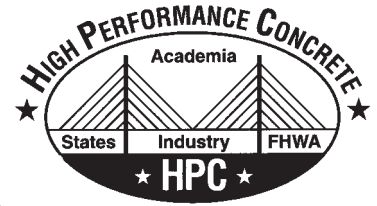




# Bridge Views



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## HPC IN ALASKA

Elmer E. Marx, Alaska Department of Transportation & Public Facilities

Since the early 1970s, the Alaska Department of Transportation & Public Facilities has been building adjacent, precast, prestressed concrete deck bulb-tee girder bridges using high strength concrete (HSC). Most of these bridges are jointless utilizing either integral or semi-integral abutments. Typical girder spans range from 85 ft (26 m) for a 42-in. (1.065-m) deep section to 145 ft (44.2 m) for a 66-in. (1.675-m) deep section. Girder spacing is usually between 6.6 ft (2 m) and 8.2 ft (2.5 m).

Initially, design concrete strengths were 5500 psi (38 MPa) at release and 6500 psi (45 MPa) at 28 days. As the years passed, the specified concrete strength increased. It was presumed that improved durability would be one benefit of the increased concrete strength and no performance requirements such as chloride permeability, abrasion resistance, or freeze-thaw resistance were specified. Time has thus far proven the assumption true.

By the late 1990s, concrete release strengths of 7500 psi (52 MPa) and 28-day strengths of 8000 psi (55 MPa) were specified. The need to consistently obtain the high release strength in a short period of time, such as 18 hours, resulted in actual 28-day concrete strengths of 10,000 psi (70 MPa) or higher. As the specified concrete release strength has increased, fabricators have maintained their daily production cycle and no significant cost increase has occurred.

HSC has permitted an increase in the girder prestressing force. The combination of high strength concrete and increased prestressing force has resulted in both longer girder spans and wider girder spacings. By increasing the girder span length, Alaska's most cost-effective bridge system can be used more often. By increasing the girder spacing, the cost of a typical bridge has been reduced by eliminating girder lines.

With the deck bulb-tee girder, bridge construction time is significantly reduced compared to conventional cast-in-place (CIP) deck systems. A typical highway overpass is often built in less than

three months from the mobilization of equipment to the installation of the bridge railing. This is particularly important in Alaska where the construction season is short and CIP concrete is not readily available outside the major population centers.

A disadvantage of the deck girder system is its heavier weight compared to the standard bulb-tee girder. Transportation and erection equipment requirements are, therefore, increased. However, the reduction in the total number of girder lines has reduced the number of girders that must be transported to the site.

Because the concrete deck and girder are cast concurrently, the bridge deck concrete strength and durability are of exceptional quality not otherwise available when using CIP concrete. The deck is an integral component of the flexural system and is designed to remain in compression in the longitudinal direction under all service load combinations. In addition, a full width waterproofing membrane and asphalt overlay have generally been used on bridges built since the early 1980s. Consequently, there has been almost no girder-related maintenance required on the 212 bridges of this style built since 1973. Although traffic volumes are low compared to other states, Alaska has more severe environmental conditions. Studded tire and chain usage is high and may occur for up to six months per year. Deicing chemicals are used in much of the state, particularly in the corrosive maritime regions where snowfall is heavy and the number of freeze-thaw cycles is high.

Both the long- and short-term costs of the deck bulb-tee girder bridge have been unbeatable for the Alaska market. As the usage and quality of HPC continue to improve, Alaska will continue to lengthen bridge spans and increase girder spacings. Alaska's most cost-effective structure will become even more economical.

## Further Information

For more information, contact the author at (907) 465-6941 or [elmer\\_marx@dot.state.ak.us](mailto:elmer_marx@dot.state.ak.us).

# HPC DECK PANELS FOR JACQUES CARTIER BRIDGE

Adel R. Zaki and Bernard Breault, SNC-Lavalin Inc.

The reconstruction of the Jacques Cartier Bridge in Montreal, Canada, involved more than 645,800 sq ft (60,000 sq m) of bridge deck and made extensive use of precast, prestressed, high performance concrete (HPC) deck panels. This case study demonstrates a good example of the benefits of using a precast deck replacement method to rapidly reconstruct a highly durable deck while maintaining normal rush hour traffic.

