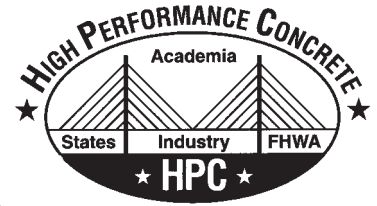




Bridge Views



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HPC FOR THE SACRAMENTO RIVER BRIDGE

Basem H. Alsamman and Mark A. Darnall, California Department of Transportation

The California Department of Transportation (Caltrans) is currently building a new four-span HPC precast, prestressed, post-tensioned bridge to carry Interstate 5 across the Sacramento River in Northern California. This bridge with a length of 614 ft (187 m), uses spliced Caltrans bulb-tee girders with a cast-in-place reinforced concrete deck. It is the first bridge of this type to be built in California.

The bridge was designed subject to several stringent constraints that included (1) minimizing the foundation footprint of the bridge in the Sacramento River, (2) reducing substructure influence to the river flow and minimizing channel disruption during construction, and (3) only working within the waterway from May 1 to October 15 of each year with all falsework removed from the river by October 15.

Due to these constraints, the minimum acceptable span length was 148 ft (45 m), and the maximum permissible structural depth was 78 in. (1.98 m) to provide the required freeboard. Therefore, Caltrans engineers opted for continuous high performance concrete (HPC) precast, prestressed, post-tensioned, spliced bulb-tee girders.

Actual span lengths were 153 ft (46.6 m) for the end spans and 154.2 ft (47.0 m) for the center spans. Each span incorporated thirteen 70-in. (1.77-m) deep girders at 10.25-ft (3.13-m) centers. Web thickness of the girders was 8 in. (200 mm). The spliced girder in each span consisted of three prestressed concrete segments approximately 49 ft (15 m) long, erected on temporary supports and spliced together with a 2-ft (610-mm) long HPC closure pour. The reinforcement between girder segments was made continuous and the segments were post-tensioned longitudinally for the complete length of the bridge. The first-stage longitudinal post-tensioning and transverse post-tensioning at pier diaphragms made the girders self-supporting. This enabled the removal of falsework and access pad from the river. The second stage post-tensioning was performed after the deck was cast.

The specified concrete compressive strength for the girders, girder closure pours, intermediate diaphragms, and pier cap diaphragms was 8700 psi (60 MPa). An additional 7 days beyond the normal 28 days was allowed in the special provisions for strength development. However, due to time constraints, it was necessary to achieve the specified strength for the cast-in-place concrete within 10 days. Consequently, a concrete mix with a water-cementitious materials ratio of 0.33 and a high-range water reducing admixture was used. The average 10- and 35-day strengths were approximately 10,000 psi (69 MPa) and 11,000 psi (75 MPa) respectively. At the time of its design, this was one of the highest concrete strengths used by Caltrans.

Phased construction of this bridge started in April 2001 and is expected to be completed in August 2004. It has a projected total cost of \$16.1 million. In the meantime, thousands of motorists who traverse this route daily will hardly have noticed the work in progress. Clearly, the design and construction incorporating HPC precast, prestressed, spliced bulb-tee girders for the superstructure have provided an outstanding solution to a complex engineering problem and have addressed the needs and concerns of the traveling public and other partners.



HPC spliced bulb-tee girders were used to satisfy the project constraints.

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MASS CONCRETE IN THE CARQUINEZ BRIDGE

Ric Maggenti, California Department of Transportation



Cementitious materials in the tower footing concrete included 35 percent fly ash to minimize the heat of hydration.

The new Carquinez Bridge is a 3000-ft (1-km) long suspension bridge spanning the Carquinez Strait at the north end of San Francisco Bay. The cables of the bridge are supported on concrete towers that rise 425 ft (130 m) above the water. The cables are anchored by concrete blocks with thicknesses up to 50 ft (15 m). Approximately 60,000 cu yd (45,000 cu m) of mass concrete are used in the footings under the two towers and in the four concrete anchors. Although a series of placements was used, each placement was large enough to be considered mass concrete.

