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16. Abstract

This study involved the construction and early performance of the first bridge in Virginia constructed with lightweight high-performance concrete (LWHPC) having a density of 120 lb/ft^3 in the beams and deck. The design strength and permeability were 8,000 psi and 1500 coulombs, respectively, for the beams and 4,000 psi and 2500 coulombs, respectively for the deck.

The concretes were tested for slump, density, air content, compressive strength, flexural strength, permeability, elastic modulus, freeze-thaw durability, and shrinkage. The effectiveness of using fibers to control cracking over one of the two piers in the continuous deck was also investigated. The results indicate that LWHPC can be produced such that the material is workable, strong, volumetrically stable, and resistant to cycles of freezing and thawing, thus leading to a long service life with minimal maintenance. After 4 years of exposure, there was limited cracking in areas both with and without fibers.

LWHPC is recommended for use in beams and decks for reduced weight. The volumetric method for measuring air content is time-consuming and can cause adverse delays when a continuous deck is placed. Density measurements to control the air content of the LWHPC are recommended after a relationship is established.

The enhanced durability of LWHPC is expected to lead to extended service life with minimal maintenance costs. The lower initial cost due to the lighter weight concrete elements and the increase in the service life of the bridge because of the enhanced durability should result in significant savings.

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FINAL REPORT

FIRST BRIDGE STRUCTURE WITH LIGHTWEIGHT HIGH-PERFORMANCE CONCRETE BEAMS AND DECK IN VIRGINIA

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ABSTRACT

This study involved the construction and early performance of the first bridge in Virginia constructed with lightweight high-performance concrete (LWHPC) having a density of 120 lb/ft^3 in the beams and deck. The design strength and permeability were 8,000 psi and 1500 coulombs, respectively, for the beams and 4,000 psi and 2500 coulombs, respectively for the deck.

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INTRODUCTION

In general, hydraulic cement concrete has served well in bridge structures. However, early deterioration in some applications has led to costly repairs. Therefore, high performance concrete (HPC) was developed to construct cost-effective structures with an extended service life. HPC generally has a low water-cementitious material ratio (w/cm) and contains pozzolans or slag.[1-3] Virginia is a lead state in promoting HPC, and the Virginia Department of Transportation (VDOT) has been using HPC since the mid 1990s. The primary use of HPC is for improved durability. In beams, HPC has been used to attain strengths exceeding 7,000 psi.

The economic benefits of HPC include reduced costs for maintenance, transportation, and erection together with increased load-carrying capacity. In addition, the longer service life will minimize replacement costs and disruption of traffic. Many bridge structures have posted load-carrying capacities that make them functionally obsolete.[4] However, decks with lightweight HPC (LWHPC) can be used to replace an existing superstructure to improve lane capacity, thus keeping the bridge functional for heavy traffic for an extended period of time.

With regard to the properties of lightweight concrete and normal weight concrete, the former has a lower modulus of elasticity, higher inelastic strains, a more continuous contact zone between the aggregate and the paste, and more moisture in the pores of aggregates for continued internal moist curing.[5] These improvements lead to reduced cracking in the concrete and are highly desirable in bridge decks.[6] Further, normal weight concrete weighs about 150 lb/ft^3, leading to a significant dead load, resulting in higher stresses for the same external loading. Therefore, structural lightweight concrete has been produced with a lower density—generally ranging from 115 to 120 lb/ft^3—through the introduction of lightweight aggregates (either lightweight coarse aggregates alone or lightweight coarse aggregates combined with lightweight fine aggregates).

PURPOSE AND SCOPE

Although LWHPC is expected to have reduced weight, improved microstructure, high compressive strength, and low permeability, which can lead to longer lasting and cost-effective bridge decks, there are concerns with the tensile strength, modulus of elasticity (low stiffness of the member), shrinkage, and creep properties in beams.

In this study, LWHPC was tested in a full-scale bridge structure to determine if the expected benefits could be achieved and if the areas of concern required further attention.

METHODOLOGY

Overview

The beams and deck of a bridge on Route 106 over the Chickahominy River near Richmond, Virginia, were constructed to evaluate the properties and performance of LWHPC. Prior to fabrication of the 15 LWHPC beams, a test program was conducted using actual size and smaller beams.[7] All test beams were made of LWHPC except for one, which was constructed with normal weight HPC (NWHPC) for comparison.

The bridge beams support a concrete deck that extends across three spans and is continuous over the two piers. Each span is 85 ft long and 43.3 ft wide. The entire deck was constructed with LWHPC; a portion of the deck over one of the piers contained fibers in the concrete so that the effect of fibers on crack control could be evaluated.

The bridge was opened to service on September 27, 2001, and carries heavy truck traffic to an industrial park, a logging business, and a large waste disposal site. Condition surveys were performed after the placement of the deck and after 4 years.

Construction

The construction of the bridge involved three phases. In the first phase, a test program focused on beam fabrication and testing. In the second phase, the actual bridge beams were fabricated and placed. In the third phase, the bridge deck was placed. Concrete properties were determined for the freshly mixed and hardened states in all three phases.

For the beams, the specified 28-day minimum compressive strength was 8,000 psi with a release strength of 4,500 psi. Grade 270 low-relaxation prestressing strands 0.5 in in diameter were used. The target density of LWHPC for the beams and the deck was 120 lb/ft^3. Permeability tests were conducted in accordance with AASHTO T 277 or ASTM 1202The specimens were moist cured at 73°F for 1 week, and then at 100°F for 3 weeks. The maximum desired permeability values was 1500 coulombs for the beam concrete and 2500 coulombs for the deck concrete. The specified 28-day minimum compressive strength for the deck concrete was 4,000 psi. Slump (ASTM C143), air content (ASTM C173), and density (ASTM C138) were determined in the fresh state, and samples were made for tests at the hardened state, as indicated in Table 1. The beams were steam cured after initial set for 13 hours to obtain high early release strengths and then stored outside. Specimens from the beams were kept in the recesses of the beam forms during steam curing, were then moist cured for 7 days, and were then air dried. The specimens were subjected to limited moist curing, but the beams were not, because of the small size of the specimens and the early test ages. The deck specimens were moist cured until testing.