The 1.7-mile (2.7-km) long bridge with five traffic lanes carries more than 43 million vehicles every year, making it one of the busiest bridges in North America when considering traffic density per lane. After more than 70 years of operation, the concrete deck slab, support beams, and many other bridge deck components had suffered severe damage and had thus reached their useful service life. In-depth investigations confirmed that major reconstruction of the deck was required.

The new bridge deck is made of precast HPC panels, which form a modular multi-stem integral deck system that, after being installed on the bridge, is transversely and longitudinally post-tensioned to provide high durability. Specified concrete compressive strength was 8700 psi (60 MPa) at 28 days.

The precast concrete panels were designed to suit the various structural systems of the existing superstructure along the bridge. The new deck structural configuration was mostly driven by construction constraints and by the existing steel bridge components.

The new deck for the north and south approach spans consists of a series of deck spans, typically 24.34 ft (7.42 m) long. Each span is made up of four precast, prestressed concrete panels installed side-by-side. Each panel has a 7-in. (180-mm) thick slab and incorporates three integral stems with depths ranging from 19.6 to 31.5 in. (500 to 800 mm). The stems are reinforced with four 0.6-in. (15.2-mm) diameter draped prestressing strands. The concrete barriers were also integrated with the panels. Following the installation of a specific number of panels on the existing floor beams, the transverse and longitudinal post-tensioning was applied. The deck panels for the approach spans represent 67



High performance concrete deck panels were used for rapid bridge deck construction.

percent of the entire surface area that was reconstructed. For the main span, similar precast panels were used but having only two stems per panel.

Because of the size of the project and the large number of precast deck panels to be installed during the two construction seasons (April to October of 2001 and 2002), it was deemed advantageous to construct a temporary precasting plant near the south approach of the bridge.

On the bridge, the existing deck was removed by saw cutting it into sections having similar dimensions to the new panels. Existing deck sections, which included the slab, steel stringers, barriers, and railings, were removed using two self-propelled telescopic cranes placed at opposite ends of a panel. During the same lifting sequence and using the same cranes, the new panels, each weighing between 22 and 42 tons (200 and 375 kN), were lifted from the transport truck and lowered onto new bearing assemblies, which had been installed by other crews working in advance during the day. Joints between panels were 1.56 in. (40 mm) wide and were filled using a rapid setting mortar, designed to provide a 3600 psi (25 MPa) compressive strength at three hours after mixing and prior to post-tensioning.

In this project, the use of high performance concrete combined with a precast, prestressed, post-tensioned, modular multi-stem integral slab and girder system reflects the state-of-the-art in regards to bridge deck reconstruction where durability, speed of construction, structural efficiency,

life-cycle costs, and impact to users are considered.

## Further Information

For further information, contact the first author at adel.zaki@snc-lavalin.com or 514-393-1000.

## Concrete Mix Proportions

Material	Quantities	
	per yd <sup>3</sup>	per m <sup>3</sup>
Cement <sup>(1)</sup>	758 lb	450 kg
Fine Aggregate	1281 lb	760 kg
Course Aggregate	1669 lb	990 kg
Water	228 lb	135 kg
Water Reducer	116 fl oz	4500 mL
Set Retarder	26 fl oz	1000 mL
Air Entrainment	10 fl oz	380 mL
w/cm ratio	0.30	

<sup>(1)</sup>Type 10SF

## Concrete Properties

Slump	8 in.	200 mm
Air Content	5 %	5 %
Concrete Strength		
at 16 hours	4650 psi	32MPa
at 7 days	8000 psi	55 MPa
at 28 days	10,440 psi	72 MPa