Concrete Mix

The concrete mix was designed so that the heat of hydration would not be detrimental to the finished concrete. Too high a temperature or temperature gradient can compromise durability and strength due to thermal cracking, self desiccation, or chemical alteration of the paste resulting in possible delayed or secondary ettringite formation.

Measures taken to cope with the heat consisted of designing the mixes to minimize the heat generated while still achieving the specified strength, selecting concrete placement temperatures, and placing thermal blankets to control the temperature gradient. The specification included a maximum concrete tempera-

ture of 158°F (70°C) and a maximum temperature gradient of 36°F (20°C). The specified concrete compressive strength for the footings and anchors, except the temporary access chambers, was 3500 psi (24 MPa) at 56 days. The chambers, which were later filled with concrete, provided space for the spinning process of the wire strands to form the main cables. The 2600 cu yd (2000 cu m) of concrete used to fill these temporary access chambers had a specified concrete compressive strength of 1000 psi (6.9 MPa) at 28 days.

The required concrete properties were achieved by minimizing the cementitious materials content and replacing portland cement with fly ash. The concrete for the footing and anchor blocks contained 560 lb/cu yd (332 kg/cu m) of cementitious materials of which 35 percent was Class F fly ash. The concrete fill for the access chamber contained 376 lb/cu yd (223 kg/cu m) of cementitious materials of which 50 percent was Class F fly ash. The concrete fill temperature never rose above 91°F (33°C).

The actual field temperatures of the in-place concrete were predicted to a reasonable degree of accuracy by the contractor for over 40 placements. One comparison of measured peak temperature and temperature near the surface with predicted values was reported to be within 4°F (2°C). With this accuracy, it was shown that concrete temperature can be controlled to limit cracking. The mass concretes were, with a few exceptions, relatively crack free. Design and specifications engineers visiting the project saw that large concrete elements could be placed with a limited amount of cracking. Large areas of the concrete surface were visibly crack free.

One notable exception to crack-free concrete was the second lift of the south anchorage. The weather on the day of casting was dry and windy and the air temperature reached 104°F (40°C). Before initial set of the concrete, an extensive network of plastic shrinkage cracks developed on the surface because no curing was applied. Thermal blankets were placed five hours after the last concrete was placed. Many of the cracks were

deeper than 14 in. (350 mm) and a few cracks extended into the entire depth of the placement. The cracks were treated with a high molecular weight methacrylate. Cores, some almost 10 ft (3 m) long, were taken through a random selection of cracks to ensure penetration and bond. Tensile splitting tests were conducted on several cores to verify an adequate repair. One area of the same placement was accidentally water cured because of a leaking cofferdam and remained crack free.

Use of Fly Ash

Prior to 1995, pozzolans, in the form of fly ash or natural pozzolans added at the batch plants, were limited by the California Department of Transportation (Caltrans) Standard Specifications to 15 percent of the total cementitious materials. This has changed and most concrete used by Caltrans now requires 25 percent fly ash. The use of fly ash to control heat in mass concrete has steadily increased. The foundation elements for structures built in Oakland in the early 1990s started out using 15 percent fly ash. Temperature monitoring showed heat to cause potential problems so up to 40 percent fly ash was substituted.

The use of fly ash at 35 and 50 percent of the total cementitious materials proved to be vital to control heat in the Carquinez Bridge project. The measured strengths for the structural concrete and the chamber fill exceeded the specified values. The concrete temperatures did not reach values that might result in delayed or secondary ettringite formation or gradients that would induce excess thermal stresses. The use of internal cooling pipes was not required. The mass concrete for this suspension bridge should serve its structural purposes as the mix designs accomplished their intended results.

Further Information

For further information, contact the author at ric_maggenti@dot.ca.gov or 916-227-8755 or see the following reference: Maggenti, R. and Haylock, G., "California's New Carquinez Bridge," Concrete International, Vol. 25, No. 2, February 2003, pp. 56-60.

