HIGH-PERFORMANCE CONCRETE
IN A BRIDGE IN RICHLANDS, VIRGINIA

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(The opinions, findings, and conclusions expressed in this report are those of the authors and not necessarily those of the sponsoring agencies.)

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ABSTRACT

The Virginia Department of Transportation built a high-performance concrete (HPC) bridge in Richlands. The beams had a minimum compressive strength of 69 MPa (10,000 psi) at 28 days and large strands, 15 mm (0.6 in) in diameter, placed 51 mm (2 in) apart. The deck concrete was designed to have a minimum compressive strength of 41 MPa (6,000 psi) and low permeability.

This report describes the development of the HPC mixes, the material and structural testing, the construction of the bridge, and the condition assessment of the bridge after two winters. Structural testing was conducted on two full-scale 9.5-m (31-ft) AASHTO Type II beams with the large-diameter strands and composite slabs. Pullout tests were conducted on the same strands as used in the beams. Tests were also conducted to determine if there were any residual phosphates on the strands since the presence of phosphates has been suspected to affect the bond between the strand and the hardened concrete.

The test program, field application, and in-service performance of the bridge indicated that HPC with high strength and low permeability can be produced using locally available material. The use of the large-diameter strands at a 51-mm (2 in) spacing also proved successful.

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INTRODUCTION

High-performance concrete (HPC) has enhanced durability and high strength, resulting in long-lasting and economical structures.[1] Most conventional concrete bridges deteriorate rapidly and require costly repairs before their expected service life is reached. Four major types of environmental distress affect concrete bridges: corrosion of the reinforcement, alkali-aggregate reactivity, freeze-thaw deterioration, and attack by sulfates.[2] In each case, water or solutions penetrate the concrete and initiate or accelerate damage. The HPC mixes designed for low permeability resist this infiltration of aggressive liquids and, therefore, are more durable. Two important issues need to be addressed in the use of HPC: the development of the mixes, and the use of strands 15 mm (0.6 in) in diameter for pretensioned applications.

Low-permeability concretes are made with a low (0.45 and less) water–cementitious material ratio (w/cm) and generally with a pozzolanic material such as fly ash, silica fume, or slag. These modifications to the mixes also result in higher compressive strengths than with conventional concretes, above 41 MPa (6,000 psi), which can lead to structures that are more economical. The initial economic benefit arises from the ability to use fewer beams, resulting in lower costs in materials, labor, transportation, and construction, and increased span lengths, necessitating fewer piers. The structural benefits include increased rigidity because of the increased elastic modulus (thereby reducing dynamic response) and increased concrete strength that raise the allowable design stresses.[3,4]

To benefit fully from the use of HPC, the larger prestressing strands in prestressed structural elements are needed to increase the strength of the members further and minimize strand congestion, which can occur with the use of a large number of smaller diameter strands. However, at the time of this project, the use of the larger strand for pretensioned applications was prohibited under a moratorium by the Federal Highway Administration (FHWA) pending further research.

During the 1995 and 1997 construction seasons, the Virginia Department of Transportation (VDOT) planned to construct seven bridges using HPC.[5] The program was expanded in 1998 to include 20 HPC bridges. The concrete used in these bridges had at least one of the two features of HPC, i.e., high strength or low permeability. The required strength of the concrete ranged from 49 to 69 MPa (7,000 to 10,000 psi) at 28 days. The low-permeability requirements with regard to the rapid chloride permeability test were a maximum coulomb value of 1500 for the prestressed concrete beams, 2500 for the cast-in-place deck concrete, and 3500 for concrete in the cast-in-place substructures.

The first HPC bridge in VDOT's program was built on Route 40 but without the use of the larger-diameter strands.[5] Since the FHWA was granting exceptions for the use of these strands if beam tests were successful, a test program was conducted for the second HPC bridge to be constructed in Richlands using HPC concrete and the larger-diameter strands. This report describes the development of the HPC mixes, material and structural testing, design and construction, and condition of the Richlands Bridge after two winters in operation.

PURPOSE AND SCOPE

The purpose of this study was to conduct research to support construction of the Richlands Bridge and to evaluate the field performance of the bridge once constructed. The scope of the work included:

- developing HPC mixes with a compressive strength exceeding 46 MPa (6,600 psi) within 20 hours and exceeding 69 MPa (10,000 psi) at 28 days and with a permeability less than 1500 coulombs

- testing for compressive strength, elastic modulus, modulus of rupture, splitting tensile strength, permeability, and drying shrinkage of the HPC mixes

- designing, fabricating, and testing two full-scale prestressed composite concrete beams with 15-mm (0.6-in) strands at a 51-mm (2-in) spacing and with a composite deck

- conducting pullout tests to determine the bond strength between the strands and the concrete

- measuring residual phosphates on the strands to assess their effect on bond performance

- constructing and instrumenting the bridge

- conducting a condition assessment of the bridge after two winters in operation.

To fulfill the purpose of the study, a test program was initiated to develop the HPC mixes and fabricate the test beams. Then, the bridge was designed, fabricated, and constructed in accordance with the findings of the test program.

TEST PROGRAM

Methodology

The high-strength, low-permeability concrete mixes were initially developed at the Virginia Transportation Research Council and then at the prestressing plant. After the trial batches, two AASHTO Type II beams with cast-in-place composite slabs were fabricated at the prestressing plant and transported to the FHWA's Turner-Fairbank Highway Research Center Structures Laboratory in McLean, Virginia, for testing. The girders were instrumented with thermocouples for measuring the temperature profile during fabrication and with mechanical strain gages and deflection gages to measure development length during testing. A concrete block containing untensioned strands was also prepared for pullout testing to evaluate bond strength. Residual phosphate on the strands was measured through a chemical test.

Material Testing

The concrete mixes were prepared with material normally available at the plant, except that Class F fly ash was used in one of the trial batches to improve workability. However, the evaluation of mixes with fly ash was discontinued since the plant did not store this material and workable concrete mixes with desired strengths could be made with the material at the plant. The cementitious material was a combination of Type I cement and silica fume. Silica fume conforming to the requirements of ASTM C 1240 was used in slurry form. Number 8 coarse aggregate with a nominal maximum size of 10 mm (3/8 in) was used: crushed limestone with a specific gravity of 2.76 and a dry rodded unit weight of 1589 kg/m^3 (99.2 lb/ft^3). The fine aggregate was crushed limestone with a specific gravity of 2.75 and a fineness modulus of 3.00. Commercially available air-entraining admixture; water-reducing admixture conforming to the requirements of ASTM C 494, Type A, or water-reducing and retarding admixture conforming to the requirements of ASTM C 494, Type D; and high-range water-reducing admixture, a combination of melamine and naphthalene condensates conforming to the requirements of ASTM C 494, Type F, were used.

Trial Batches

In the laboratory, three small trial batches, T1 through T3, measuring 0.04 m^3 (1.5 ft^3) were prepared. Then, a larger trial batch, T4, of 1.5 m^3 (2 yd^3) was prepared at the plant in a ready-mix concrete truck. The mixture proportions are given in Table 1. The specimens were subjected to accelerated curing after the time of initial set. In the laboratory, accelerated curing was maintained by subjecting sealed specimens to 55° C or 65° C (130° F or 150° F) in an environmental chamber for 16 hours, after an initial delay of 4 to 5.5 hours. At the plant, steam curing was used. After an initial delay of 7.5 hours (which was longer than the initial time of setting for convenience), the concrete mixes were steam cured for 12.5 hours. The concrete mixes were tested in the freshly mixed state for air content (ASTM C 231), slump (ASTM 143), and unit weight (ASTM C 138). After the high-temperature or steam curing, specimens were

Table 1. Mixture Proportions for Trial Batches

Material (kg/m^3)	T1	T2	T3	T4

Portland cement	415	446	454	454
Silica fume	30	31	45	45
Fly ash	89	0	0	0
Coarse aggregate	1068	1068	1068	1068
Fine aggregate	602	725	699	683
Water	160	144	145	155
w/cm	0.30	0.30	0.29	0.31
High-range water-reducing admixture (mL/kg)	9.8	13.4	15.3	12.4
Water-reducing and retarding admixture (mL/kg)	2.0	2.0	2.0	
Water-reducing admixture (mL/kg)				2.3

kept in air or in the moist environment and tested in compression. Some specimens were moist cured only at room temperature. Concrete in batches T1 through T3 was tested for compressive strength and that in batch T4 for compressive strength and elastic modulus at various times up to 56 days. The compressive strength was determined in accordance with AASHTO T 22 using neoprene pads in steel end caps, and the elastic modulus was determined in accordance with ASTM C 469.

Beam and Slab Mixes

After the trial batches, two 9.5-m (31-ft) AASHTO Type II beams were cast with composite slabs. For each beam, one 3.1 m^3 (4 yd^3) batch of concrete was prepared. Both slabs were cast from one batch of concrete measuring 5.0 m^3 (6.5 yd^3). The concrete mixes were prepared in a ready-mix concrete truck with mixture proportions as given in Table 2. The mixes were tested for air content and slump in the freshly mixed state. Type T thermocouples were attached to various points in the test beams to measure the temperature of the concrete. In addition to the test beams, other specimens were cast for testing at the hardened state as indicated in Table 3. Temperature-matched cured (TMC) cylinders were also prepared for compressive strength tests. The TMC cylinders are cast in molds with heating elements and followed the same temperature rise as the beams. Specimens representing concrete for the beams were steam

Table 2. Mixture Proportions for Test Beams

Materials (kg/m^3)	B1 (Beam 1)	B2 (Beam 2)	S (Slab)
Portland cement	454	454	377
Silica fume	45	45	27
Coarse aggregate	1068	1068	1068
Coarse aggregate size	No. 8	No. 8	No. 67
Fine aggregate	715	715	899
Water	139	139	145
w/cm	0.28	0.28	0.36
High-range water-reducing admixture (mL/kg)	16.3	16.3	18.3
Water-reducing and retarding admixture (mL/kg)	2.0	2.0	2.0

Table 3. Tests and Specimen Sizes

Test	Specification	Size (mm)	Test Age
Compressive strength	AASHTO T 22[a]	100 x 200	1, 28, 56 d and 1 yr
Flexural strength	ASTM C 78	75 x 75 x 285	28 d
Splitting tensile strength	ASTM C 496	100 x 200	28 d
Elastic modulus	ASTM C 469	150 x 300	28 and 56 d
Permeability	AASHTO T 277	50 x 100	28 d, 1 yr
Drying shrinkage	ASTM C 157	75 x 75 x 285	11 mo

[a]Except that neoprene pads in steel end caps were used for capping.

cured for 12 hours, after an initial delay of 7 hours for beam 1 and 5.3 hours for beam 2. Both beams were cured in the same bed. The initial delay is usually longer than the initial time of setting because the steam is turned on when the last batch of concrete attains the initial time of set (the earlier batches would be past at this stage). After steam curing, the specimens were cured outdoors with two exceptions: the acceptance specimens were kept in the moist room after steam curing and tested at 28 and 56 days and the TMC specimens were stripped and moist cured after steam curing and tested at 56 days. For specimens cured outdoors, compressive strengths were determined at the end of the curing period, at 28 and 56 days, and at 1 year.

Composite slabs measuring 200 mm (8 in) in depth and 1.3 m (4 ft) in width were cast when the beams reached a strength of 69 MPa (10,000 psi). The beams were simply supported when these slabs were cast. After screeding, these slabs were covered with wet burlap and heavy plastic and kept moist for 7 days using soaker hoses. Specimens representing the slab were kept moist at room temperature in the laboratory after a day in the field.

The splitting tensile strength was determined at 28 days in accordance with ASTM C 496. For the permeability test, the top 51 mm (2 in) of the 100 x 100 mm (4 x 4 in) cylinders was cut and tested at 28 days and 1 year. The specimens were tested in accordance with AASHTO T 277 and moist cured at room temperature, except for a set of two permeability specimens for the slab that were kept at 38° C (100° F) the last 3 weeks prior to testing at 28 days.