# THE AIR VOID ANALYZER

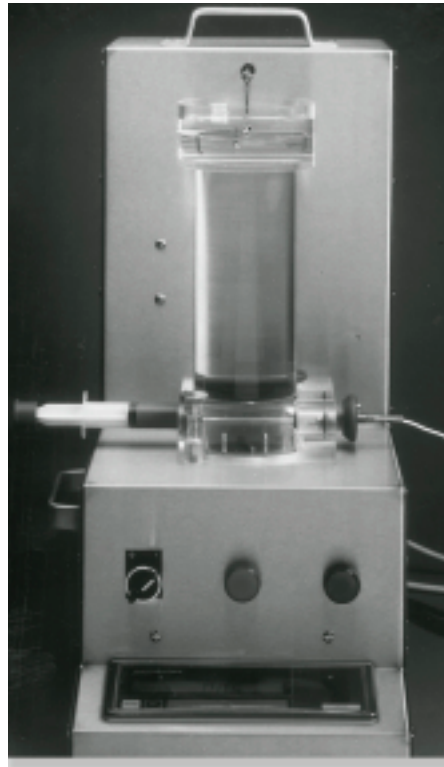
Gary Crawford, Federal Highway Administration and Leif Wathne, Soil and Land Use Technology

For over 50 years, researchers have known that concrete is susceptible to freeze-thaw deterioration unless a system of air bubbles is present in the concrete to protect it. The size and spacing of these air bubbles or voids are important. If the air voids are too large or too far apart, water cannot reach an air void to relieve expansive pressure as the water freezes. As the concrete continues to cycle through freezing and thawing, micro-cracking occurs and eventually the concrete sustains significant damage.

The most common methods used to measure air content in fresh concrete today are the pressure and volumetric methods. Both methods measure the volume of air only and not the size or spacing between the voids known as the spacing factor. Nevertheless, these methods have worked well in the past, since the volume of air has been a successful surrogate measure of the spacing factor, and indirectly the concrete freeze-thaw durability. But, the ingredients and processes used to make concrete have changed over the years, and the traditional relationship between volume of air, air-void system, and freeze-thaw durability may no longer be valid. Consequently, methods are needed to measure size and spacing of the air-voids in the fresh concrete. The Air Void Analyzer (AVA) was developed in Europe during the 1980s to meet this need.

## How does the AVA work?

A 0.68 fl oz (20 mL) mortar sample is obtained from the fresh concrete using a vibrating wire cage and a syringe. The sample is then injected into the bottom of a transparent cylinder or riser column filled at its base with a viscous liquid and topped with water. The sample is then stirred for 30 seconds releasing the air bubbles and allowing them to rise through the liquids. The rate of rise depends on the bubble size. The air bubbles are collected under a submerged dish attached to a balance. The change in suspended mass of the dish is recorded for 25 minutes after the stirring period. A computer algorithm uses the weight change with time to calculate the void size distribution, total air volume, spacing factor, and specific surface.



The Air Void Analyzer. (Photo courtesy of Germann Instruments)

## How does the AVA compare to other tests?

The Federal Highway Administration (FHWA) first purchased an AVA test unit in 1993. The equipment was used on a variety of projects throughout the United States. Results showed that the spacing factor was consistent with results obtained on hardened concrete using ASTM C 457, but the AVA tended to report smaller void sizes when compared to the ASTM C 457 examination.

After upgrading the equipment in 1999, the FHWA has used the AVA on a variety of field projects in nine different states. Projects have included pavements, precast sheet piles, foundation elements, and bridge decks. For six of these projects, accompanying hardened air content tests were also performed.

Data show that the difference in observed spacing factors between the AVA and ASTM C 457 test is relatively small, and fall well within the range of averages for between-laboratory precision for two test results reported in ASTM C 457. More importantly, in 9 of the 14 cases where the concrete met the total air vol-

ume requirements based on pressure meter tests (ASTM C 231), it did not meet generally accepted durability criteria limits based both on AVA and ASTM C 457 spacing factor results. This means that in approximately 65 percent of the cases where a deficient concrete was delivered, it was deemed adequate by the current test practice of total air volume. Implementing the use of the AVA can, therefore, significantly improve the quality of concrete placed in the United States from a freeze-thaw perspective.

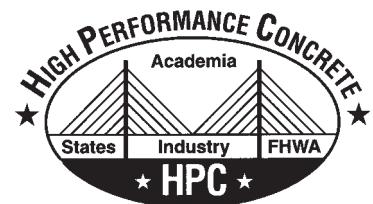
## What's next?