EVOLUTION OF HPC SPECIFICATIONS IN TEXAS

Kevin R. Pruski, William R. Cox, and Mary Lou Ralls, Texas Department of Transportation

The Texas Department of Transportation (TxDOT) specifications for high performance concrete (HPC) have evolved through TxDOT's experience. The first two HPC projects in Texas in the 1990s were guided by researchers at the University of Texas, Austin, in conjunction with demonstration projects sponsored by the Federal Highway Administration (FHWA). Although these two projects focused on high-strength HPC, the cast-in-place concrete deck on one of the two adjacent bridges in each project was constructed with normal strength HPC. Concrete strengths of 9000 psi (62 MPa) and higher with good durability characteristics are readily achieved in pretensioned concrete beams in Texas. Now, the priority in Texas is to improve the durability of normal strength cast-in-place concrete. Therefore, subsequent HPC bridge projects have focused on normal strength cast-in-place HPC. This article discusses activities that address this priority.

For early HPC bridge projects in Texas after the first two, the contractor was expected to obtain durable HPC by adherence to TxDOT's standard specifications with only slight modification and additional performance-related requirements. This approach relied on the contractor's knowledge and experience to satisfy contract requirements. However, bridge projects following this approach demonstrated contractors' inexperience at designing concrete to meet performance-based durability requirements. Consequently, non-compliance with the new performance-related criteria as well as higher concrete costs resulted.

Specifications were, therefore, adjusted to shift responsibility to TxDOT to specify more durable concrete. The first two HPC bridge projects in Texas, HPC bridge projects in other states, and the results of various HPC research projects convinced TxDOT that inclusion of supplementary cementitious materials (SCM) makes concrete more durable. With this knowledge, TxDOT added prescriptive requirements to include specific amounts of SCM in concrete that was required to have increased durability; this concrete includes the designation "HPC" in its bid

item. By forcing the use of SCM, TxDOT anticipates that Texas contractors, materials suppliers, and engineers will develop the experience to obtain HPC with the needed durability requirements.

TxDOT HPC specifications now require the contractor to develop strength-versus-time curves for the concrete with strengths measured at 4, 7, 28, and 56 days for mix design approval. This was introduced to address concerns about prescriptively specifying SCM when the materials supplier and the contractor lack experience with the effects of SCM on strength gain of concrete.

Fly ash or ground granulated blast-furnace slag can slow strength gain, especially in cooler weather, and requiring the contractor to plot this curve helps synchronize the concrete mix with the construction schedule. Additional concrete samples are taken and sent to the central laboratory for AASHTO T 277 rapid chloride permeability (RCP) tests and AASHTO T 259 salt ponding tests.

Sample Bridge Project

TxDOT recently replaced two deteriorated concrete bridges in the Lubbock District in the Texas panhandle. HPC was specified because of the significant use of deicing chemicals related to the 70 annual freeze-thaw cycles.

Ordinarily, the Class C concrete used for a bridge substructure in TxDOT projects has a minimum specified compressive strength of 3600 psi (25 MPa) at 28 days and a maximum water-cementitious materials (w/cm) ratio of 0.53. The Class C (HPC) provision required replacing 4 percent of the cement with silica fume and 26 percent with Class F fly ash, and limiting the w/cm ratio to a maximum of 0.47. The Lubbock District has only recently started using fly ash routinely, and it had never used silica fume. Although the average rapid chloride permeability (RCP) was not specified, the average measured value was 676 coulombs at 56 days.

TxDOT specified Class S (HPC) concrete for the bridge deck. Class S concrete normally has a minimum compressive strength of 4000 psi (28 MPa) at 28 days

and a maximum w/cm ratio of 0.44. The Class S (HPC) provision required replacing 30 percent of the cement with Class F fly ash. There were concerns about attaining the 4000 psi (28 MPa) compressive strength at 28 days due to slower strength gain from the Class F fly ash, especially in cold weather. Therefore, the 28-day strength requirement was lowered to 3000 psi (21 MPa) but a requirement of 4000 psi (28 MPa) at 56 days was added. Although 3000 psi (21 MPa) is sufficient to resist design loads on adjacent box beam structures, 4000 psi (28 MPa) was required to be consistent with the Standard Specifications. The average RCP test value for the HPC was 1057 coulombs at 56 days compared to 4000 coulombs for concrete without fly ash.