The drying shrinkage test was conducted in accordance with ASTM C 157. However, concrete specimens representing the beam were stored outdoors for the first 6 months and then air dried in the laboratory for an additional 5 months. The specimens representing the slab were moist cured for 28 days and then air cured.

A concrete block measuring 0.6 x 0.6 x 0.9 m (2 x 2 x 3 ft) containing eight untensioned strands 15 mm (0.6 in) in diameter was cast for the pullout tests to determine bond strength. The strands used in this block came from the same reel as used for the girders. A schematic of the block is shown in Figure 1.

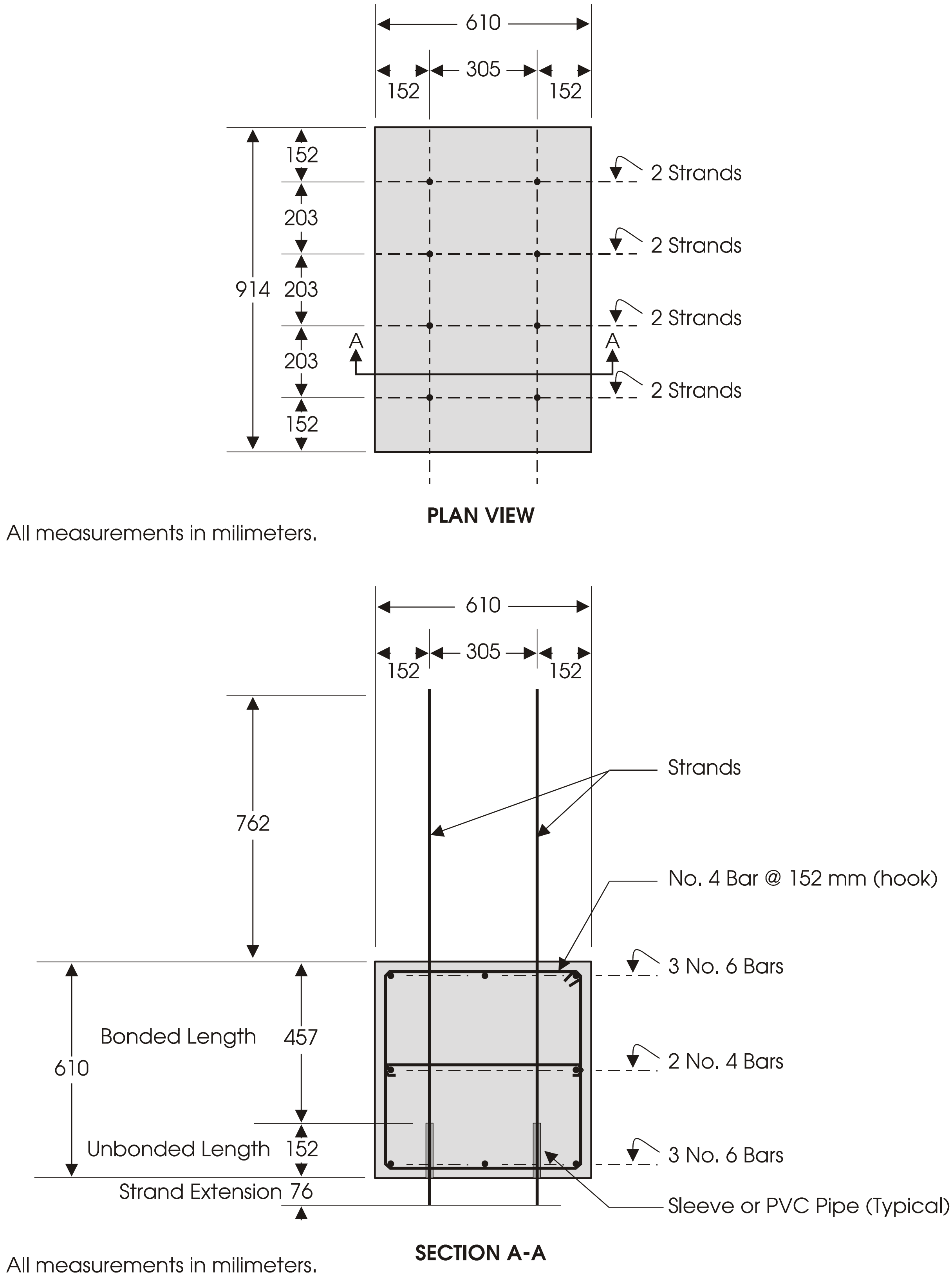

Figure 1. Concrete Block Used for Pullout Tests

Structural Design and Testing

Beam Design

The two 9.5 m (31 ft) AASHTO Type II girders were designed in accordance with AASHTO and VDOT specifications. They were designed assuming a minimum concrete compressive strength of 69 MPa (10,000 psi). Each girder contained eight prestressing strands with six straight and two draped strands as shown in Figure 2. The strands were uncoated, 15-mm (0.6 in) diameter seven-wire, low-relaxation strands placed at 51 mm (2 in) center-to-center spacing and conformed to the requirements of ASTM A 416, Grade 270. All reinforcing steel was uncoated Grade 60 conforming to the requirements of ASTM A 615.

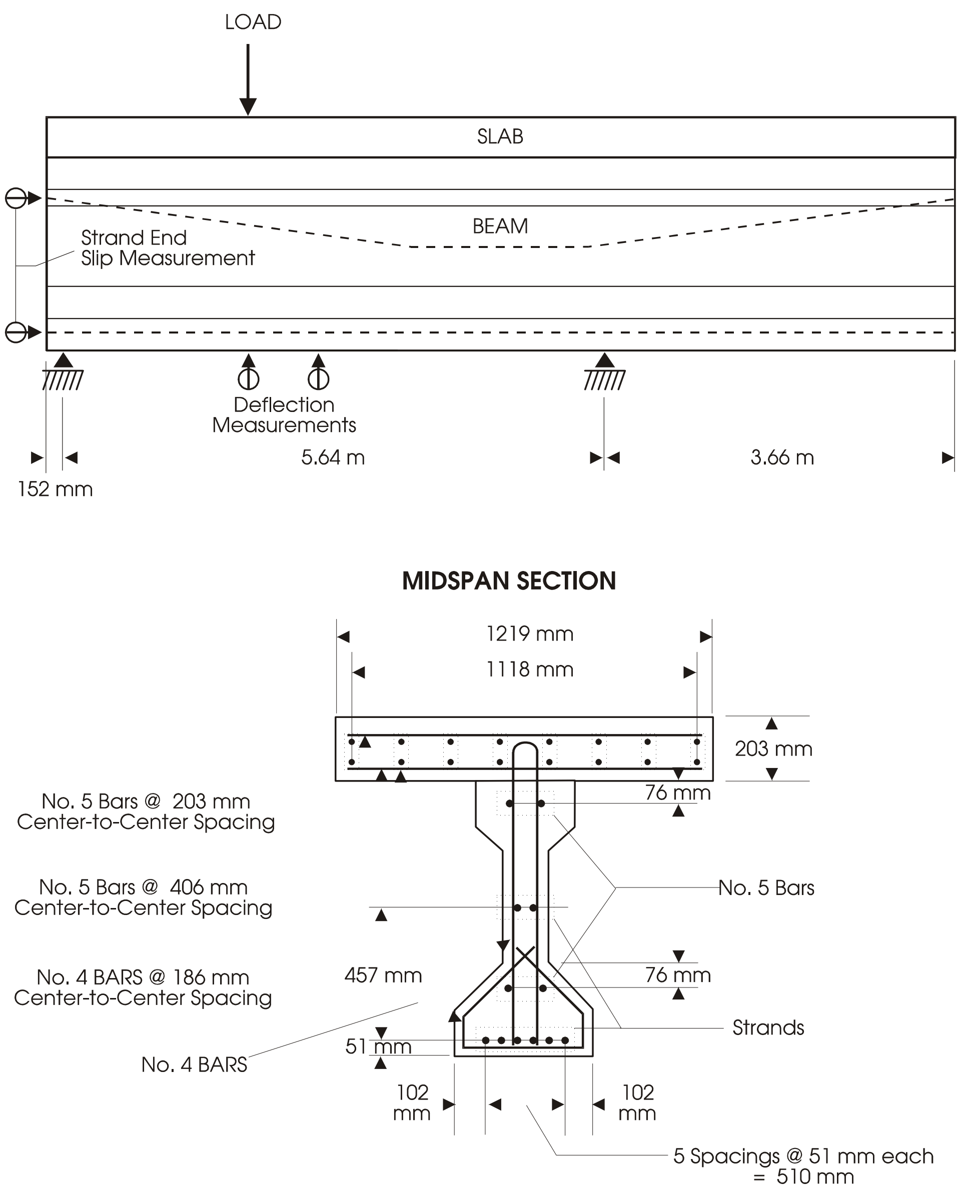

Figure 2. Test Beam and Laboratory Setup

A full-length concrete slab with a depth of 0.2 m (8 in) and a width of 1.3 m (4 ft) for each girder was designed to simulate composite action as in a real structure. The design of the slabs was based on a minimum strand strain of 0.030 to 0.035 at failure. The minimum strand strain to failure for strands conforming to the requirements of ASTM A416 is 0.035. A recent FHWA report indicated that pretensioned members with strand strains greater than 0.025 at failure of the member need greater embedment lengths than currently required by AASHTO Equation 9-2.[6] To ensure a conservative determination of the development length, the slab was designed accordingly. The minimum design compressive strength of the concrete in the slab was 41 MPa (6,000 psi).

Instrumentation

The girders were instrumented during fabrication and before the start of the structural testing. Type T thermocouples were attached to various points in the test beams to measure the temperature of the concrete (Figure 3) during fabrication as mentioned previously. For the transfer length measurements, mechanical Whittemore gage studs were embedded in the lower flange of the beams. The studs were attached to the metal formwork prior to the concrete placement. These brass studs were spaced at intervals of 100 mm (4 in) for a length of 1.8 m (6 ft) from both ends and 200 mm (8 in) in between along the path of the bottom strands. Additionally, the test beams were configured with end slip channel gages to measure slip of the tendons at the time of detensioning and periodically thereafter.

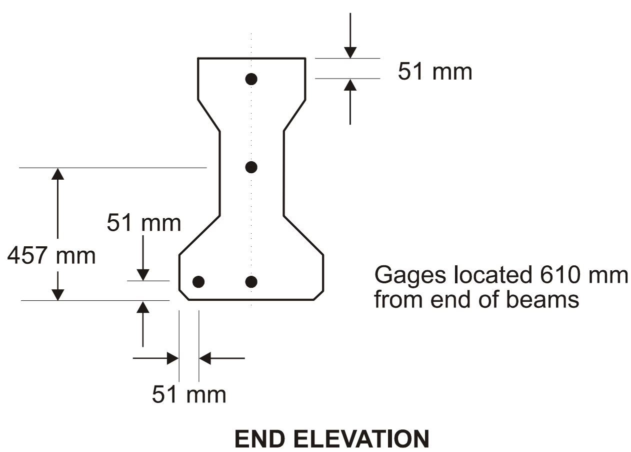

Figure 3. Location of Temperature Gages in Test Beams

Since end slippage of the prestressing tendons is indicative of bond failure, during load testing, linear variable differential transformers (LVDTs) were placed on the exposed strand ends to measure any slippage during the load test, as shown in Figure 4. The LVDTs were also placed under the test beam, at the position of the load and at the span centerline to measure the deflection during loading. Two electrical resistance strain gages were placed on the surface of the slab, at either end of the actuator bearing, to monitor concrete strains during the loading.

Figure 4. Measurement of End Slippage by LVDTs

Structural Testing

For structural testing of the girders, loads were applied by hydraulic jacks and monitored by attached load cells. Loads were applied incrementally, and the transducers were monitored continuously. The mode of failure was documented. The test beams were loaded so as to determine the development length of the prestressing tendons. By use of the test setup shown in Figure 1, it was possible to conduct two tests for each girder. After testing one end of the girder, the girder was rotated and the other end tested. Using this scheme, four tests could be conducted with the two beams.