Flexural strength and elastic modulus were also determined using the theoretical equations for comparison with the test values. The theoretical elastic modulus was calculated using $E_c = w_c^{1.5} C\sqrt{f'_c}$, where w_c is the unit weight and C is a constant related to the compressive strength of the lightweight concrete, and actual density and compressive strengths were used (ACI 213) C values were obtained from linear extrapolation beyond C = 29 at the maximum f'c of 6,000 psi specified in ACI 213.

Table 1. Test and Specimen Sizes

Tests	Specifications	Size (in)
Compressive Strength	AASHTO T 22	4 x 8
Elastic Modulus	ASTM C69	4 x 8
Splitting Tensile Strength	ASTM C496	4 x 8
Permeability	AASHTO T 277, T 259	2 x 4, 12 x 12
Drying Shrinkage	ASTM C157	3 x 3 x 11$^1/_4$
Creep	ASTM C512	6 x 12
Freeze-Thaw Durability	ASTM C666	3 x 4 x 16
Unit Weight	ASTM C567	4 x 8

Test Beams

Five test beams were fabricated on November 8, 2000. Three were 32-ft-long AASHTO Type II beams); two had LWHPC and the third NWHPC. All three beams were later cast with a normal weight composite deck section. The third beam was intended to serve as a control for comparison. The other two beams were 84-ft-long AASHTO Type IV beams made of LWHPC. They were identical in design with the actual beams for the bridge constructed later on Route 106. The test beams were instrumented with vibrating wire gages and were analyzed for strain, shrinkage, and prestress losses. In addition, Type T thermocouples were used to monitor temperature continuously during the placement of the beams.

Materials

The mixture proportions for the LWHPC and NWHPC for the test beams are provided in Table 2. 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 relative density of 2.98, and a bulk density of 108.4 lb/ft^3. The lightweight coarse aggregate was ¾ in to No. 4 expanded slate with an absorption of 6 percent, a relative density of 1.47, and a bulk density of 49 lb/ft^3. The fine aggregate was natural sand with an absorption of 1.4 percent, a relative density of 2.61, and a fineness modulus of 2.60. The

Table 2. Mixture Proportions for Concrete in Test Beams (lb/yd^3)

Material	Normal Weight	Lightweight
Portland Cement	451	451
Slag	301	301
Water	255	250
w/cm	0.34	0.33
Fine Aggregate	1208	1419
Coarse Aggregate Normal Weight	1873	----
Coarse Aggregate Lightweight	----	800
Air (%)	5.5 ± 1.5	5.5 ± 1.5
Air-Entraining Admixture (fl oz)	15	12
Water-reducing and retarding (fl oz)	22	23
High-Range Water-Reducing Admixture (fl oz)	56	56

admixtures were a commercially available air-entraining admixture (AEA); a vinsol rosin complying with the requirements of ASTM C260; a water-reducing admixture (WRA); a lignin complying with the requirements of ASTM C494, Type A; and a high-range water-reducing admixture (HRWRA), which is a polycarboxylate complying with the requirements of ASTM C494, Type F.

Structural Testing

The structural evaluation of the test beams was conducted at the plant. The beams were tested for transfer length, development length, flexural strength, and prestress losses. Prestress losses were also calculated using the models of the American Concrete Institute (ACI)[8] and the Precast/prestressed Concrete Institute (PCI)[9] that incorporate lightweight structural concretes.[7]

To determine the transfer length, 18 Whittemore points spaced 4 in apart stretching 64 in along the beam ends on each side of the Type IV beams were used. Each strain reading represented the average strain along the 8-in gage length of the Whittemore device. Because the inserts were spaced 4 in apart, successive strain readings overlapped 4 in (or one middle Whittemore point).

Each end of the three Type II beams was loaded to failure at various embedment lengths, L_e, to establish the development length, L_d, and the flexural strength of the specimens. In theory, the development length is established when a test specimen has attained its ultimate flexural strength at the shortest tested embedment length.

Bridge Beams

The bridge has 15 AASHTO Type IV beams 84 ft long, which were fabricated in March 2001 at the same plant that made the test beams.

Materials

The mixture proportions for the bridge beams are given in Table 3. The aggregate proportions were different than those used for the test beams because of difficulties in achieving

Table 3. Mixture Proportions for Concrete in Bridge Beams (lb/yd^3)

Material	Amount
Portland Cement	451
Slag	301
Water	255
w/cm	0.34
Fine Aggregate Normal Weight	541
Fine Aggregate Lightweight	390
Coarse Aggregate Normal Weight	605
Coarse Aggregate Lightweight	696
Air	5.5 ± 1.5%
Water-Reducing Admixture (fl oz)	22
High-Range Water-Reducing Admixture (fl oz)	56
Air-Entraining Admixture (fl oz)	12
Calcium Nitrite (gal)	3

the specified strength in the test beams. The same materials used in the test beams were used in the bridge beams; however, a portion of the fine aggregate used was lightweight sand, which had an absorption of 5 percent and a relative density of 1.88. Further, the coarse aggregate consisted of both normal weight aggregate and lightweight aggregate. In addition to AEA, WRA, and a HRWRA, calcium nitrite (30% solids) was used to help improve the strength.

Deck

Deck placement started around 11 P.M. on July 9, 2001, from the north end and continued until 7:30 A.M. the next morning. Concrete was mixed and delivered in ready-mix concrete trucks from a plant located 11 miles away. At the job site, air content was determined by the volumetric method (ASTM C173) on the first three truckloads of concrete and then on every third load before the concrete was delivered to the deck.

