First Use of Lightweight High-Performance Concrete Beams in Virginia

by C. Ozyildirim, T. Cousins and J. Gomez

Synopsis: An experimental program was planned and executed to demonstrate the feasibility of using lightweight high performance concrete (LWHPC) bridge beams and decks. The prestressed American Association of State Highway and Transportation Officials (AASHTO) beams were designed for a minimum 28-day compressive strength of 8,000 psi (55 MPa) and the deck concrete for 4,000 psi (27.6 MPa). The maximum permeability was 1500 coulombs for the beam and 2500 coulombs for the deck concrete. The density was less than 120 lb/ft^3 (1920 kg/m^3). This program was a necessary prelude to an LWHPC demonstration bridge built over the Chickahominy River on Route 106 at the Charles City County–New Kent County line in Virginia.

Keywords: development length; flexural strength; high-performance concrete; lightweight concrete; low-permeability concrete; prestressed concrete bridge beams; structural testing; transfer length

Biographical Sketches:

Dr. Celik Ozyildirim is a Principal Research Scientist with the Virginia Transportation Research Council in Charlottesville. He has more than 30 years of research experience in concrete materials.

Dr. Tommy Cousins is an Associate Professor at the Virginia Polytechnic Institute & State University in Blacksburg. His area of expertise is prestressed concrete, with a particular emphasis in transportation structures.

Dr. Jose Gomez is an Associate Director at the Virginia Transportation Research Council in Charlottesville. His areas of interest are bridge design and behavior.

INTRODUCTION

High performance concretes (HPC) are characteristically higher in strength and more durable than their regular counterparts because of their denser cementitious matrix as a result of the lower water–cementitious materials ratios (w/cm) and mineral admixtures in their makeup. These attributes contribute directly to desired structural and economic efficiencies in the design and construction of highway bridges. Recently HPC was used successfully in bridge design and construction [1,2]; however, the bridges were built with normal weight concretes with a density of generally 145 lb/ft^3 (2320 kg/m^3). Encouraged by the successful implementation of HPC technology in demonstration bridges built in Virginia by the Virginia Department of Transportation (VDOT), the Virginia Transportation Research Council (VTRC) is seeking to employ this technology further by using lighter weight concretes. In this effort, lightweight aggregates (either lightweight coarse aggregates alone or lightweight coarse combined with lightweight fine aggregates) have been used. To produce lightweight HPC (LWHPC) with strengths exceeding 8,000 psi (55 MPa), high-quality lightweight coarse aggregates are needed. However, there has been uncertainty regarding the tensile strength, modulus of elasticity, shrinkage, and creep in both lightweight concrete (LWC) and LWHPC. These material properties directly affect the transfer length, development length, flexural strength, and prestress losses of prestressed beams.

To that end, an experimental program was planned and executed to demonstrate the feasibility of producing prestressed AASHTO beams made of LWHPC with a density of about 120 lb/ft^3 (1920 kg/m^3) and a 28-day compressive strength of 8,000 psi (55 MPa). Material properties were determined and structural testing of beams was conducted to evaluate the effects of using LWHPC on transfer length, development length, flexural strength, and prestress losses. This paper describes the experimental program conducted to determine the feasibility of using LWHPC on a bridge structure in Virginia. The planned result of this testing program was an LWHPC demonstration bridge to be built over the Chickahominy River on Route 106 in Virginia.

METHODS AND RESULTS

Five bridge beams were fabricated for the experimental program. Three were fabricated for structural testing and consisted of two AASHTO Type II beams fabricated with LWHPC and a third Type II beam fabricated with normal weight HPC (NWHPC). All three beams were later cast with a NWHPC composite deck section to ensure full composite action. The third beam was intended to serve as a control and comparison specimen. The other two beams were AASHTO Type IV beams made of LWHPC. They were identical in design with the bridge beams to be used in the bridge to be constructed on Route 106. The beams were tested for transfer length, development length, flexural strength, and prestress loss. Prestress losses were also calculated using the ACI and PCI models that incorporate lightweight structural concretes.