The prestressing strand transfer length was experimentally evaluated for the two beams by measuring change in concrete surface strains with the Whittemore gages. Reference gage readings were taken just before and just after release of the prestressing force and several times before testing. End slip readings were also taken periodically using a depth micrometer and end slip brackets. These readings measured the distance each strand slipped into the concrete after stress transfer.

Since development length cannot be directly measured, it was obtained indirectly by loading the beams at points of assumed embedment lengths. The *embedment length* is defined as the length of bonded tendon from the end of the member to the location of maximum moment. The tendons are fully developed at some point between the beam end and centerline.

If adequate shear reinforcement is provided, the failures can be either flexural or bond failures. In flexural failure, the embedment length allows the strands to yield, eventually crushing the concrete in the extreme compression fibers. In bond failure, the bond between the concrete and the prestressing tendons fails, resulting in a sudden shear. Bond failure occurs if the embedment length is not sufficient to develop the stress in the prestressing tendons.

If the assumed embedment length was adequate, during testing, the tendons would yield and the failure mode would be crushing of the concrete in the top flange. If a bond failure occurs, the embedment length, and hence the point of application of the load, should be increased for the next test. If a flexural failure occurs, the embedment length should be decreased for the next test until a balanced failure condition (simultaneous bond and flexural failure) is reached. The sum of the embedment length at balanced failure and the transfer length is defined as the development length.

Pullout tests were conducted as follows. A hydraulic jack and load cell were clamped to one strand on the top surface of the block. The load cell was placed between the hydraulic jack and top surface of the concrete block, and an LVDT was attached to the strand on the bottom surface. Load was applied to the strand by the hydraulic jack and controlled and monitored by the load cell. Strand slip was detected by the LVDT and also continuously monitored. Load was applied until strand failure in tension or bond failure resulting in complete slip through the block.

Researchers have speculated that there may be negative effects on bond performance from chemical residues left on the strands when these strands are manufactured. There may also be residual material other than phosphates on the strands, but tests are available only for phosphates. Hence, a chemical test was conducted to determine any residual phosphate on the strands.[7] in accordance with descriptions in *Ferrous Wire* (The Wire Association International, Inc).

Results and Discussion

Material Testing

Trial Batches

The air content, slump, and unit weight values for the trial batches are given in Table 4. Batches T1 through T3 met the air content requirement of 5.5 ∀1.5 percent for the prestressed members containing high-range water-reducing admixture, but T4 had a lower air content. Slump values were above 90 mm (3.5 in) and were adequate for proper consolidation.

Table 4. Characteristics of Freshly Mixed Concrete for Trial Batches

Items	T1	T2	T3	T4
Air content (%)	5.6	4.9	4.3	3.0
Slump (mm)	165	90	130	100
Unit weight (kg/m^3)	2352	2403	2397	---

The compressive strengths given in Tables 5 and 6 indicate that high early strengths exceeding 46 MPa (6,600 psi) and 28-day strengths exceeding 69 MPa (10,000 psi) can be

Table 5. Compressive Strength of Trial Batches (T1-T3)(MPa)

Age (d)	Cure (°C)	T1	T2	T3
1	65	47.4	58.3	63.4
1	55	44.5	53.5	61.2
1	23	27.0	34.3	36.5
14	65	53.1	65.8	68.0
14	55	54.7	66.3	71.8
14	23	64.5	78.4	86.6
28	65	57.0	64.1	72.3
28	55	57.2	71.9	70.3
28	23	75.9	86.1	94.1
56	65	58.5	68.4	
56	55	56.0	68.1	

Table 6. Compressive Strength and Elastic Modulus of Trial Batch (T4)

Age (d)	Cure	Compressive Strength (MPa)	Elastic Modulus (GPa)
1	MC	26.5	
2	Steam + MC	53.4	40.4
7	Steam + MC	55.2	42.7
7	Steam + Air	55.3	
7	MC	61.5	
14	Steam + MC	59.0	42.8
28	Steam + MC	61.0	
28	Steam + Air	59.7	
28	MC	82.5	
56	Steam + MC	63.6	
56	Steam + Air	59.9	
56	MC	79.8	

MC = moist cured.

obtained with a w/cm less than 0.30. The elastic modulus values for T4 were close to 41 GPa (6 million psi) at 2 days and higher afterward.

Beams and Slabs

The air content and slump of the concrete batches for the beams and slab are given in Table 7. The specified air content for the deck concrete was 6.5 ± 1.5 percent. The air contents were in accordance with the specification. The slumps exceeded 120 mm (4.7 in), which were adequate for proper consolidation.

Table 7. Characteristics of Freshly Mixed Concrete for Test Beams and Slab

Characteristic	B1	B2	Slab
Air content (%)	4.0	5.3	5.0
Slump (mm)	120	175	180

The properties of hardened concrete are shown in Table 8. Within 19 hours of steam curing, compressive strengths of 58.7 MPa (8,520 psi) for B1 and 55.0 MPa (7,970 psi) for B2 were obtained from cylinders kept inside the enclosure under the cover in the steam bed. However, TMC cylinders had higher strengths, 68.1 MPa (9,880 psi) for B1 and 61.4 MPa (8,900 psi) for B2. The temperature development for the first beam is given in Figure 5 and shows that peak temperatures close to 71° C (160° F) were reached. Unfortunately, because of a malfunctioning recorder, the enclosure and cylinder temperatures were not available for the first beam. This data and beam temperatures were obtained for the second beam, which was in the same steam bed (Figures 6 and 7). Figure 6 shows the temperature drop in the beam (tracked by the temperature of the cylinder in the enclosure) after the steam was turned off or reduced at 8 hours. However, the beam and TMC cylinders shown in Figure 7 were unaffected by this change. The higher temperature or maturity in the TMC specimens relates to the higher early strengths compared to those for the cylinders in the enclosure. The 28-day compressive strengths were higher than 69 MPa (10,000 psi). Cylinders that were moist cured after steaming had strengths similar to those for the cylinders kept outdoors for this particular set. At 1 year, strengths were 76.9 MPa (11,150 psi) or more.

The 1-day compressive strength of the concrete in the slab was 27.8 MPa (4,030 psi); the concrete in the TMC cylinders had higher strength, 31.4 MPa (4,550 psi). The 28-day and 1-year strength of the moist-cured slab concrete was higher than the steam-cured beam concrete even though the w/cm was higher for the slab concrete, 0.36 versus 0.28. This indicates the harmful effects of high early temperature on the ultimate strength of concrete.

The minimum splitting tensile strength was 5.14 MPa (745 psi) for B1 at 28 days, and the flexural strength was 8.31 MPa (1,205 psi) for the slab at 28 days. The elastic modulus was

Table 8. Properties of Hardened Concretes for Test Beams

Test	Age (d)	Cure	B1	B2	Slab
Compressive strength (MPa)	1	TMC	68.1	61.4	31.4
	1	Steam	58.7	55.0	
	1	MC			27.8
	28	Steam + Air	70.1	73.2	
	28	Steam + MC	70.5	70.5	
	28	MC			81.2
	56	Steam + MC	74.9	75.2	
	56	TMC		74.5	
	1 yr	MC			96.9
	1 yr	Steam + Air	76.9	79.2	
Elastic modulus (GPa)	28	Steam + MC	42.5	42.6	44.3
	56	Steam + MC	44.3	42.7	
Splitting tensile strength (MPa)	28	Steam + MC	5.14	5.45	6.48
Flexure (MPa)	28	Steam + MC	8.48	9.48	8.31
Permeability (coulombs)	28	Steam + MC	159	152	
	28	MC			753
	28	MC (3 wk at 38° C)			310
	1 yr	MC			321
	1 yr	Steam + MC	194	194	

TMC: Temperature-matched cure; MC = moist cure.