Materials

Mixture proportions for the deck concrete are given in Table 4. The cementitious material was a combination of Type II cement and Class N calcined shale natural pozzolan. The coarse aggregate was a No. 57 lightweight aggregate with an absorption of 6 percent, a relative density of 1.5, and a density of 49.4 lb/ft^3. The fine aggregate was natural sand with an absorption of 0.8 percent, a relative density of 2.63, and a fineness modulus of 2.90. The admixtures were a commercially available AEA, an inorganic solution of sodium salts; water-reducing and retarding admixture (WR+R), a lignin conforming to the requirements of ASTM C494, Type D; and a WRA, a hydroxlated carboxylic acid conforming to the requirements of ASTM C494, Type A. Some batches contained HRWRA and a monofilament fiber (9 lb/yd^3), which consisted of a synthetic blend of polypropylene and polyethylene resins that fibrillated during mixing. The purpose of the fibers was to control cracking in the deck, and the HRWRA was added to compensate for the loss in workability because of the fibers. Fibers were added to the concrete toward the south end of the deck to cover the area over the pier. This section started 18 ft north of the pier and continued to 25 ft south of the pier.

Table 4. Mixture Proportions for Deck Concrete (lb/yd^3)

Material	Amount
Portland Cement	489
Pozzolan	163
Water	292
w/cm	0.44
Fine Aggregate	1228
Coarse Aggregate	900
Air (fl oz)	6.5 ± 1.5 %
Air-Entraining Admixture	as needed
Water-Reducing Admixture (fl oz)	25
High-Range Water-Reducing Admixture (fl oz)	42

Each load of concrete measured 9 yd^3; a total of 42 loads were used. For the fiber section, 3 loads of concrete were delivered by pump, and 5 loads were delivered by crane and bucket. The reason for the bucket delivery was that the concrete containing fibers had difficulty passing through the pump grate, even though a vibrator was used at the grate. After passing through the grate, the fiber-reinforced LWHPC was easier to pump than the regular LWHPC because of the presence of HRWRA and possibly because of the addition of too much water. All loads without the fiber reinforcement were pumped.

The concrete was consolidated by internal vibrators and finished using the vibratory roller screed. The curing for the bridge deck was done through several stages. Fog misting began immediately following the screeding. Then, the deck was covered with plastic. The next day, soaker hoses were placed under the plastic. Fourteen days later, the plastic and hoses were removed and a curing compound was applied. The temperature of the concrete was monitored by thermocouples. One set of thermocouples was placed at each end of the deck. Each set consisted of one thermocouple in the top, one in the middle, and one in the bottom of the deck.

Condition Surveys

After construction, condition surveys of the deck were conducted for 4 years to determine the surface conditions. The degree of surface scaling and cracking was determined.

RESULTS AND DISCUSSION

Test Beams

Fresh Concrete

The characteristics of the freshly mixed concrete are given in Table 5. The measured air contents were all within the specified 5.5 ± 1.5 percent range. The slump values for the LWHPC were higher than for the NWHPC, and both concretes were workable. The lower than expected density was attributed to the high air content and excess water in the mixture, since air contents in the hardened state were higher than in the fresh state (Table 6) and desired strengths were not achieved (Table 7). The low air content results by the volumetric test may be attributable to the difficulty in releasing the bubbles in the usual time spent to conduct the test. The conditioning of the lightweight aggregate may have been the cause of the extra water; the aggregates were prewet using a sprinkler system. However, the sprinkler system used 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.

Table 5. Characteristics of Freshly Mixed Concrete for Test Beams

Test	NW B1	LW B2	LW B3	LW B4	Specified
Air (%)	5.2	5.6	5	4.6	3-7
Slump (in)	4.5	7.5	6.5	6	0-7
Unit Weight (lb/ft^3)	145.2	115.6	112.8	114.8	-
Concrete Temperature (°F)	71	70	71	70	40-100

NW = normal weight; LW = lightweight.

Table 6. Air-void Parameters for Concrete in Test Beams

Concrete	Batch	Voids > 1 mm (%)	Total Voids (%)	Spacing Factor (mm)
NW	B1	2.3	5.4	0.3813
LW	B2	2.4	6.5	0.3467
LW	B3	2.9	7.9	0.3067
LW	B4	3.4	7.3	0.4216

NW = normal weight; LW = lightweight.

Hardened Concrete

The strength and permeability values are summarized in Table 7. The results are an average of two specimens. At 28 days, the NWHPC was close to the specified compressive strength of ≥ 8000 psi, but 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, with an actual average density of 114 lb/ft^3. As experience has shown that the strength ceiling of this lightweight aggregate is more than 10,000 psi, it is anticipated that higher concrete strengths would be achieved by improvements in mixture proportioning and control of water. At transfer, the average strength of the LWPHC beams was 4,780 psi, which is about 25 percent lower than the release strength of 6,400 psi for the test beams. It is suspected that an undesirably high w/cm for the mixture and high air contents resulted in its lower strength. The ratio of the 28-day flexural strength to the compressive strength was 8.4 percent for the LWHPC and 10.8 percent for the NWHPC. The lightweight concrete provides lower flexural strengths.[6] NWHPC had the lowest permeability value, whereas two of the three LWHPC batches had marginal values attributed to the high w/cm, as with the strength values. The specified permeability value is < 1500 coulombs at 28 days when tested in accordance with AASHTO T 277 after a 7-day moist cure at 73°F and a 3-week moist cure at 100°F. The elastic modulus values of the LWHPC were lower than those of the NWHPC, as expected. The elastic modulus was measured and calculated (theoretical). The measured and calculated values matched well, as shown in Table 7.