For the beams, the specified 28-day minimum compressive strength was 8,000 psi (55 MPa), with a release strength of 5,600 psi (39 MPa). Grade 270 low-relaxation prestressing strands 0.5 in (13 mm) in diameter were used. The maximum specified density of LWHPC for the beams was 120 lb/ft^3 (1920 kg/m^3), and the maximum permeability (ASTM C 1202) was 1500 coulombs. The beams were steam cured to obtain high early-release strengths. Specimens from the beams were kept in the recesses of the beam forms during steam curing and were then air-dried.

Concrete properties were determined for the freshly mixed and hardened states. Slump (ASTM C 143), air content (ASTM C 173), and density (ASTM C 138) were determined in the fresh state. Compressive strength (AASHTO T22), elastic modulus (ASTM C 69), and splitting tensile strength (ASTM C496) were determined in the hardened state.

Materials Evaluation

The mixture proportions for the lightweight and normal weight concretes are listed in Table 1. The cementitious material was a combination of finely ground Type II cement and Grade 120 slag. The coarse aggregate used for the control mixture was No. 68 granite with an absorption of 0.6 percent, a specific gravity of 2.98, and a bulk density of 108.4 lb/ft^3 (1737 kg/m^3). The lightweight coarse aggregate was ¾ in to No. 4 (19.0 mm to 4.75 mm) expanded slate with an absorption of 5 percent, a specific gravity of 1.47, and a bulk density of 49 lb/ft^3 (785 kg/m^3). The fine aggregate was natural sand with an absorption of 1.4 percent, a specific gravity of 2.61, and a fineness modulus of 2.60. The admixtures were a commercially available air-entraining admixture (AEA); a vinsol rosin complying with the requirements of ASTM C 260; a water-reducing admixture (WRA); a lignin complying with the requirements of ASTM C 494, Type A; and a high-range water-reducing admixture (HRWRA), which is a polycarboxylate complying with the requirements of ASTM C 494, Type F.

The characteristics of the freshly mixed concrete are given in Table 2. The air contents were all within the specified 5.5 ± 1.5 percent range. The slump values for the LWHPC were higher than those for the NWHPC, and both concretes were workable.

The densities for the LWHPC were lower than expected. The lower density was attributed to the high air content and excess water in the mixture. The air contents for the hardened state given in Table 3 were higher than those for the freshly mixed concrete and the design air contents. The conditioning of the lightweight aggregate may have been the cause of the extra water; the aggregates were prewet using a sprinkler system. The sprinkler system may not be effective in controlling and maintaining uniform moisture content within and at the surface of the aggregate. The extra water would increase the w/cm, thus decreasing the density and compressive strength, as discussed later.

The hardened concrete properties are summarized in Table 4. At 28 days, the NWHPC was close to the specified compressive strength, but the strength for the LWHPC was much lower. At 6 months, the NWHPC exceeded the specified strength, but the LWHPC did not. Excess water could have caused the lower compressive strength. In fact, the highest average strength recorded at the time of testing (6 months from placement) was 6,910 psi (48 MPa), with an actual average density of 114 lb/ft^3 (1830 kg/m^3). At transfer, the strength of the LWPHC beams was 4,780 psi (33 MPa), 15 percent lower than the specified strength of 5,600 psi (39 MPa). It is suspected that an undesirably high w/cm for the mixture and high air contents resulted in its lower strength. The elastic modulus values of the LWHPC were lower than for the NWHPC, as expected.

Structural Evaluation

Two types of test beams were used. The first type was an AASHTO Type II beam with a normal weight composite concrete deck 48 in x 8 in (1220 mm x 203 mm). Each Type II beam was prestressed with eight strands. Six strands were straight throughout the beam, and two were harped 14 ft (4.27 m) from each end of the beam. These beams were 36 ft (11 m) in length. The second type of test beam was a Type IV beam identical with the beams to be used in the Chickahominy Bridge. These beams were prestressed with 38 strands, with 30 straight strands and 8 strands harped 32 ft 8 in (9.96 m) from each end of the beam. These beams were 84 ft (25.6 m) in length.