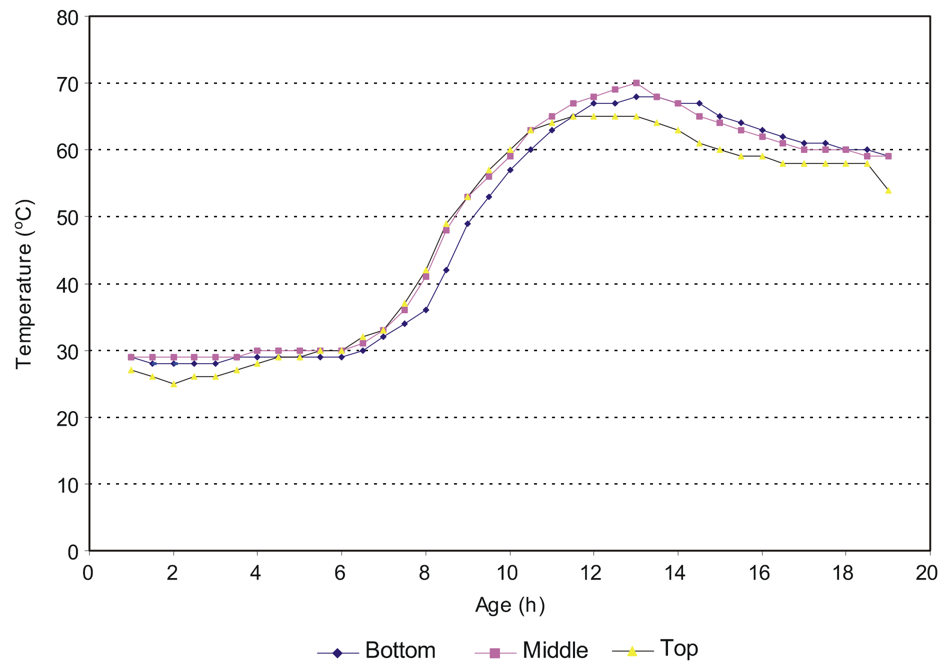

Figure 5. Temperature Development in Test Beam 1

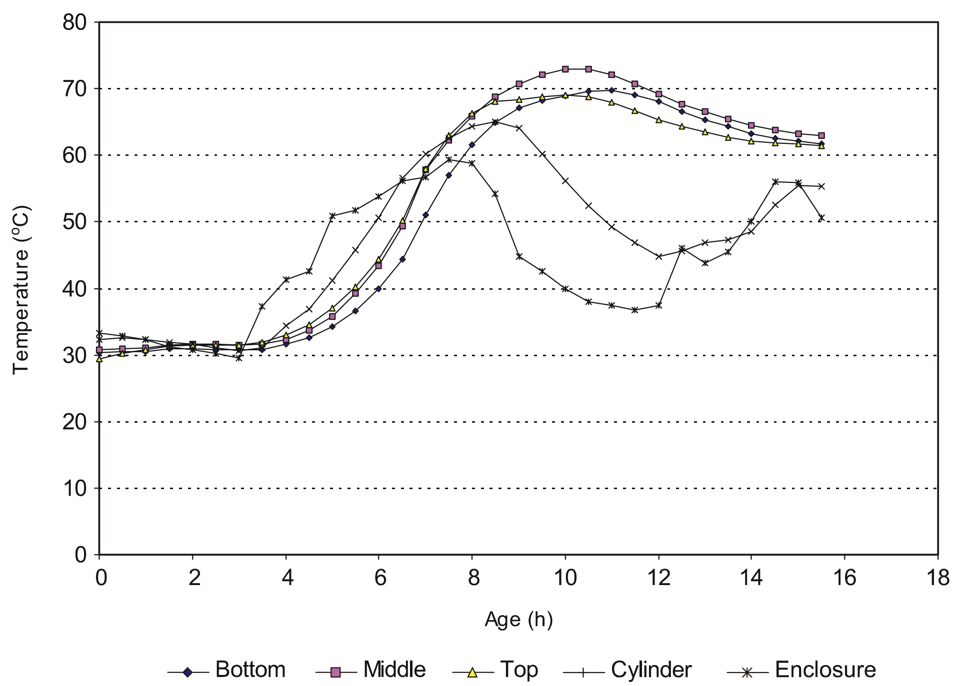

Figure 6. Temperature Development in Test Beam 2

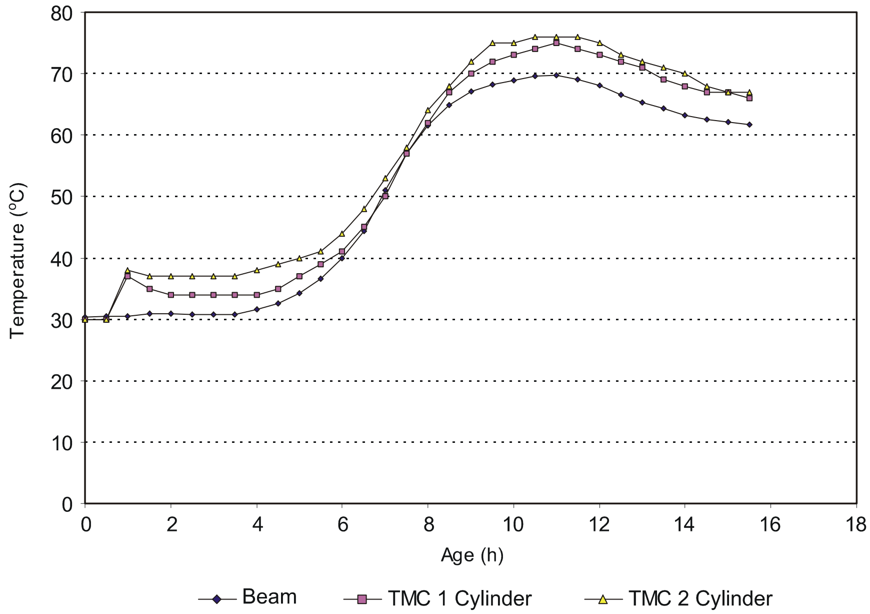

Figure 7. Temperature Development in TMC Cylinders

above 41 GPa (6 million psi) for all mixes. The permeability values at 28 days and 1 year were very low, less than 200 coulombs for beams, much lower than the required maximum value of 1500 coulombs. The slab concrete, which was cured at 23° C (73° F) for 1 week and then at 38° C (100° F) for 3 weeks, had a permeability value of 310 coulombs at 28 days, which was also much lower than the 2500 required under the new VDOT special provisions. However, the coulomb values for this same mix when cured at room temperature was 753 at 28 days and 321 at 1 year. The drying shrinkage values for the beam concrete are summarized in Table 9 and displayed in Figure 8. The values were 0.0410 percent for B1 and 0.0500 percent for B2 at 11 months (6 months outdoors and 5 months in the laboratory) and are considered satisfactory. In the outdoor exposure, shrinkage values fluctuated with time because of the moisture variations. The drying shrinkage values for the test slab are displayed in Figure 9. At 1 year, shrinkage was less than 0.05 percent, which is considered satisfactory.

Table 9. Drying Shrinkage of Test Beams (%)

Age	Test Specimen 1	Test Specimen 2
7 d	0.0157	0.0203
28 d	0.0170	0.0237
2 mo	0.0107	0.0187
4 mo	0.0183	0.0270
6 mo	0.0097	0.0187
8 mo	0.0267	0.0350
10 mo	0.0317	0.0353
11 mo	0.0410	0.0500

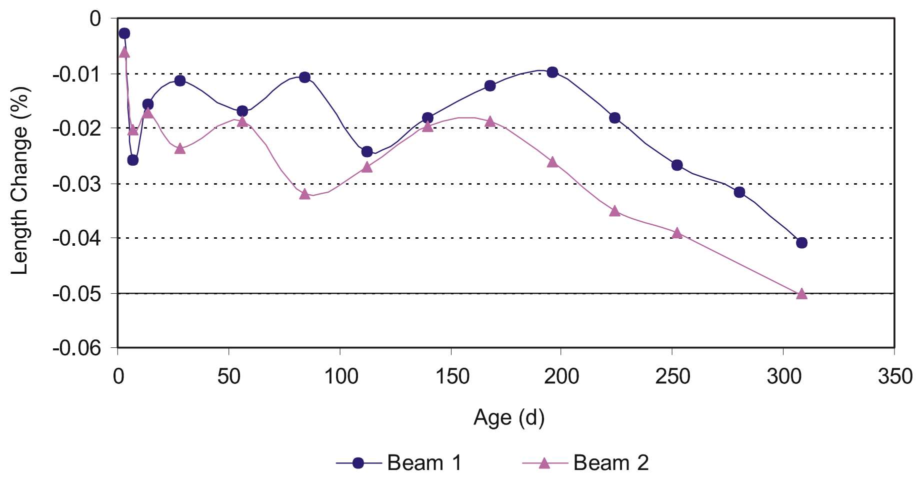

Figure 8. Length Change Data for Test Beams

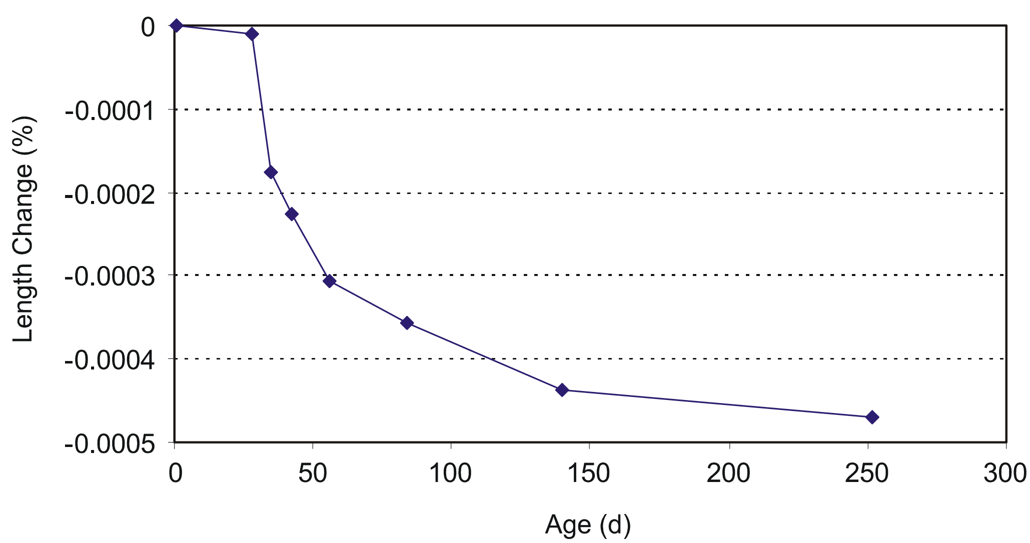

Figure 9. Structural Testing

The transfer and development lengths were successfully determined (Table 10). The results of the theoretical calculations and actual cracking and ultimate loads during testing are given in Table 11. The actual cracking and ultimate loads were the result of the structural testing, and the theoretical cracking and ultimate load determination was arrived at by strain-compatibility analysis. Strain-compatibility analysis yields a moment value. Through knowledge of the support conditions, span length, and load location, statics were used to determine the theoretical values reported in Table 11.

To determine the transfer lengths, raw strain data were placed in a spreadsheet program to produce a concrete strain profile for each end of each beam. A smoothing technique was used prior to the determination of the transfer length from the strain profile. In this technique, a width

Table 10. Transfer Length of Test Beams

Beam	Transfer Length, *L* (mm)	Load Location From End (m)
1 end A	555	2.36
1 end B	325	2.06
2 end A	389	1.75
2 end B	269	1.75

Table 11. Theoretical and Actual Cracking and Ultimate Loads (kN) for Test Beams

Beam	Theoretical Cracking	Actual Cracking	Theoretical Ultimate	Actual Ultimate
1 end A	890	979	1334	1601
1 end B	1170	1334	1655	1913
2 end A	979	1201	1815	1913
2 end B	899	1245	1815	2180

of three data points was averaged in an overlapping manner. The strain profiles are shown in Figures 10 through 13.

To determine the transfer length from the smoothed strain profiles, the plateau strain value was visually determined. A horizontal line was then drawn parallel with the *x*-axis, at the average of the strain values beyond the plateau point. This horizontal line represents the mean value of the concrete surface strain between the zones of prestress transfer. Another horizontal line was drawn at 95 percent of this mean value. Two vertical lines were drawn, connecting the intersection of the mean and 95 percent mean values with the smoothed strain profile curve and the *x*-axis. Both 95 and 100 percent prestress transfer lengths were determined. The 95 percent values are reported in Table 10 and were used in the structural testing for the evaluation of development length.

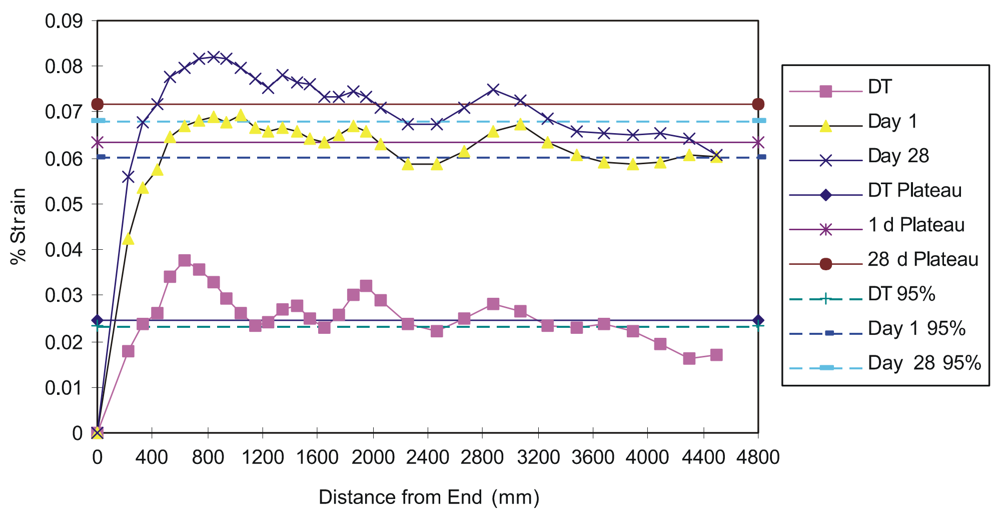

Figure 10. Concrete Surface Strain for Transfer length Determination of Test Beam 1, End A

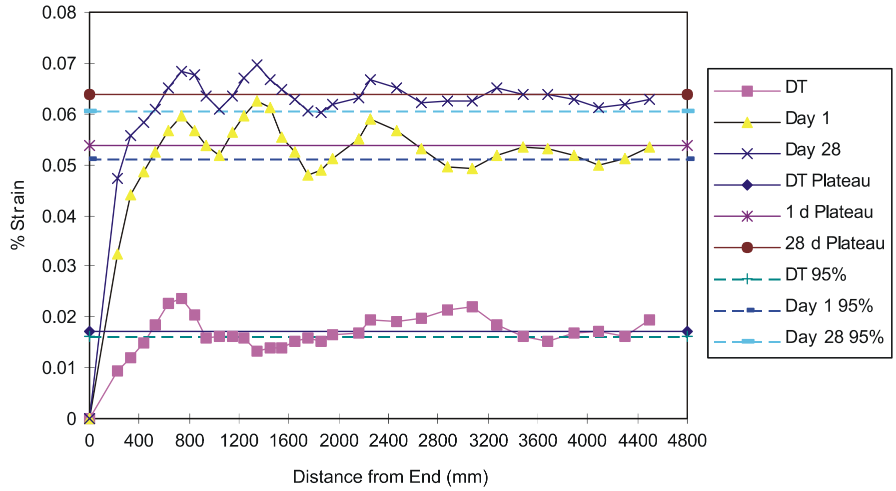

Figure 11. Concrete Surface Strain for Transfer length Determination of Test Beam 1, End B