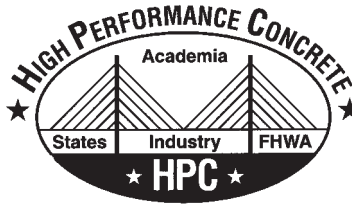
The Future of HPC Use in Texas

TxDOT continues to specify normal strength HPC to increase the durability of its bridges. The use of prescriptive HPC specifications has allowed contractors and fabricators to gain experience with the production and placement of concrete containing various types of SCM. TxDOT has now begun allowing the contractor the option of using the prescribed mix or another mix that meets the durability performance requirements. Ultimately, the department is moving toward the full implementation of performance-based durability specifications.

Concrete Mix Proportions for Lubbock Project

Material	Quantities, lb/yd ³	
	Class C (HPC)	Class S (HPC)
Portland Cement*	367	397
Fly Ash, Class F	137	181
Silica Fume	25	-
Fine Aggregate	1241	1174
Course Aggregate	1854	1854
Water	250	260
w/cm	0.47	0.45

* Type I/II



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

Question:

What can be done to reduce cracking in a concrete bridge deck?

Answer:

Research over the past 30 years has addressed the causes of cracking in bridge decks. The results of the studies provide guidance on modifications in materials, construction techniques, and design that will reduce the amount of cracking. Concrete shrinkage is a major cause of cracking in bridge decks. Shrinkage can be reduced by decreasing the volume of water and cement paste (cement and water) in the concrete. Workability can be enhanced even with reduced paste contents by using water-reducing and high-range water reducing admixtures. Optimized aggregate gradations can also help. With careful attention to the air-void system, shrinkage-reducing admixtures can also play a role. Increased compressive strength often associated with high performance concrete increases cracking. In studies of bridge decks in Kansas,^(1,2) monolithic decks with average measured compressive strengths of 4500 psi (31 MPa) exhibited average crack densities of only 0.05 ft/ft² (0.16 m/m²) compared with 0.15 ft/ft² (0.50 m/m²) for decks with average measured strengths of 6500 psi (45 MPa).

During construction, settlement and plastic shrinkage cracks serve as locations for subsequent drying shrinkage cracks to form as the concrete ages. Settlement cracks that form over transverse reinforcing bars can be reduced using increased concrete cover, decreased bar size, and lower concrete slump. Plastic shrinkage cracks increase as the rate of evaporation from the concrete surface increases. Even when plastic shrinkage cracking is not specifically observed, conditions associated with high evaporation rates correlate with increased total cracking in a completed deck. Techniques such as windbreaks and fogging have had a positive impact, as has thorough curing. An excellent solution involves curing immediately behind the finishing equipment. Longer curing periods are often hotly debated because of the effect on construction time. However, they have the added advantage of increasing the amount of water that combines chemically with the cement. This reduces potential evaporation losses and associated drying shrinkage.

Data from the field demonstrates that bridges can be constructed that have far less

cracking than occurs on many bridges today. A project is currently underway, through the University of Kansas, with the goal of constructing 20 bridge decks in several states using the best practices to achieve near crack-free bridge decks.

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2. Schmitt, T. R. and Darwin, D., "Effect of Material Properties on Cracking in Bridge Decks," Journal of Bridge Engineering, ASCE, Vol. 4, No. 1, Feb. 1999, pp. 8-13.

Answer contributed by David Darwin of the University of Kansas. He may be contacted at daved@ku.edu or 785-864-3827.

HPC BRIDGE CALENDAR

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 Thomas H. Adams, American Concrete Institute at 248-848-3742 or thomas.adams@concrete.org.

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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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