Table 7. Properties of Hardened Concrete for Test Beams

Test	Age	NW B1	LW B2	LW B3	LW B4
Compressive Strength (psi)	1d (TMC)	6770	6130	5430	-----
	1 d	6040	5080	4210	5040
	7 d	6430	5430	5410	5250
	28 d	7800	6660	6320	6140
	6 mo	8990	6910	6900	6220
	1 yr	8160	6930	6990	7030
Flexural Strength (psi)	28 d	845	600	520	490
Permeability (coulombs)	28 d	938	1110	1507	1454
Elastic Modulus (E + 06 psi)	28 d	4.90	2.95	2.78	2.72
	1 yr	4.41	2.97	2.73	2.89
E Theoretical[a] (E + 06 psi)	28 d	5.10	2.82	2.70	2.77

TMC = temperature-matched control; NW= normal weight; LW = lightweight.
[a] $E_c = w_c^{1.5} C \sqrt{f'_c}$; C = 27 for 7,000 psi; C = 28 for 6,500 psi; C = 29 for 6,000 psi for LW concrete.

The drying shrinkage values are displayed in Table 8. The values were about 600 microstrain or less at 1 year. The LWHPC had higher shrinkage than did the NWHPC. This difference was attributed mainly to the type of aggregate used, because lightweight aggregate provides less restraint. However, the suspected high water content may also have contributed to the higher shrinkage.

Characteristics of the air-void parameters were determined using the linear traverse method (ASTM C457) and are summarized in Table 6. For both NWHPC and LWHPC, the values for voids larger than 1 mm were higher than the 2 percent expected for properly consolidated concrete.[10] Large voids are generally a result of improper consolidation or extra water in the mixture. However, consolidation is not expected to be a problem since workable concretes were obtained and vibrators were used. The total air content of the hardened LWHPC was higher than at the freshly mixed state. The large differences between the fresh and hardened states were attributed mainly to the presence of extra water and to the testing process, since releasing air bubbles in the volumetric test is difficult. In all mixtures, the spacing factors were higher than the generally expected 0.008 in and the specific surface values were less than the minimum accepted value of 600 in^2/in^3, indicating the absence of a sufficient number of small bubbles for satisfactory resistance to cycles of freezing and thawing.[11]

In addition to an air-void analysis, the concretes were tested for resistance to cycles of freezing and thawing. The acceptance criteria at 300 cycles were a maximum weight loss of 7 percent, a minimum durability factor of 60, and a maximum surface rating of 3. The results are shown in Table 9. The tests should have continued for 300 cycles but were stopped at 200 cycles because all four batches exceeded the 7 percent maximum allowable weight loss. However, at

Table 8. Shrinkage Values for Test Beams (in microstrain)

Batch	28 Days	4 Months	8 Months	1 Year
NW B1	340	400	440	505
LW B2	345	440	485	555
LW B3	375	490	545	615
LW B4	310	430	490	565

NW = normal weight; LW = lightweight.

Table 9. Freeze-Thaw Data for Concrete in Test Beams at 200 Cycles

Batch	Weight Loss (%)	Durability Factor	Surface Rating
NW B1	7.7	94	2.6
LW B2	17.1	85	3.9
LW B3	11.0	100	2.6
LW B4	11.4	100	2.6

NW = normal weight; LW = lightweight.
Note: Testing terminated at 200 cycles since weight loss > 7%.

200 cycles, the four batches had durability factors that were much higher than the minimum acceptable value of 60, and three had surface ratings below the maximum acceptable value of 3.

The fact that the samples failed the weight loss criteria at 200 cycles while meeting the durability factor requirements signifies that surface scaling was occurring. LWHPC showed more scaling than NWHPC. This scaling was probably attributable to the test conditions, with 2 percent NaCl solution in the test water, and the fact that the lightweight aggregate was prewet prior to batching. Drying for an extended period before testing as specified in ASTM C330 would have improved the resistance to freezing and thawing.[12] However, the lack of a proper air-void system confirms the marginal performance of the concrete for the test beams. The test beams were not of the desired quality; the concrete had too much water and a poor air-void system and failed the freeze-thaw test.

Figure 1 presents the temperature data for Beam 1; the temperatures in the beam reached a maximum of about 172°F. In Beam 2, the maximum temperature was about 177°F.

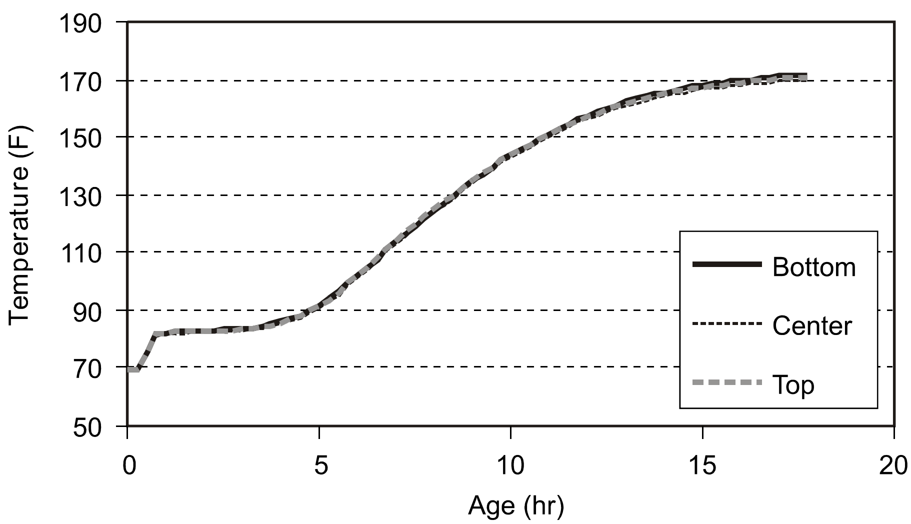

Figure 1. Temperature Data for Bridge Beam 1

Structural Testing

The transfer length as determined from the strain profiles of each test beam end is shown in Table 10. The averaged strain data profile for the Type IV beams is shown in Figure 2. The transfer length from this strain profile was 17.0 in, which is less than that calculated using the AASHTO LRFD (load and resistance factor design) equation, 60 d_b, where d_b is the prestressing