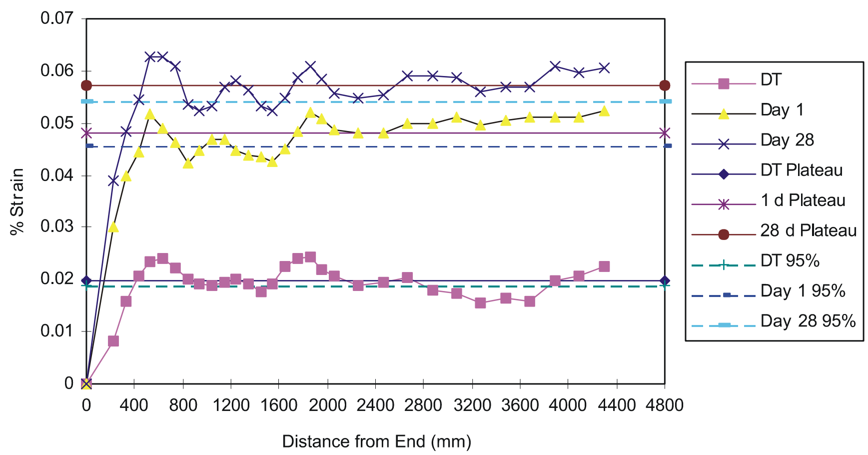

Figure 12. Concrete Surface Strain for Transfer Length Determination of Test Beam 2, End A

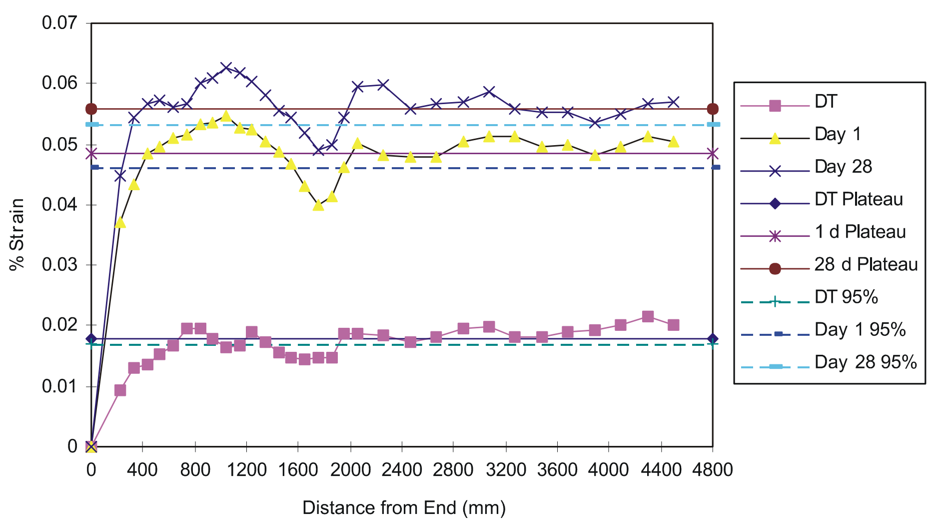

Figure 13. Concrete Surface Strain for Transfer Length Determination of Test Beam 2, End B

The load for the first test was placed at 2.36 m (93 in) from the beam end. This distance corresponds to the development length predicted in accordance with AASHTO Equation 9-32 for the 15 mm (0.6 in) strands used in the test beams. The equation is as follows:

$$L_d = (f_{su}^* - 2/3 f_{se})D$$

where L_d = development length

f_{su}^* = stress in prestressed reinforcement at nominal strength

f_{se} = effective stress in prestressed reinforcement after all losses

D = nominal diameter of prestressing strand.

Hence, a flexural failure was expected, with little or no strand slippage. However, several strands slipped at loads well below the predicted flexural capacity. Figure 14 shows the strand notation for the end slip evaluation. Strands 5 and 6 slipped at approximately 400 kN, well in the expected service range of the beam, as shown in Figure 15. Strand 4 slipped shortly thereafter, at about 590 kN. All of the strands in the bottom, with the exception of strand 1, slipped at 640 kN, again, within the expected design service range. Shear cracks appeared early, at around 810 kN, propagating from the end support. The first flexural crack occurred at 979 kN, producing a cracking moment of 1343 kN-m, which is 3 percent lower than the predicted value of 1387 kN-m, as determined by strain-compatibility analysis. Plots of load versus midspan deflection are shown in Figures 16 through 19. The initial cracking and strand slippage were attributed to poor consolidation at the beam end during casting. One indication of poor consolidation was the exceptionally long transfer length, as compared with the other transfer length measurements in Table 10. This conclusion was further verified when the pullout tests were conducted on the concrete block.