A composite test specimen (Type II beam with cast-in-place concrete deck) was chosen to provide insight into the behavior of the actual composite LWHPC prestressed beams to be used in the Chickahominy Bridge. All decks of the test beams were NWHPC having a design 28-day compressive strength of 4,000 psi (28 MPa).

The mixture proportions for the Type II and Type IV LWHPC beams were intended to develop a concrete with a compressive strength of 8,000 psi (55 MPa) at 28 days with a density of about 120 lb/ft^3 (1920 kg/m^3). As stated previously, however, this mixture design did not develop its 28-day design strength. In all succeeding calculations and representations of strength in this paper, an average 28-day compressive strength of 6,375 psi (44 MPa) is considered for both the Type II and Type IV LWHPC beams.

The maximum load that could be applied during the flexural tests was limited by the strength of the load frame. The load frame (Figure 2) was designed to withstand the maximum anticipated load that could be carried by the test beams based on the beams' calculated flexural capacity. However, the actual loads applied to the beams exceeded the design loads by as much as 30 percent. As a result, two of the six tests were stopped at a maximum load of 285 kips (1,268 kN) before failure of the beam was reached, and an asterisk in Table 6 notes these tests. Table 6 also presents the embedment length results for each of the flexural strength tests.

Table 6 summarizes the findings of the development length and flexural strength testing and presents a comparison of the measured flexural strength and theoretical flexural strength. The theoretical flexural strength (M_{AASHTO}) was calculated in accordance with the AASHTO LRFD specifications, as was the theoretical development length, L_d [$(f_{ps} - 2/3 f_{se})\ d_b$].

Table 6 clearly shows that all the specimens tested exceeded the design strength predicted by the AASHTO equation. This could be interpreted to indicate that each beam was fully developed at the tested strand embedment length. A more stringent criterion for evaluating the test results is to use the observed failure mode (given in the fifth column of the table) to determine if the strand was fully developed at a particular embedment length. Based on the observed failure mode, both LWHPC and NWHPC specimens were fully developed at embedment lengths of 72 in (1829 mm), which is less than the AASHTO calculated development length as shown in Table 6. Accordingly, it could be argued that estimation of the "development length" as defined by AASHTO to enable a beam to develop its "design strength" is conservative for both normal weight and lightweight prestressed concrete beams of this study.

<u>Prestress Losses</u>

Prestress losses in pretensioned concrete members are composed of instantaneous and long-term losses. Instantaneous losses are caused by elastic shortening of the concrete and by slip of strands at the transfer of prestress. Long-term losses are due to creep and shrinkage of concrete and relaxation of steel over time.

The two AASHTO Type IV beams were fitted with internal vibrating wire gages at midspan, which measured concrete strains at the center of gravity of the prestressing strands. Three vibrating wire gages were installed in each beam to ensure consistent measurements. A data logger recorded strain readings from the gages at 2-hour intervals. Prestress loss determined directly from the concrete strains throughout a 9-month period was compared to that derived from the ACI and PCI models.