Table 10. Transfer Length, L_t, for Each Type IV Beam End (in)

Beam	End	L_t	L_t	L_t
1	A	22.8		
1	B	13.7	18.3	17.0
2	C	18.8		
2	D	13.4	16.1	

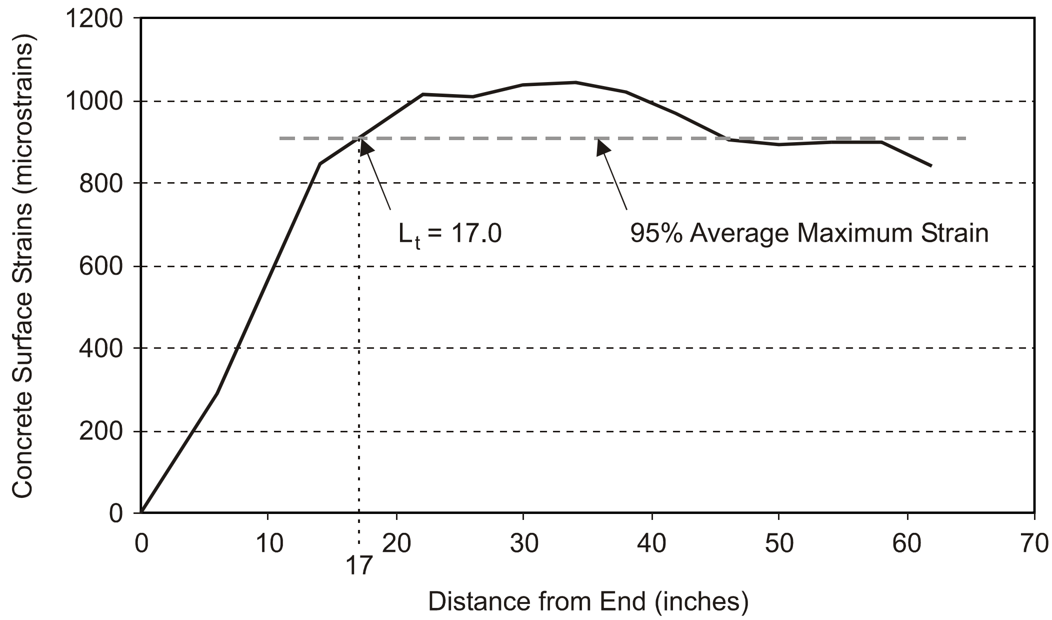

Figure 2. Average Strain Data Profile for Type IV Beams

strand diameter.[13] For a diameter of 0.5 in, the AASHTO equation predicts a transfer length of 30 in. The measured transfer length (averaged for all beam ends measured) for the LWHPC prestressed beams was 43 percent lower than the estimated value given in the AASHTO LRFD Specification.[13] However, the actual transfer length measured at any one end of a beam did vary considerably, from a low of 13.4 in, 55 percent below the AASHTO estimate, to a high of 22.8 in, 24 percent below the AASHTO estimate.

The maximum load that could be applied during the flexural tests was limited by the strength of the load frame. The load frame was designed to withstand the maximum anticipated load that could be carried by the test beams based on the calculated flexural capacity of the beams. 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 before failure of the beam was reached (see Table 11). Table 11 also presents the embedment length results for each flexural strength test.

Table 12 summarizes the findings of the development length and flexural strength testing and presents a comparison of the measured flexural strength and the theoretical flexural strength. The theoretical flexural strength (M_{AASHTO}) was calculated in accordance with the AASTO LRFD specifications, as was the theoretical development length, L_d [$(f_{ps} - 2/3 f_{se})\ d_b$].[13]

Table 11. Flexural Test Designation and Embedment Length

Test No.	Concrete Type	Embedment Length (in)
T1	LW	72
T2[a]	LW	60
T3	LW	72
T4	LW	96
T5[a]	NW	96
T6	NW	72

LW = lightweight; NW = normal weight.
[a]Tests were stopped before failure because of the capacity of the test apparatus.

Table 12. Comparison of Actual Embedment Length to Theoretical Development Length and Actual Flexural Strength to Theoretical Flexural Strength

Specimen ID/ Embedment Length (in)	L_d (in)	Failure Type	M_{TEST} (kip-in)	M_{AASHTO} (kip-in)	M_{TEST}/M_{AASHTO}
T1/72	76	Flexure	14,700	11,500	1.27
T2/60	76	Bond/Shear	12,600	11,400	1.11
T3/72	76	Flexure/Bond	14,300	11,500	1.24
T4/96	76	Flexure	15,100	11,600	1.30
T5/96	75	None	15,200	11,500	1.32
T6/72	75	Flexure	15,900	11,600	1.37

Table 12 clearly shows that all 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 third 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 equal to or greater than 72 in, which is 5 percent less than the AASHTO calculated development length of 76 in for these beams. Accordingly, it could be argued that estimation of the "development length" as defined by AASHTO[13] to enable a beam to develop its "design strength" is conservative for both normal weight and lightweight prestressed concrete beams.

Jacking stress was 76 percent of the ultimate strength, and the final prestress force after losses was about 56 percent of the ultimate strength. The actual compressive strains measured at the level of the strands were less than that estimated by the ACI model[8] and slightly more than that estimated by the PCI model[9] as shown in Figure 3.