Several state highway agencies are evaluating the use of the AVA and the American Association of State Highway and Transportation Officials (AASHTO) included the AVA as a focus technology in its 2002 Technology Implementation Group (TIG) program. The TIG has provided leadership and technical assistance to promote implementation of the AVA technology by the states.

The primary benefit of the AVA is that it measures the air content, spacing factor, and specific surface of fresh concrete in about 30 minutes, allowing for the timely detection of concrete that will not be resistant to freeze-thaw cycles. Adjustments can then be made to minimize the delivery of fresh concrete with a deficient air-void structure. The FHWA field experience confirms the ability of the AVA to detect substandard air-void systems with accuracy comparable to that of ASTM C 457 results for hardened concrete.

## Further Information

For more information on the TIG's activities go to [www.aashtotig.org](http://www.aashtotig.org). For other details, contact the authors [gary.crawford@fhwa.dot.gov](mailto:gary.crawford@fhwa.dot.gov) or 202-366-1286 or [leif.wathne@fhwa.dot.gov](mailto:leif.wathne@fhwa.dot.gov) or 202-366-1335.



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## Question

When using match curing to determine the release strength of prestressed concrete beams, where should the temperature sensor in the beam be placed?

## Answer

Match curing is a procedure for curing concrete cylinders at the same temperature as that monitored at a specific location in a concrete member. Consequently, the compressive strength of the cylinder more accurately represents the in-place concrete strength of the member. The method is particularly useful for determining the concrete compressive strength at early ages in the fabrication of prestressed concrete products and is permitted by some state departments of transportation.

The temperature sensor in the beam needs to be placed at the most critical location for strength development. This generally means the location with the slowest and least concrete temperature rise. This location depends on the beam cross section and method of curing. For most I-beams and bulb-tee beams that are cured without the use of external heat, the coolest temperature is likely to be in the top flange since it has the highest ratio of surface area to volume compared to the web or bottom flange and, therefore, cools more rapidly. It is also cooler on the surface than on the inside. For beams where the predominant amount of heat is supplied externally to the beam, the least temperature rise in most I-beams and bulb-tee beams is likely to be in the middle of the bottom flange since this is furthest from the heat source. However, if there is also heat of hydration present, the center may not be the coolest location. Consequently, it is recommended that the producer should determine the critical location within the cross section based on the plant procedures. To avoid subsequent disputes, the location should be approved by the Engineer.

The critical location along the length of the beam should also be defined. Generally, the last concrete placed in the forms has the coolest temperature until peak temperatures are reached. It has also been observed that the ends of beams closest to the ends of the precasting line may not achieve as high a temperature as at other locations along the line. The prestressing strands projecting from the ends of the beam act as conductors removing heat from the beam. Therefore, a location near the end of a beam closest to the end of the precasting bed where the last concrete was placed may be more critical than other locations along the bed. Irrespective of the selected location, it is important to measure the concrete temperature and not the temperature of the surrounding air, steam, or formwork.

## HPC BRIDGE CALENDAR

### September 8-10, 2004

The Second National Prefabricated Bridge Elements and Systems Workshop, New Brunswick, NJ. Sponsored by NJDOT, AASHTO, and FHWA. Contact Helene.Cook@fhwa.dot.gov for more information.

### September 13-15, 2004

International Symposium on Ultra High Performance Concrete, Kassel, Germany. Organized by the University of Kassel. See [www.uni-kassel.de/uhpc2004](http://www.uni-kassel.de/uhpc2004) for more information.

### October 17-20, 2004

PCI National Bridge Conference — Bridges for Life,™ Atlanta, GA. See [www.pci.org](http://www.pci.org) for more information.

### June 20-24, 2005

Seventh International Symposium on Utilization of High Strength/High Performance Concrete, Washington, DC. Organized by ACI. See [www.concrete.org](http://www.concrete.org) and click on Events and International Conferences for more information.



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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](mailto: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](mailto: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](mailto:john.volker@dot.state.wi.us)

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

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

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