Concrete strains at the level of the strands are a function of prestress, eccentricity of the strands, and the gravity loading at the critical section. Because of a low modulus of elasticity and relatively low strength at detensioning (4780 psi [33 MPa]), the elastic shortening loss was relatively high at 20.5 ksi (141.3 MPa), i.e., 10 percent of the initial jacking stress of 205 ksi (1413 MPa). The overall prestress loss estimated for the LWHPC Type IV girders over a 9-month period was 28.65 ksi (197.2 MPa) using the ACI model and 16.20 ksi (111.7 MPa) using the PCI model. Creep constituted 55 to 63 percent of the calculated losses, and shrinkage about 33 to 36 percent. The actual compressive strains measured at the level of the strands in the LWHPC Type IV girders were less than that estimated by using either the PCI or the ACI model as shown in Figure 3. The ACI model, which is the more conservative, predicts an effective prestress of about 143 ksi (986 MPa) at the end of service of the prestressed beams. That is to say, the final effective prestress would be about 53 percent of the ultimate strength of the strands. This implies that it is safe to use the AASHTO equations to determine the stress in the strands and subsequently the strength of the LWHPC prestressed bridge girders, since the retained prestress in the members can safely be assumed to be greater than 50 percent fpu as stipulated in AASHTO.

CONCLUSIONS

The measured transfer length (averaged for all beam ends measured) for the LWHPC prestressed beams was lower than the estimated value given in the AASHTO LRFD specification. The actual transfer length measured at any one end of a beam varied considerably; however, even the largest measured transfer length was less than the AASHTO estimate, indicating that the AASHTO estimate over predicts the transfer length for this LWHPC.

Testing of Type II beams to flexural failure at various embedment lengths indicated that the strand could be fully developed at an embedment length below the required development predicted by the AASHTO LRFD specification. This indicates that the AASHTO development length is conservative for this LWHPC mixture.

The elastic shortening loss of Type IV beams was relatively high. The actual compressive strains measured at the level of strands in the LWHPC Type IV beams over a 9-month period were less than that estimated by the PCI and ACI models, which indicates that the models are conservative in estimating the prestress losses in LWHPC. It is safe to use the AASHTO equations to determine the stress in the strands and subsequently the strength of the LWHPC prestressed bridge beams, since the retained prestress in the members can safely be assumed to be greater than 50 percent fpu as stipulated in AASHTO.

Upon completion of this testing, the Chickahominy River Bridge was constructed using AASHTO Type IV beams similar to those that were tested. The 28-day compressive strengths were near or above the target value of 8,000 psi (55 MPa). The bridge was instrumented to record prestress losses over the life of the bridge. The next phase of the current research with LWHPC is to compare these losses to those predicted by current models and make recommendations for determining the losses in LWHPC in Virginia.

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REFERENCES

1. Ozyildirim, C., Gomez, J., "High-performance Concrete in a Bridge in Richlands, Virginia," VTRC Report No. FHWA/VTRC 00-R6, Virginia Transportation Research Council, Charlottesville, 1996.

2. Ralls, M. L., "Texas High-performance Concrete Bridges: How Much Do They Cost?," *Concrete International* (20) n 3, 71-74, 1998.

3. American Association of State Highway and Transportation Officials, "AASHTO LRFD Bridge Design and Specifications: Customary U.S. Units, Second Edition," Washington, D.C., 1998.

4. American Concrete Institute, Committee 318, "Building Code Requirements for Structural Concrete (318-99) and Commentary (318R-99)," Farmington Hills, Michigan, 1999.