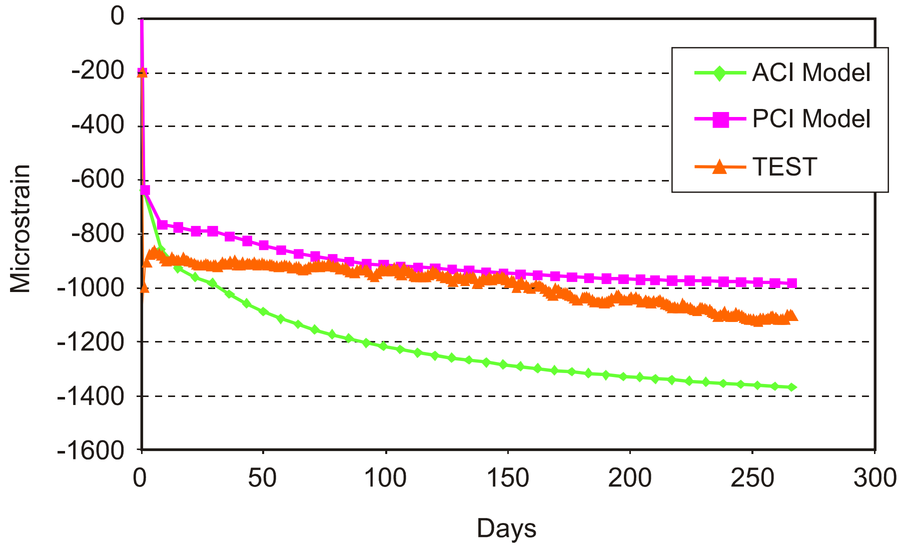

Figure 3. Concrete Strains Over Time

Bridge Beams

Fresh Concrete

Table 13 lists the characteristics of the freshly mixed concrete in the bridge beams. The air contents were within the specified limits. The concretes were workable and had high slump. The density values were about 120 lb/ft^3.

Table 13. Characteristics of Freshly Mixed Concrete for Bridge Beams

Test	B1	B2
Air (%)	5.5	6.0
Slump (in)	7.0	8.5
Unit Weight (lb/ft^3)	122.0	118.8
Concrete Temperature (^{o}F)	70	68
Air Temperature (^{o}F)	----	50

Hardened Concrete

The properties of the hardened concrete are given in Table 14. The 28-day compressive strengths were near the target value of 8,000 psi (55 MPa). The permeability values at 28 days were above 1500 coulombs, partially due to the presence of calcium nitrite; however, at 1 year, they were low or very low. The elastic modulus values were similar to those for the test beams. The average flexural strengths were 7.9 percent of the compressive strength.

Table 15 shows the results of the freeze-thaw tests. Both batches performed very well, exhibiting only minor weight loss, and had excellent surface ratings and acceptable durability factors. Since the freeze-thaw tests were successful, an air void analysis was not conducted. The shrinkage values are provided in Table 16 and are similar to those obtained for the test beams.

Table 14. Properties of Hardened Concrete for Bridge Beams

Test	Age	B1	B2
Compressive Strength (psi)	1 d (TMC)	4320	4370
	1 d	4800	4640
	7 d	7110	6900
	28 d	8310	7900
	1 yr	7740	7550
Flexural Strength (psi)	28	695	580
Permeability (coulombs)	1 yr	1034	799
Elastic Modulus (E + 06 psi)	28	2.91	3.04
	56	2.88	2.63
E Theoretical[a] (E + 06 psi)	28	2.99	2.91

[a]$E_c = w_c^{1.5}C\sqrt{f'_c}$, C = 24 for 8,500 psi, C = 25 for 8,000 psi, C = 26 for 7,500 psi for lightweight concrete.

Table 15. Freeze-Thaw Data for Bridge Beams at 300 cycles

Batch	Weight Loss (%)	Durability Factor	Surface Rating
B1	1.8	84	1.8
B2	3.2	62	1.0

Table 16. Shrinkage Values for Bridge Beams (in microstrain)

Batch	28 Days	4 Months	8 Months	1 Year
B1	475	545	600	615
B2	410	490	555	580

Deck

Fresh Concrete

Four batches of concrete were tested; only the last two contained fibers (B3, B4). The concrete was sampled after pumping except for Batch 3, which was sampled before being delivered in a bucket.

The fresh concrete properties are listed in Table 17. Although relatively low, the air contents were within specifications. Slump values indicated that workable concrete was achieved. Although the density was higher than specified in the pumped concrete, the density met the specification when tested before pumping. This result indicates that air was lost during

Table 17. Properties of Fresh Deck Concrete

Test	B1	B2	B3F	B4F
Air (%) fresh	5.0	5.5	5.0	5.7
Slump (in)	3.5	3.7	4.1	5.5
Unit Weight (lb/ft^3) After Pump	126	126.4		121.6
Unit Weight (lb/ft^3) Before Pump		118.8	118.0	
Concrete Temperature (^{o}F)	----	77	----	73

pumping. However, extreme caution should be exercised when pumped concrete is sampled. The vertical drop when samples were taken was longer than the vertical drop when concrete was deposited on the deck and the flow of concrete was not continuous, which could have contributed to a large loss of air that would adversely affect the resistance to freezing and thawing.[14]

Air content tests using the volumetric method took a long time, from 15 minutes to 1 hour for a single test. With air content being checked for the first three truckloads and then every third load, there were delays in deck placement. Extended delays may cause concrete to stiffen, leading to difficulty in finishing and cold joints. One solution for alleviating this condition would be to develop a relationship between the density and the air content in the fresh concrete and then to use density values for acceptance.

Hardened Concrete

The hardened concrete properties are given in Table 18. The compressive strengths were higher than the specified values; however, the strength of the LWHPC with fibers was much lower than that of the LWHPC without fibers. Similarly, the permeability values were satisfactory and lower than specified; however, the concretes without fibers had lower values than the concretes with fibers. The large differences in the strength and permeability of concretes with and without fibers were attributed to the extra water added to compensate for the loss of workability due to the addition of fibers rather than the addition of an adequate amount of HRWRA.

