerns about its cost and difficulties with finishing.⁽¹⁾

Consideration was also given to corrosion inhibitors, which would appear to be effective in extending the bridge's service life. But at the dosages rates of 2 to 4 gal/cu yd (10 to 20 L/cu m) recommended by the suppliers, and with the large quantity of concrete required, they were not cost effective. Typical pier columns have a 6 to 8 ft (1.8 to 2.4 m) diameter, and adding corrosion inhibitor to such large placements became very expensive. There was also a concern that the inclusion of a corrosion inhibitor would reduce the concrete setting times, particularly in warm weather.

A performance based approach was used to establish the requirements for the concrete. Application of Fick's law of diffusion to the design mix indicated that it would take 95 years for the chlorides to reach a corrosion initiation threshold value of 2 lb/cu yd (1.2 kg/cu m) at the level of the reinforcement. An additional 5 years for the propagation period provided the 100-years service life.⁽²⁻⁵⁾ Initial studies were conducted to establish the salinity of the Cooper River at the bridge site. Several reports document an average salinity of 25 parts per thousand. For reference, sea water is 35 parts per thousand. Salinity varies during each tide cycle and

is also affected by the volume of the river flow. Therefore, an average value was used in the diffusion analysis. The analysis also incorporated the effect of the average monthly temperature in Charleston as temperature affects the rate of diffusion.

Studies were done to evaluate the chloride contents for the adjacent Pearman and Grace Bridges. It is notable that the older Grace Bridge built in 1929 tended to have lower chloride concentrations than the newer Pearman Bridge built in 1966, as shown in Table 1.

Table 1. Typical chloride quantities at a depth of 2 in. (50 mm)

Pier	Grace	Pearman
Land	1.30 lb/yd ³	2.12 lb/yd ³
River	4.63 lb/yd ³	12.90 lb/yd ³

For portions above the splash zone, very little data are available on chloride exposure rates, so considerable judgment was required in estimating the rate to be used in the analysis. Based on the relationship between concrete chloride permeability in coulombs and its diffusion coefficient recommended by Pfeifer,⁽⁶⁾ the maximum permeabilities needed to provide a 100-year service life for various depths of cover at various locations were determined as listed in Table 2.

Local concrete suppliers were asked to develop and price concrete mixes that met the chloride permeability criteria as well as the water-cementitious materials ratio (w/cm), slump, maximum aggregate size, and volumes of concrete. The two suppliers who did develop mixes used different approaches. One supplier used slag cement for partial replacement of portland cement, while the other supplier used Class F carbon burn-out fly ash. This fly ash has a lighter color than typical fly ash as most of the carbon has been removed and the loss on ignition is typically less than 3 percent. Based on quotes submitted by the concrete suppliers, fly ash concrete was the most economical. Two typical concrete mix proportions used on the project are shown in Table 3.

As concrete with fly ash is slow in developing its impermeability, typically a year,⁽⁷⁾ it is not practical to wait that long to conduct the permeability tests, nor is it

Table 2. Maximum permeability values

Location	Concrete Cover, in.	Maximum Permeability,* coulombs
Footings	6	1400
Footings	4	500
Piers above +20 ft	3	1000

* Per AASHTO T 277 (ASTM C 1202)

practical to prevent exposure of the sub-structure elements to chlorides for one year. To address the testing issue, the sample is tested per AASHTO T 277 at 28 days after accelerated curing of the sample at 73°F (23°C) for 7 days and then at 100°F (38°C) for 21 days. This has been demonstrated to conservatively predict results of 1-year old concrete.⁽⁶⁾ In the field, the concrete is protected from ingress of chlorides for the first year by applying a sealer shortly after the forms are stripped. Thereafter, the concrete itself achieves the desired low permeability.

The use of fly ash in concrete has other advantages. The large placements, which qualify as mass concrete, require special measures to control heat build up. This is mitigated by the inclusion of fly ash, thus reducing the contractor's costs in cooling mass concrete placements.

There have been several instances during construction when the actual concrete cover was observed to be less than the

specified values including the allowable placing tolerances, which are typically 1/2 in. (13 mm). In these instances, a silane or siloxane sealer meeting the criteria in NCHRP 244 was applied.⁽⁹⁾ NCHRP 244 demonstrated that the use of those types of sealers would extend the service life for vertical surfaces. Hence, when a reduced cover does occur, the service life has been analyzed as described

above using the reduced cover but adding in the extension to service life provided by the sealer. With this approach, we have been able to demonstrate that the 100-year criterion is still satisfied.