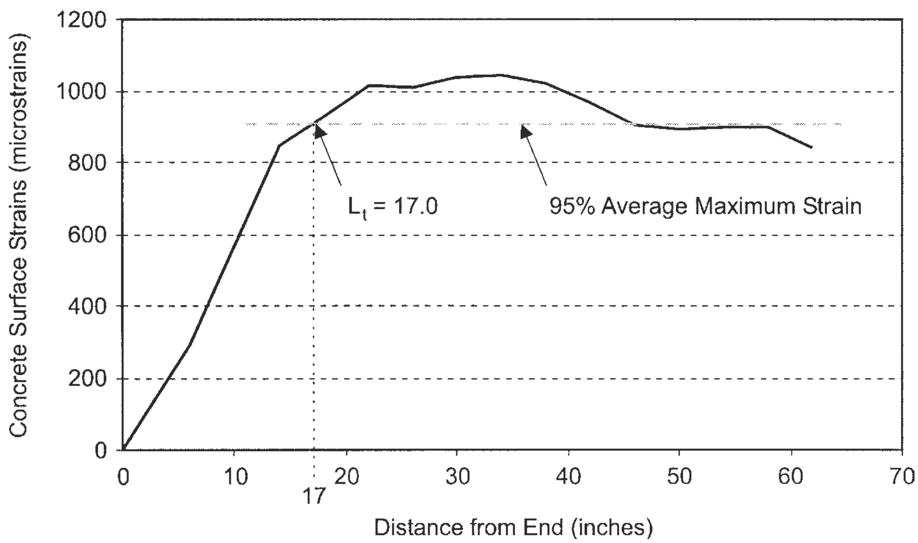

Figure 1. Average strain data profile for Type IV beams (1 in = 25.4 mm)

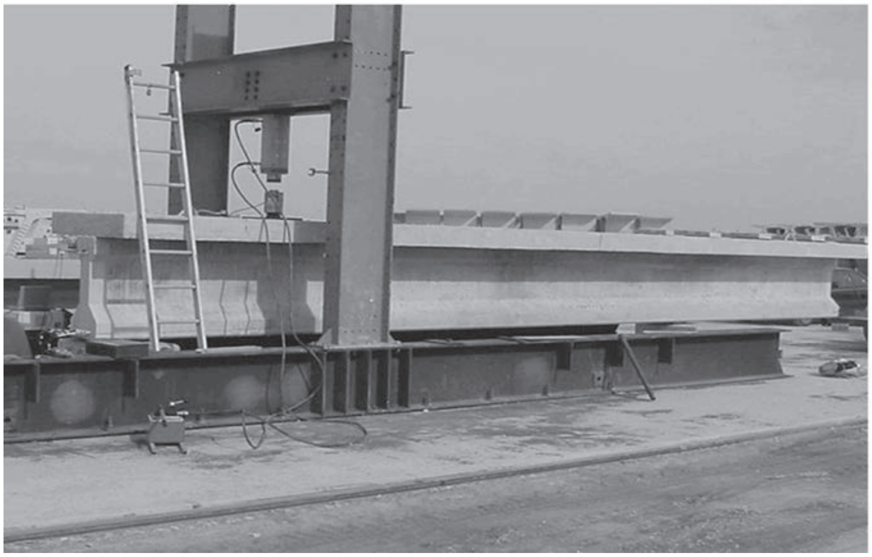

Figure 2. Load frame

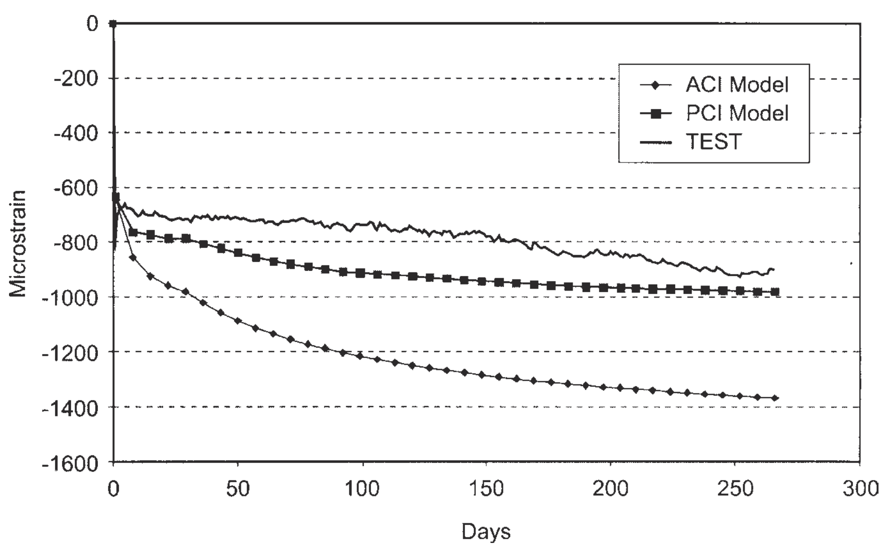

Figure 3. Concrete strains over time

Transfer and Development Length of 0.6-inch Strand in High Strength Lightweight Concrete