LWHPC with and without fibers had a low elastic modulus. The calculated values were higher than the measured values in B1 and B2 but close to those in B3 and B4. A low elastic modulus in a bridge deck is desirable for improving strain capacity and crack control. The flexural strength values far exceeded the theoretical values.

ASTM C1399 indicates which deflection points are used to calculate the residual strength, which is the strength of the material after the first crack. Unreinforced concrete has zero residual strength. However, the data in Table 17 show that fibers provide a large amount of residual strength to concrete, with values exceeding 300 psi.

Table 18. Hardened Concrete Properties for the Deck

Test	Age	B1	B2	B3F	B4F
Compressive Strength (psi)	1 d	4570	4910	3310	3240
	7 d	5230	5700	3870	3910
	28 d	7070	7380	4820	5055
Flexural Strength (psi)	7 d			640	675
	28 d	805	755	750	730
Permeability (coulombs)	28 d	747	916	1098	1645
Residual Strength (psi)	28 d			320	304
Elastic Modulus (E + 06 psi)	28 d	2.75	2.75	2.78	2.78
E Theoretical[a] (E + 06 psi)	28 d	3.19	3.20	2.79	2.94

[a] $E_c = w_c^{1.5} C \sqrt{f'_c}$, C = 27 for 7,000 psi; C = 31 for 5,000 psi; C = 32 for 4,500 psi for lightweight concrete.

Freeze-thaw data are given in Table 19. The results indicated acceptable values for weight loss and a surface rating indicative of scaling (except for the surface rating for Batch B4, which was marginal). The durability was satisfactory for Batches B1 and B3 but poor for Batches B2 and B4.

Similarly, the linear traverse data shown in Table 20 indicate that Batches B1 and B3 had the lowest spacing factors, with that for B1 being marginally higher than the generally accepted limit. Batches B2 and B4 had unsatisfactory void system. Considering the more than 40 years of satisfactory performance of the lightweight aggregates used in this testing program, LWHPC with the proper air void system provides satisfactory resistance to cycles of freezing and thawing. Pumping affects the air-void system, and concrete should be tested after placement and consolidation.

Measurements for concrete shrinkage are provided in Table 21. The values were less than 400 microstrain at 28 days and 700 microstrain at 4 months, which are the recommended maximums for satisfactory performance for bridge decks.[15]

Figure 4 shows the temperatures at varying depths of the deck concrete. As anticipated, the temperatures were high early on in the curing process because of the heat of hydration and then decreased in a few days, matching the cycles in the air temperature. The top thermocouple generally registered the highest temperature. Both ends of the bridge deck demonstrated a similar pattern in temperature fluctuation.

Table 19. Freeze-Thaw Data for Deck Concrete at 300 Cycles

Batch	Weight Loss (%)	Durability Factor	Surface Rating
B1	1.80	94	1.28
B2	4.35	45[a]	1.89
B3F	2.75	99	1.91
B4F	6.92	35[a]	3.05

[a]The durability factor test for B2 ended at 150 cycles and for B4 at 100 cycles because of the inability to obtain dynamic modulus values.

Table 20. Air-void Parameters for Concrete in Bridge Deck

Batch	Voids > 1 mm (%)	Total Voids (%)	Specific Surface (mm^{-1})	Spacing Factor (mm)
B1	0.7	2.5	24.8	0.2671
B2	2.6	3.7	6.7	0.8209
B3F	0.5	5.4	28.3	0.1641
B4F	1.4	4.5	15.5	0.3261

Table 21. Shrinkage Values for Deck Concrete (microstrain)

Batch	28 Days	4 Months	8 Months	1 Year
B1	325	565	600	595
B2	360	565	620	615
B3F	500	670	700	685
B4F	445	640	660	650

Note: B1 and B2 were control batches without fibers; B3F and B4F contained fibers.