Further Information

For further information, contact the author at abrahams@pbworld.com or 212-465-5185

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Table 3. Typical concrete mixes

Property	Footings/Splash Zone	Superstructure
Specified Strength,* psi	5000	4000
Permeability,** coulombs	206	837
Mix Proportions, lb/yd ³		
Portland Cement	400	520
Fly Ash	300	130
Fine Aggregate	1015	1125
Coarse Aggregate	2000	1875
Water	265	263
w/cm	0.38	0.40

* At 28 days **Per AASHTO T 277 (ASTM C 1202) at 28 days



SPECIFICATION CHANGES FOR HPC

Four previous issues of *HPC Bridge Views** have contained descriptions of changes needed in the AASHTO Specifications to facilitate the implementation of HPC. The proposed revisions are based on work performed as part of a pooled-fund project administered by the FHWA as Project No. DTFH61-00-C-00009.

At its June 2003 Annual Meeting, the AASHTO Subcommittee on Bridges and Structures approved a number of revisions to the LRFD Bridge Design Specifications and the LRFD Bridge Construction Specifications to facilitate the implementation of high performance concrete. Although some of the changes were initiated because of the use of high strength concrete, the changes will apply to all high performance concrete. Please note that the following is only a summary of the approved revisions. The revisions do not become the official specification articles until they are published by AASHTO in 2004, at which time, they will have the following impact:

Design Specifications

Recognize that concrete unit weight increases as concrete compressive strength increases (Table 3.5.1-1)

Extend some provisions to concrete compressive strengths greater than 10.0 KSI (Articles 5.1 and 5.4.2.1)

Facilitate specifying concrete compressive strengths at ages other than 28 days (Article 5.3)

Allow the use of ground granulated blast-furnace slag (Article C5.4.1)

Allow higher cementitious materials content for high strength concrete (Article 5.4.2.1)

Construction Specifications

Introduce two new classes of high performance concrete (Articles 8.2, 8.3.1, 8.4.1, 8.4.4, 8.6.6, 8.6.7, and 8.11.1)

Allow the use of ASTM 1157 Blended Hydraulic Cement (Article 8.3.1)

Allow a combined aggregate grading (Article 8.3.5 and a new Appendix)

Allow the use of ground granulated blast-furnace slag (Articles 8.3.7, 8.4.4, and 8.6.4.1)

Allow higher cementitious materials content for high strength concrete (Article 8.4.3)

Recognize the use of 4x8-in. cylinders (Article 8.5.7.1)

Facilitate concrete compressive strengths being specified at ages other than 28 days (Articles 8.5.7.3 and 8.5.7.5)

Require the use of match-cured cylinders for high strength concrete (Article 8.5.7.5)

Ensure proper curing of high performance concrete (Articles 8.6.4.1, 8.11.1, 8.11.3.5, 8.11.4, and 8.13.4)

In addition to the above revisions, other changes are under consideration by the AASHTO Subcommittee on Bridges and Structures and the AASHTO Subcommittee on Materials. These changes will be described in future editions of *HPC Bridge Views*.

*See *HPC Bridge Views*, Issue Nos. 23, 24, 25, and 28

HPC BRIDGE CALENDAR

October 19-22, 2003

Third International Symposium on High Performance Concrete, Orlando, FL. Jointly sponsored by FHWA and PCI. Contact John Dick, Precast/Prestressed Concrete Institute at 312-786-0300, info@pci.org or www.pci.org

May 17-18, 2004

2004 Concrete Bridge Conference on High Performance Concrete Bridges and Rapid Bridge Construction, Charlotte, NC. Jointly sponsored by FHWA and NCBC. See www.nationalconcrete-bridge.org/cbc/index.html for more information.

June 20-24, 2005

Seventh International Symposium on Utilization of High Strength/High Performance Concrete, Washington, DC. Organized by ACI. Contact Phyllis Erebor, American Concrete Institute at 248-848-3784 or phyllis.erebor@concrete.org.

HPC COMPILATION

Information about 19 HPC bridges is now available on a compact disc from FHWA. Copies may be obtained by contacting Jerry Potter, Office of Bridge Technology at phone: 202-366-4596, fax: 202-366-3077, or e-mail: jerry.potter@fhwa.dot.gov. The compilation can also be downloaded at www.nationalconcretebridge.org/hpconcrete.html.

HPC Bridge Views is published jointly by the Federal Highway Administration and the National Concrete Bridge Council. Previous issues can be viewed and downloaded at <http://www.cement.org/br/newsletters.asp>.

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For further information on High Performance Concrete, contact:

FHWA Headquarters: Jerry L. Potter, 202-366-4596; (fax) 202-366-3077; e-mail: jerry.potter@fhwa.dot.gov

AASHTO Subcommittees on -

Bridges and Structures: William N. Nickas, 850-414-4260; (fax) 850-488-6352; e-mail: william.nickas@dot.state.fl.us

Materials: John B. Volker, 608-246-7930; (fax) 608-246-4669; e-mail: john.volker@dot.state.wi.us

Construction: Gene R. Wortham, 208-334-8426; (fax) 208-334-4440; e-mail: gwortham@itd.state.id.us

NCBC: Shri Bhidé, PCA, 847-972-9100; (fax) 847-972-9101; e-mail: sbhide@cement.org; website: www.nationalconcretebridge.org

Editorial Committee:

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