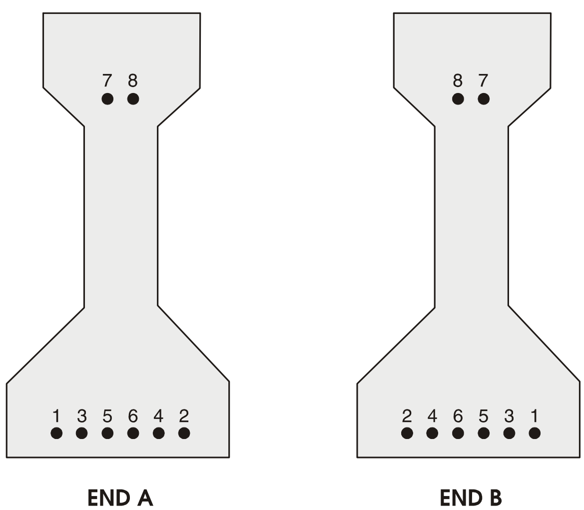

Figure 14. Strand Notation for End Slip

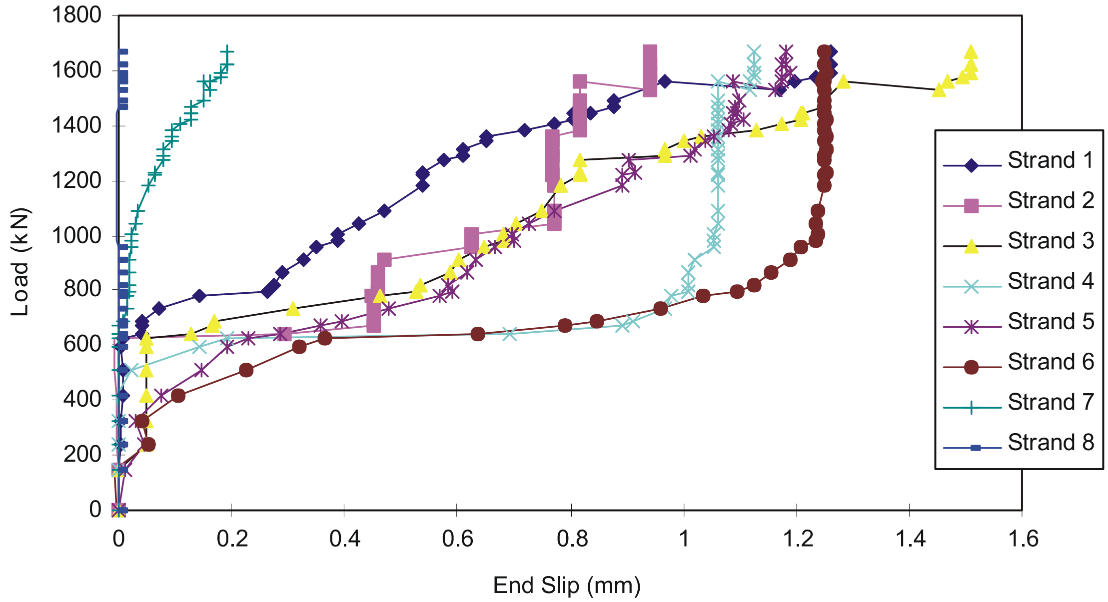

Figure 15. End Slip Versus Load for Test Beam 1, End B

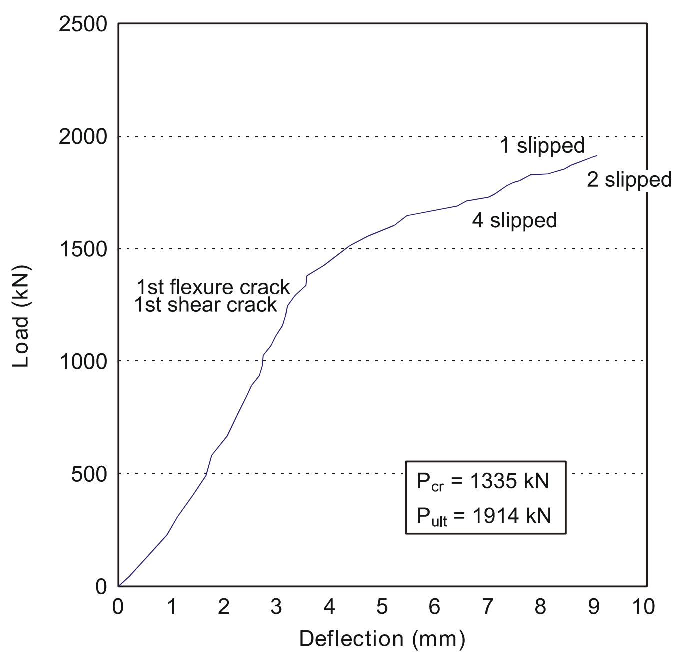

Figure 16. Load Versus Midspan Deflection for Test Beam 1, End B

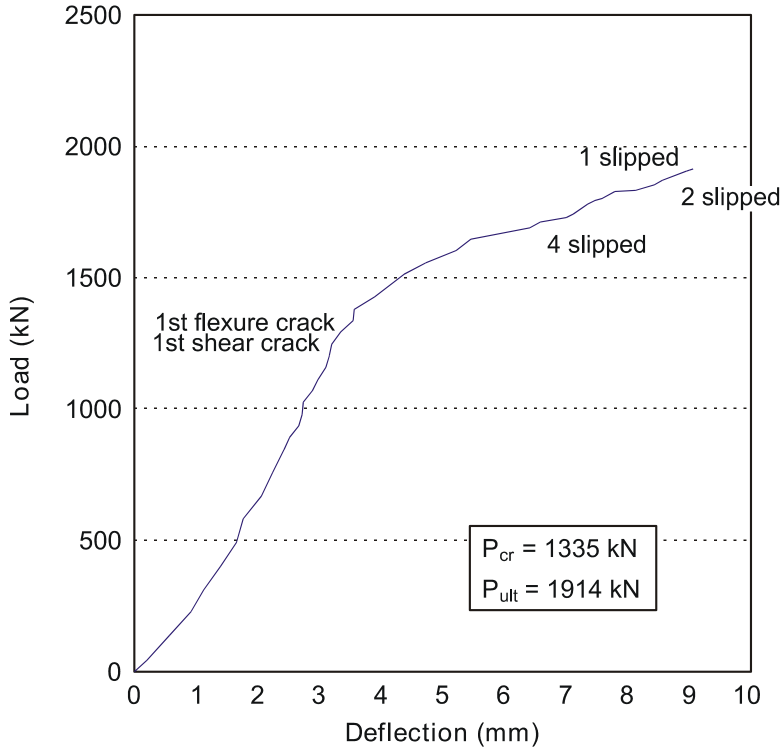

Figure 17. Load Versus Midspan Deflection for Test Beam 1, End A

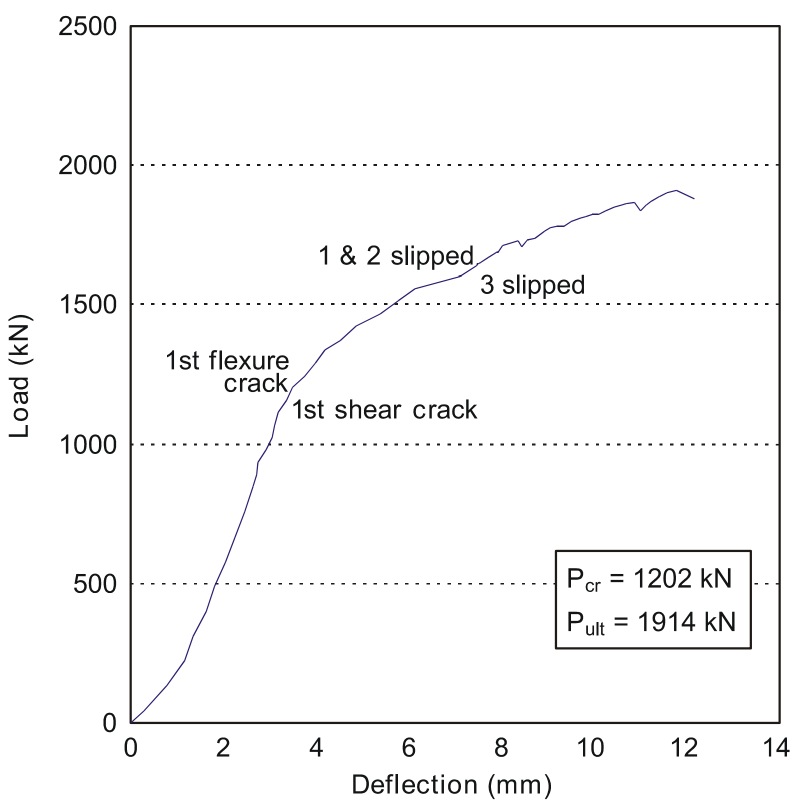

Figure 18. Load Versus Midspan Deflection for Test Beam 2, End B

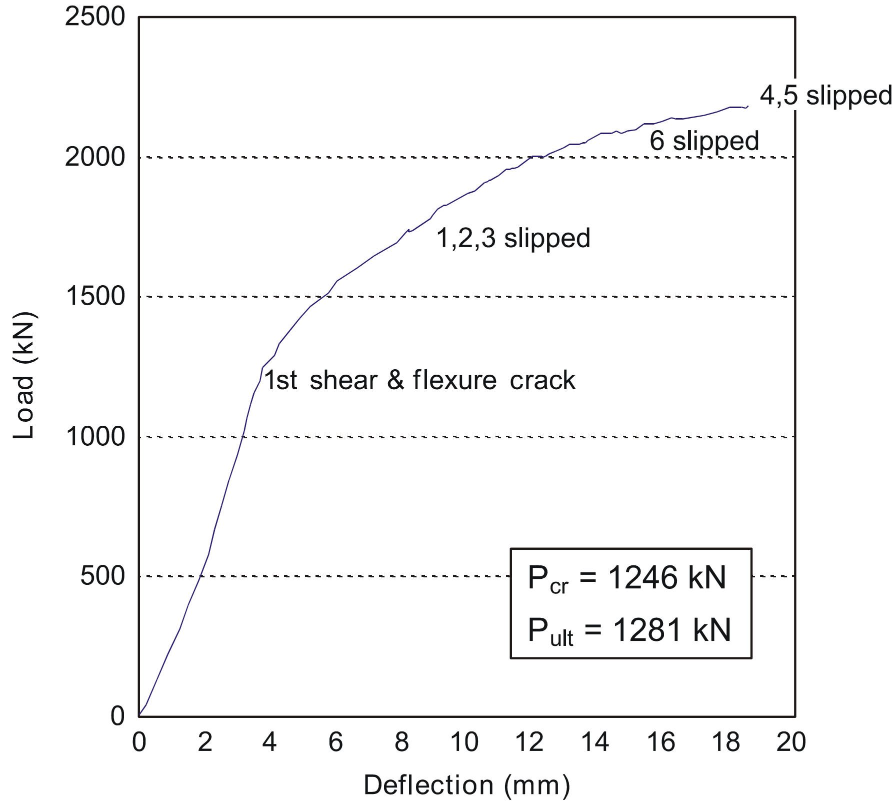

Figure 19. Load Versus Midspan Deflection for Test Beam 2, End A

The beams were designed with a maximum amount of shear reinforcing at the ends, making consolidation at the ends difficult. The test beams were consolidated with internal vibrators. Even though slip occurred unexpectedly, the beam was able to carry loads in excess of the maximum design load, reaching an ultimate moment of 2198 kN-m, 15 percent above the predicted (theoretical) value of 1904 kN-m. The strand slippage did stabilize, as shown in Figure 15, thereafter behaving as expected. As observed in Figure 15, strands 1 and 6 reached a maximum value of 1.26 mm at the peak load. Strands 2, 4, and 5 reached maximum values of 0.95, 1.13, and 1.21 mm at the peak load. Strand 7 slipped at 1512 kN, reaching a maximum value of 0.19 mm at the peak load. The deflection at the maximum load was 14.8 mm. Cracking extended into the deck slab, but testing was terminated before a crushing failure occurred because of safety concerns.

For the second test, the first beam was turned around and the opposite end was tested. The load was placed at 2.05 m from the beam end. The simple span was reduced to 4.3 m because of the extensive cracking resulting from the first test. Typically, the 5.64-m simple span length is preferred to ensure that cracking from the first test does not extend into the critical end for the second test. However, because of cracking from the first test, this span length could not be achieved for this second test. As observed in Figure 17, the maximum sustained load was 1914 kN, producing a maximum moment of 2500 kN-m, 16 percent over the predicted value of 1977 kN-m (Table 11). The corresponding maximum deflection was 9.7 mm. As observed in Figure 20, no appreciable strand slippage (as defined as any slippage exceeding 0.1 mm) occurred until approximately 1690 kN, when strand 4 slipped. Prior to this, a shear crack was detected at a load of 1245 kN, and shortly thereafter, a flexural crack was observed at a load of 1334 kN, producing a cracking moment of 1744 kN-m. The predicted cracking moment for this test setup is 1423 kN-m, 22 percent lower than the actual value. Strand 2 slipped at 1868 kN. At 1914 kN, strand 1 slipped. All three strands continued to slip as the load was increased to the peak value, with strand 4 reaching a maximum value of 0.41 mm. Strands 1 and 2 reached values of 0.14 and 0.29 mm, respectively, at the peak load. The test was terminated at this point because of safety concerns of a sudden failure. This loading was 16 percent higher than the theoretical value (Table 11). A compression failure was imminent because cracking extended into the deck, but because of the high-strength concrete in the deck, a violent compression failure was anticipated. Loss in stability of the beam while under load was of primary concern and, thus, the test was terminated.

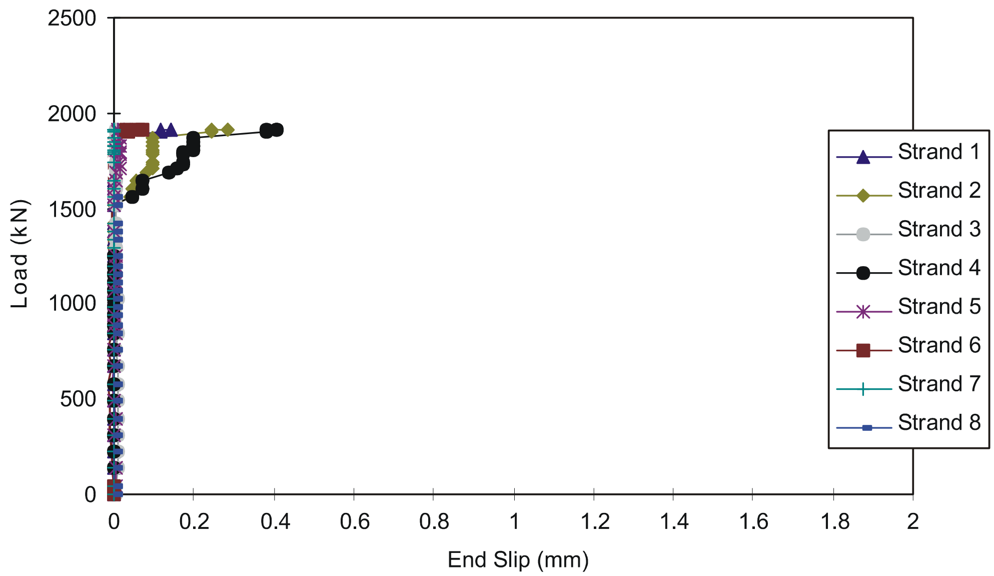

Figure 20. End Slip Versus Load for Test Beam 1, End A

The second test beam was placed in the load frame. The third structural test was conducted, with a load placed at 1.75 m from the beam end. As observed in Figure 18, the maximum sustained load was 1914 kN, producing a moment of 2309 kN-m, which is 8 percent over the predicted value of 2132 kN-m (Table 11). The corresponding maximum deflection was 14.5 mm. As observed in Figure 21, no appreciable strand slippage (again, as defined by slippage exceeding 0.1 mm) occurred until approximately 1601 kN, when strands 1, 2, and 3 slipped. Prior to this, a shear crack was detected at 1156 kN, and shortly thereafter, a flexural crack was observed at 1202 kN, producing a cracking moment of 1450 kN-m. The predicted cracking moment was 1401 kN-m, which is 3.5 percent less than the actual value. The maximum applied load was 1914 kN. Strands 1 and 3 continued to slip as the load was increased to its peak value, reaching maximum values of 1.02 and 0.95 mm, respectively. Strands 4 and 6 slipped at approximately 1700 kN, joining strand 2 in reaching a maximum value of 0.25 mm at the peak

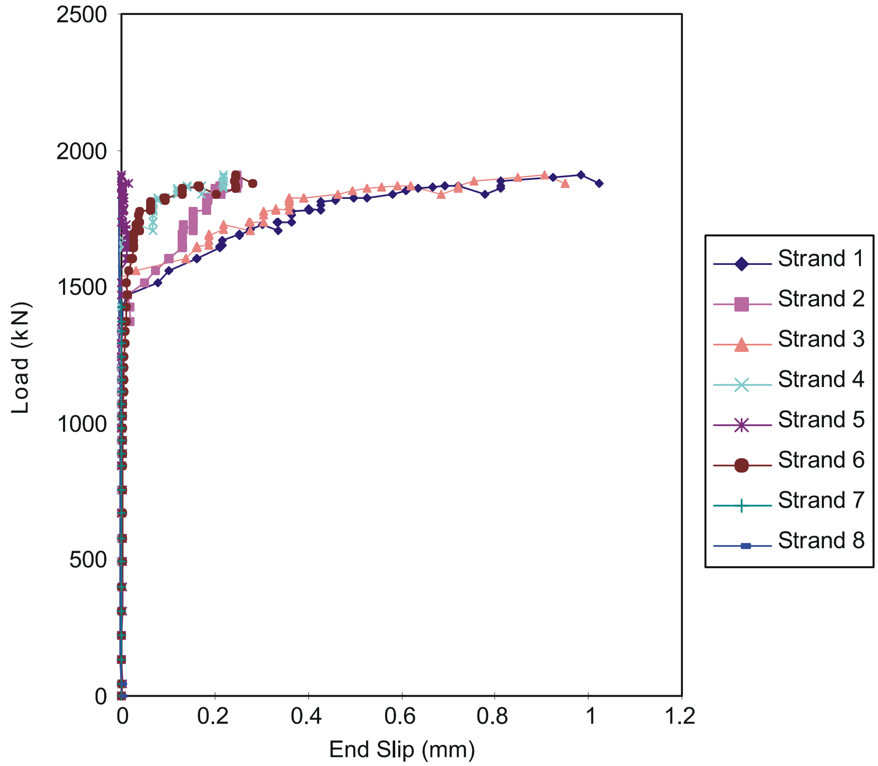

Figure 21. End Slip Versus Load for Test Beam 2, End B

load. Again, the test was terminated at this point because of safety concerns. A compression failure was imminent because cracking was extending into the deck. Significant strand slippage occurred in this third test, just before reaching ultimate strength, indicating that the minimum development length was 1.75 m (69 in) from the beam end. The researchers decided to verify this result in the fourth test.