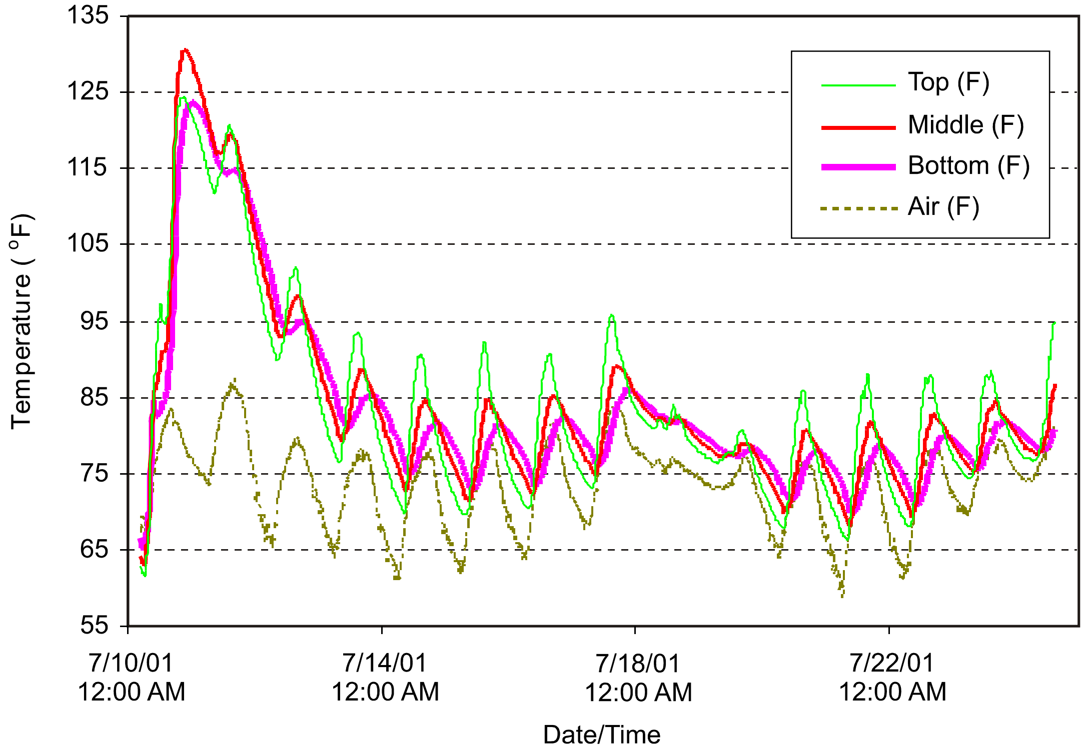

Figure 4. Temperatures at Varying Depths of Deck Concrete

Condition Survey

While the soaker hoses were being set the day after placement, transverse cracks over both piers were discovered. Cracks propagated across the entire width of the deck over the control pier, but there was cracking over only half the width of the deck with the fibers. A few plastic shrinkage cracks were observed at the south end of the deck in a section without fibers. Three weeks after the placement of the bridge deck, these cracks were not noticeable, probably because the warmer temperatures had caused the concrete to expand and close the cracks.

The roadway was opened to traffic on September 27, 2001. In April 2002, another site visit was made; no cracks were visible on the deck. However, in May 2002, a very tight transverse crack was visible above each pier. During a more extensive survey in July 2002, two transverse cracks over both piers extended across the width of the bridge; the cracks in the section that was not reinforced by fibers were 0.20 mm to 0.25 mm wide, and those in the fiber section were about 0.20 mm wide. The fiber section had three shorter transverse cracks 0.20 mm wide that averaged 10 ft in length; the remainder of the bridge had only one other transverse crack, which was 10 ft long and 0.20 mm wide. There were also 15 longitudinal cracks in the control section, 6 to 12 in long and about 0.15 mm wide. There were no plastic shrinkage cracks in the fiber section of the deck; the control section had a few plastic shrinkage cracks near the parapets in hand-finished areas. In April 2003, another visit to the deck indicated that cracking was still very limited, without noticeable change compared to the previous year. Two transverse cracks with a width of 0.30 mm were observed over each pier, including in the concrete both with and without fibers. On June 15, 2005, a final visit showed much of the same—limited cracking in both the control and the fiber deck sections. Over the piers, both the control and the fiber sections of the deck exhibited two transverse cracks. The ones in the control portion were about 33 ft long and 0.36 mm wide, and the two in the fiber section were about 29 ft long and 0.29 mm wide.

CONCLUSIONS

- LWHPC can be produced such that the material is lightweight, workable, strong, volumetrically stable, and durable.

- The elastic modulus for lightweight concrete is lower than for normal weight concrete. The elastic modulus can be calculated theoretically using the appropriate constant. Currently, ACI 213 provides constants for strengths up to 6,000 psi. Extrapolating linearly to higher strengths appears to provide satisfactory results.

- In the test beams, the actual compressive strains measured at the level of the strands were less than those estimated by the ACI model[8] and slightly more than those estimated by the PCI model.[9]

- Structural tests for the trial beams indicated that the members had similar or higher actual flexural strength compared to the theoretical values.

- LWHPC and NWHPC beams had shorter transfer lengths than were predicted.

- In the test beams, specified strengths were not achieved. The LWHPC had much lower strength than did the NWHPC, which was attributed mainly to extra water in the LWHPC. Thus, better water control, including aggregate moisture, is needed in the production of LWHPC.

- The ratio of the flexural strength to the compressive strength was lower in LWHPC than in NWHPC.

- Shrinkage in the LWHPC deck was satisfactory and within the limits recommended in the literature.[15] Shrinkage of LWHPC was greater than shrinkage of NWHPC in the beams because lightweight aggregates offer less restraint and the LWHPC was suspected to have a high water content.

- Fibers provide residual strength that is expected to control cracking; however, the limited transverse cracking observed in the deck suggests that LWHPC does not require fibers in the initial years of exposure. Fibers appear to aid in controlling plastic shrinkage cracking since none was visible in the fiber section but a few were present in the control section

- The presence of fibers tends to reduce workability, which may be compensated for by additional water. However, water adversely affects the strength, permeability, and durability of concrete. Therefore, water-reducing admixtures should be used to improve workability.

RECOMMENDATIONS

- VDOT's Structure & Bridge Division should use LWHPC in beams and decks for reduced weight.

- VDOT inspectors should use density measurements to control the air content of the lightweight concrete after a relationship is established. The volumetric method for measuring air content is time-consuming and can cause adverse delays when a continuous deck is placed.

BENEFITS AND COSTS ASSESSMENT

In assessing the cost-effectiveness of LWHPC, the up-front material cost for lightweight concrete must be weighed against a variety of benefits, many of which accrue over the life cycle of the structure. The premium paid for lightweight concrete relative to normal weight concrete may range from 25 to 30 percent of the cost per cubic yard, which is expected to decrease with more use of this material. However, this increase is less than 10 percent considering the per cubic yard cost for in-place concrete. In the total cost of the bridge, the increase is much smaller, within a few percentage points.

Several benefits of LWHPC are realized immediately and are expected to offset this cost largely. The reduced dead load of LWHPC translates directly into longer spans, reduced pier or smaller piers, and reduced substructure requirements, resulting in large cost savings. The cost of transporting and erecting LWHPC superstructure elements would be significantly reduced relative to those of normal weight concrete. In bridge decks, the internal curing and the lower modulus of lightweight concrete are expected to minimize cracking, as observed in this study. The enhanced durability of LWHPC is expected to lead to extended service life with minimal maintenance costs. The lower initial cost because of the lighter weight concrete elements and the increase in the service life of the bridge because of the enhanced durability should result in a reduction in life cycle costs of at least 10 percent. The savings for VDOT should be greater than $20 million per year.

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