by K. F. Meyer and L. F. Kahn

Synopsis: This paper presents the findings of a research project conducted at Georgia Tech that tested six pretensioned AASHTO Type II girders constructed using expanded slate lightweight concrete with design strengths of 8,000 and 10,000 psi (55.2 and 68.9 MPa). Actual strengths ranged from 8,790 to 11,010 psi (60.6 to 75.9 MPa). Each was prestressed using 0.6-inch (15.2-mm) diameter low relaxation strands tensioned to 75% of strand ultimate stress. External strain measurements showed transfer lengths of 21.9 inches (556 mm) and 15.6 inches (396 mm) for the 8,000 and 10,000 psi (55.2 and 68.9 MPa) concretes; these were 73 percent and 52 percent of the design values given by AASHTO 16th Edition. Three-point bending tests were conducted on each beam to determine development length characteristics. The distance from the beam end to the load point was varied from between 70 and 100 percent of the AASHTO specified development length. Strand slip was measured for each test. Results indicated that the development lengths were 91 inches (2.31 m) and 67 inches (1.70 m) for the 8,000 and 10,000 psi (55.2 and 68.9 MPa) concretes; these were 95 percent and 70 percent of the design development lengths given by AASHTO 16th Edition.

Keywords: development length; direct pull-out; high-strength concrete; lightweight concrete; prestressed concrete; prestressing strand slip; pretensioned girders; silica fume; transfer length

Lieutenant Colonel Karl F. Meyer is an Associate Professor in the Department of Civil and Mechanical Engineering at West Point. He earned his Doctorate in Civil Engineering at the Georgia Institute of Technology where he conducted research on pretensioned high strength lightweight concrete bridge girders. He is an associate member of the American Concrete Institute Committee 213, and a member of Committee E802, Teaching Methods and Educational Materials.

Lawrence F. Kahn, FACI, is an Associate Professor in the School of Civil and Environmental Engineering at the Georgia Institute of Technology. His research interests include prestressed concrete and structural retrofit techniques. He is currently a member of the American Concrete Institute Committees TRRC, CRC, 364 and 546.

INTRODUCTION

Advances in concrete quality and engineering practices have enabled the design and construction of precast prestressed concrete bridge girders over 150 feet (45.7 m) in length. High-strength lightweight concrete (HSLC) can be used to reduce member self-weight in order to facilitate easier road movement and erection.

Previous research conducted relating to lightweight concrete (LWC) used in prestressed applications involved concrete with compressive strengths less than 7,500 psi (51.7 MPa). The research presented herein was designed to extend the application of lightweight concrete to design strengths up to 10,000 psi (68.9 MPa) and to verify the application of current design standards to the higher strength materials. An earlier analytical study showed that the use of HSLC would be beneficial for extending the lengths of bridge girders (1).

BACKGROUND

Transfer Length

Transfer length is defined as the distance required to transfer the effective prestressing force from the strand to the surrounding concrete. The transfer length is developed when the pretensioning strands are released by flame cutting or other method. Numerous factors are thought to contribute to determining the transfer length including size and surface condition of the prestressing strand, concrete strength and modulus of elasticity at time of strand release, level of prestressing in the strand, the amount of confining steel, and concrete consolidation in the transfer length region. Many experimental programs have focused on the above factors in determining an expression to predict transfer length (2-12).

Initial transfer length testing by Janney (2) in 1954 concluded that transfer length was attributable to diameter and surface condition of the prestressing wire and concrete strength.

Hanson and Kaar's (3) work in 1959 found an average transfer bond stress of 400 psi (2.75 MPa). Equation 1 was proposed as a way to determine the transfer bond stress.