The second test beam was turned around and placed in the load frame for the fourth test. The fourth load test was set up with load applied at a distance of 1.75 m from the beam end to verify the simultaneous bond and flexural failure in the previous test. As observed in Figure 19, the maximum sustained load was 2180 kN, resulting in a moment of 2631 kN-m, 23 percent over the predicted nominal moment capacity of 2132 kN-m (Table 11). As observed in Figure 22, no appreciable strand slippage occurred until approximately 1735 kN, when strands 1, 2, and 3 slipped. Prior to this, a shear crack and flexural crack were observed at 1246 kN. The moment corresponding to this load was 1503 kN-m, which is 7 percent over the predicted, cracking moment value of 1401 kN-m. Strands 1 and 3 continued to slip as the load was increased to its peak value, reaching maximum values of 1.17 and 2.77 mm, respectively. Strand 2 reached a maximum value of 0.57 mm at the peak load. Strands 4, 5, and 6 slipped at approximately 1800 kN, reaching maximum values of 0.23, 0.25, and 0.19 mm at the peak load. Cracking extended into the deck and, again, the test was terminated at this point because of safety concerns.

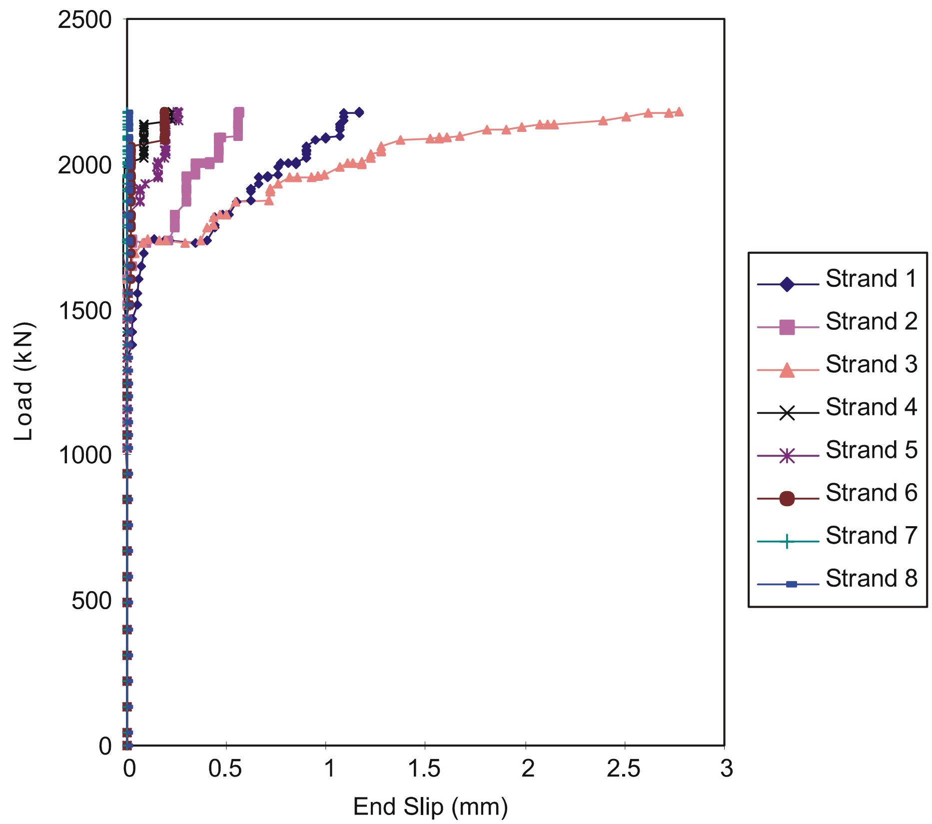

Figure 22. End Slip Versus Load for Test Beam 2, End A

Pullout Test

A pullout test was conducted on each of the eight untensioned strands in the concrete block with concrete from the batch used for the second beam. The results indicated excellent bond strength (Table 12) because all the failures were due to tensile rupture of the strands.

Table 12. Pullout Test Results

Test No.	Maximum Load (kN)
1	253
2	270
3	238
4	244
5	262
6	267
7	203
8	207

Phosphate Test

The residual phosphate was 4638 mg/m^2 (431 mg/ft^2), with a standard deviation of 1345 mg/m^2 (125 mg/ft^2) based on 11 samples. This amount of residual phosphate was determined not to be detrimental, judging from the high bond strengths obtained in the pullout tests.

BRIDGE DESIGN, FABRICATION, AND ASSESSMENT

Overview of Bridge Design and Construction

Based on the success of the test program, VDOT built the Richlands Bridge, which replaced a structure built in 1932 carrying traffic on Virginia Avenue over the Clinch River. The bridge is a two-span structure continuous for live load. Each span is 22.6 m (74 ft) in length and 13.0 m (42.7 ft) in width carrying two lanes of traffic and with sidewalks on either side (Figures 23 and 24). The original design called for seven AASHTO Type III beams per span, using conventional concrete, and with conventional prestressing strands. Because of the test program, VDOT was able to change the design and produce a more economical structure. This new bridge design used HPC mix, five beams, and the larger-diameter prestressing strands. Each girder contained 30 prestressing strands with 16 straight and 14 draped strands. The strands and steel were the same as used in the test program. As shown in Figure 24, the cross section consists of five AASHTO Type III girders, with a deck 216 mm (8.5 in) thick. The concrete in the girders and deck were designed for a minimum concrete compressive strength of 69 MPa (10,000 psi) and 41 MPa (6,000 psi), respectively.

Concurrent with the casting of the beams in July 1997, the pier and abutments were formed and cast. In August, the beams were transported to the site and placed on the

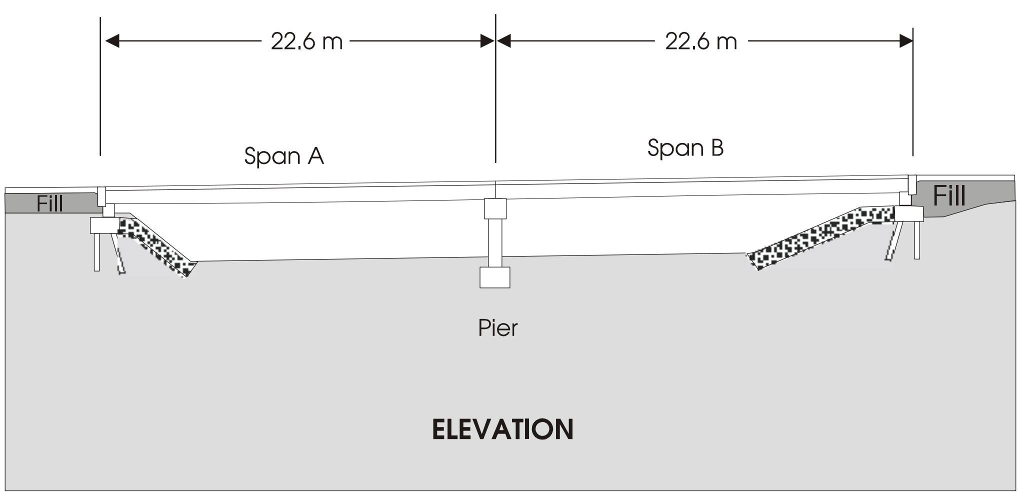

Figure 23. Elevation View of Bridge

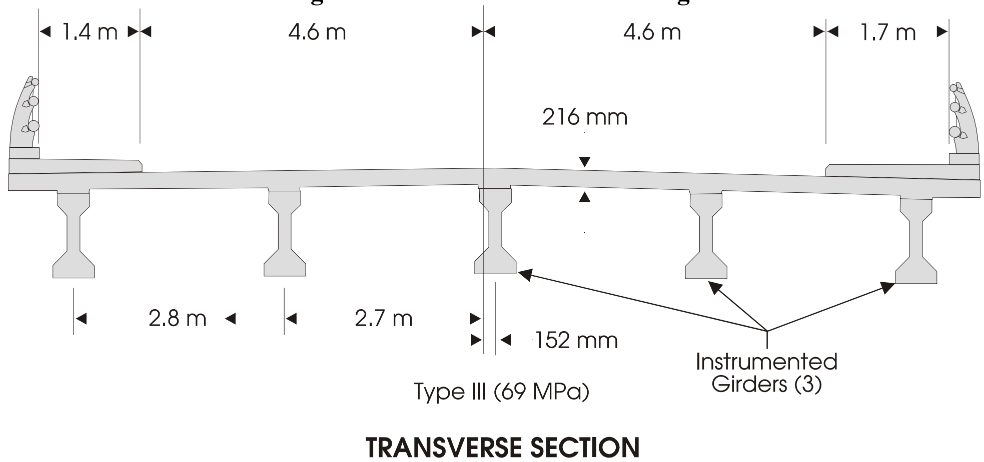

Figure 24. Transverse Section of New Bridge Showing Location of Instrumented Beams

substructure. The deck was formed using galvanized steel stay-in-place forms, and the reinforcing steel placed in September. Deck concrete was placed on October 15, and the bridge was let to traffic on December 18, 1997.

The reduced number of beams from seven to five resulted in initial cost savings. The unit cost (total cost of bridge per deck area) of $657/m^2 ($61/ft^2) was lower than an average cost of $743/m^2 ($69/ft^2) for similar bridges let at that time by the VDOT.

Methodology

Beams

Materials

The concrete mixes were prepared at the same plant where the initial test beams were made using similar material. Type I cement was used, and silica fume conforming to the requirements of ASTM C 1240 was added in slurry form. The coarse and fine aggregates were crushed limestone from the same source used in the test program. Number 7 coarse aggregate with a nominal maximum size of 13 mm (0.5 in) was used. Commercially available air-entraining admixture; water-reducing and retarding admixture conforming to the requirements of ASTM C 494, Type D; and high-range water-reducing admixture, naphthalene condensates, conforming to the requirements of ASTM C 494, Type F, were used. The beams were steam cured, then air cured.

The mixture proportions for the beams are given in Table 13. At the freshly mixed state, air content (ASTM C231, pressure method), slump (ASTM C143), and air and concrete temperatures were measured. Specimens were prepared for testing of hardened concrete. These specimens were steam cured within the steam enclosure in the recesses of the beam forms. The specimens were then air cured in the same manner as the beams.

Table 13. Mixture Proportions for Bridge (kg/m^3)

Ingredient	Beam	Deck
Portland cement	446	332
Silica fume	45	----
Fly ash	----	83
Coarse aggregate size	No. 7	No. 57
Coarse aggregate amount	992	1023
Fine aggregate	801	596
Water	137	187
w/cm	0.28	0.45
High-range water-reducing admixture (mL/kg)	16	----

Cylinders measuring 100 by 200 mm (4 x 8 in) were prepared and tested for compressive strength at 1, 28, and 56 days and 1 year in accordance with AASHTO T22 except that neoprene pads in steel end caps were used for capping. The elastic modulus was obtained from compressive strength cylinders at 1 year. Cylinders were also tested for splitting tensile strength at 28 days. Beams measuring 75 x 75 x 285 mm (3 x 3 x 11 1/4 in) with end studs were used for length change measurements (ASTM C 157).

To determine the resistance of the concrete to damage from cycles of freezing and thawing, prisms measuring 75 x 100 x 400 mm (3 x 4 x 16 in) were tested at 2 months in accordance with ASTM C 666, Procedure A, except that 2 percent NaCl was used in the test water. The criteria for satisfactory resistance for the average of three specimens at 300 cycles include:

- a weight loss of 7 percent or less
- a durability factor of 60 or more (ASTM C 666)
- a surface rating of 3 (rating per ASTM C 672).

Cylinders measuring 100 by 200 mm (4 x 8 in) were used to determine chloride permeability (AASHTO T 277 or ASTM 1202) at 28 days. The top 50 mm (2 in) of cylinders was tested. Resistance to chloride penetration was also determined using the ponding test (AASHTO T 259). Slabs measuring 0.3 m x 0.3 m (1 ft x 1 ft) were cast and at 28 days ponded with 3 percent NaCl solution and tested after 1 year of ponding. Chloride samples were obtained at four average depths of 13 mm (0.5 in), 25 mm (1 in), 38 mm (1.5 in), and 51 mm (2 in). The top 6 mm (1/4 in) was discarded. Each sample was obtained from a layer 13-mm (0.5-in) thick.

Fabrication and Instrumentation

The 10 beams were fabricated in 4 days as follows: three on July 7, two on July 9, three on July 15, and two on July 18, 1997. Prestressing strands were placed in the bed and stressed. Shear reinforcement was placed next. Side forms were placed. The next morning, concrete was delivered using the plant's ready-mix trucks and placed in the forms. The concrete was placed in layers and consolidated by internal and external vibration. The top surfaces of the beams were screeded by hand and covered with wet burlap to prevent the loss of moisture from the concrete. The insulating blankets were then placed over the beams, and that afternoon the steam was turned on. Test cylinders were placed in the recesses of the formwork, under the blanket. The next morning, the cylinders placed in the steam bed were tested for strength to ensure the release strength was reached. The forms were removed and strands detensioned by flame-cutting. After detensioning, the beams were removed from the bed and placed in a storage yard.

Of the 10 beams cast, 3 were instrumented with thermocouples to monitor concrete temperatures. The instrumented beams were the center beam, one outer beam, and one beam in between, as shown in Figure 24. Temperature readings were measured to determine the temperature rise attributable to heat of hydration, temperature for match-cured cylinders, and temperature gradients within the components. Proper temperature management is essential to achieve high early and ultimate strengths and to ensure an overall quality product.[9] The thermocouples were placed at the following locations, as shown in Figure 25:

- approximate location of the center of gravity of top flange of beam
- mid-depth of the beam
- approximate location of the center of gravity of the bottom flange of the beam
- near the bottom surface of the beam.

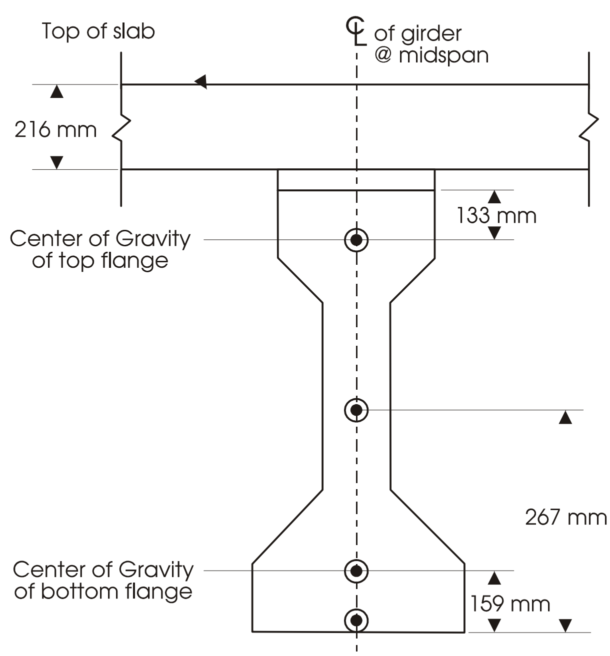

Figure 25. Locations of Thermocouples

The thermocouples were placed in each of the three beams at midspan and 300 mm (1 ft) from one end.

Strain gages were placed at the midspan of the beams at the center of gravity of the prestressing force to measure the concrete strains at detensioning and continuously thereafter. Long-term strain measurements were to be used for determining prestress losses attributable to elastic shortening, creep, and shrinkage. Three gages, for redundancy of instrumentation, were spaced longitudinally on a piece of reinforcing steel and placed at the center of gravity of the prestressing force at midspan, as shown in Figure 26.

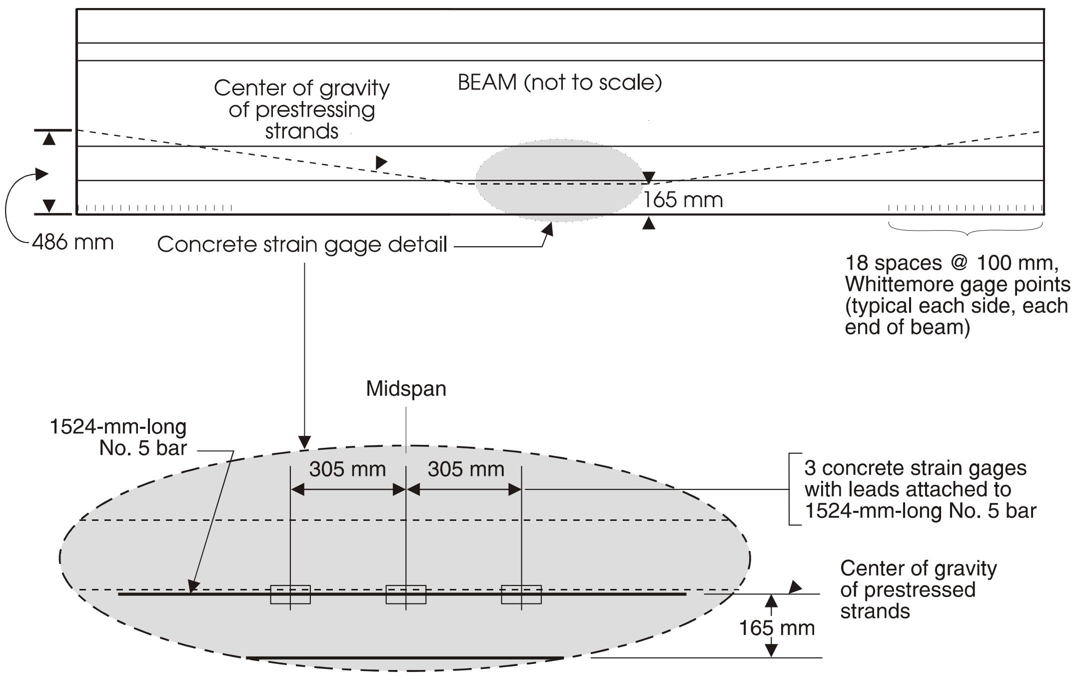

Figure 26. Location of Strain Gages

Deflection of the beams was determined with a precise level and leveling rod. Measurements were taken at the ends and quarter points of each of the three instrumented beams prior to and just after detensioning. Measurements were taken after placement at the site and again after placement of the deck. Measurements were to be taken periodically thereafter, until the completion of the project.

For end slip determination, 12 strands on both the load and dead ends of each of the 3 beams were instrumented with end slip measurement gages identical with the ones used in the test program. Eight strands across the bottom row, two on the next row up, and two of the draped strands were measured. Measurements were to be taken prior to and just after detensioning and again at 1 day.

Transfer length was determined by measurement of the concrete strains on the surface of the lower flange of the beams at the depth of the prestressing strands. Both ends of the 3 beams, on both sides (four locations per beam), were instrumented with brass inserts (Whittemore strain gage points) spaced at 100-mm (4-in) intervals for a total distance of 1800 mm (72 in). Measurements with a Whittemore mechanical strain gage were taken just before and after detensioning and at 1 day and 14 days after detensioning. The difference between the initial readings (just prior to detensioning) and the readings taken after detensioning, normalized with respect to the initial readings, yields a strain profile along the face of the beam at the location of the brass studs. Ideally, this strain profile reaches a plateau, indicating that the transfer of prestressing force from the strands to the concrete has occurred.

Deck Materials and Fabrication

The mix proportions for the deck concrete are given in Table 13. The concrete was delivered in a truck mixer and was deposited on the deck using a pump. The concrete was then consolidated and screeded using a vibratory screed. At the edges, where the roller screed could not reach, concrete was consolidated with immersion-type vibrators. Hand floats were used to level the surface at the edges. After screeding, a curing compound was sprayed on the surface. When concrete reached final set, wet burlap was placed and covered with white plastic sheeting and moist cured for 7 days.

The concrete from the truck mixer was tested for air content using the pressure method. Then, it was pumped and the air content determined again. The pumped concrete was used to make test specimens. Two batches of concrete were tested for compressive strength, elastic modulus, splitting tensile strength, resistance to cycles of freezing and thawing, and permeability. The specimens were kept near the bridge overnight and were transported to the laboratory at the Virginia Transportation Research Council the next day. The specimens for strength measurements were moist cured until tested. Specimens tested for freezing and thawing resistance were moist cured for 2 weeks and air dried until tested at 6 months. Specimens for chloride content were moist cured for 2 weeks and air dried for 4 weeks prior to ponding for a year.

Results and Discussion

Material Testing of Beams

The characteristics of freshly mixed concrete for the three bridge beams cast on July 15 are summarized in Table 14. Seven truckloads of concrete were necessary to cast these three beams. Two representative batches were sampled, one from truckload 2 (batch 1) and the other from truckload 6 (batch 2). The air content was acceptable even though that for batch 1 was at the lower limit. The air content requirement of the prestressed concrete is 4.5 ± 1.5 percent with the use of high-range water-reducing admixture. The workability of the concrete mixes was satisfactory. The concrete temperatures were above the air temperature. The temperature

Table 14. Characteristics of Freshly Mixed Concrete for Bridge Beams

Items	Batch 1	Batch 2
Air (%)	4.0	5.1
Slump (mm)	125	160
Concrete temperature (°C)	29	32
Air temperature (°C)	23	26

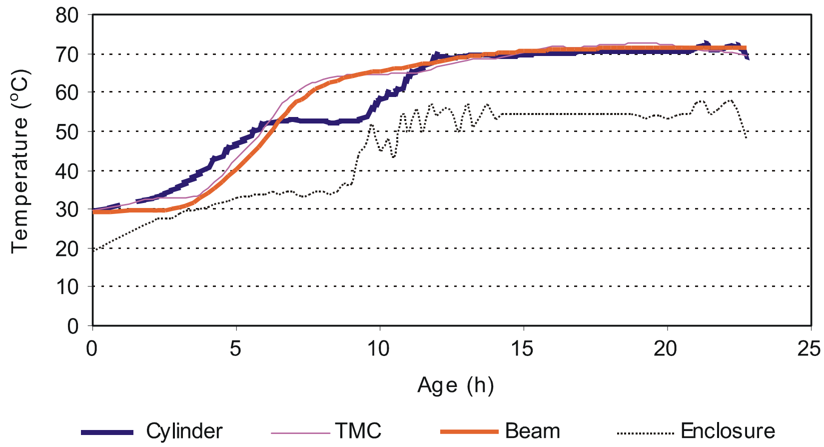

Figure 27. Temperature Development in Bridge Beam

development in the beam, the cylinders cured in the enclosure in the recesses of the beam molds, the enclosure, and the TMC specimens after placement until the end of steam curing are shown in Figure 27 for beam 1. The steam was applied for about 14.5 hours. The next morning, the strength was determined. The forms were removed, and the strands detensioned. The beams were then removed and placed in the storage yard. The beds were then cleaned and oiled and the strands pulled for the next casting. The results of testing hardened concrete are given in Table 15. The compressive strengths of the TMC and regular concrete cylinders after steam curing were close and exceeded 65 MPa (9,500 psi), approaching the specified 28-day compressive strength of 69 MPa (10,000 psi). The temperature development in the regular steam-cured samples closely followed that of the TMC specimens except at the beginning of the steam curing. Therefore, there were no large differences in strengths between the regular steam-cured and TMC specimens. The 28-day compressive strength values were in excess of the 69 MPa (10,000 psi) specified. The elastic modulus values were calculated using the empirical equation. The unit weight was assumed to be 2403 kg/m^3 (150 lb/ft^3). The test values were close to the empirical elastic modulus values as shown in Table 15. The splitting tensile strength at 28 days was 7.8 and 7.6 percent for batch 1 and batch 2, respectively, of the compressive strength. The permeability was very low, 122 and 127 for batch 1 and batch 2, respectively. The chloride content is given in Table 16. At the top depth of 13 mm (0.5 in), the values were about equal or higher than the corrosion threshold value of 0.77 kg/m^3 (1.3 lb/yd^3), but at lower depths, they were much less than the threshold value.

Table 15. Properties of Hardened Concrete for Bridge Beams

Test	Age	Number	Batch 1	Batch 2
Temperature-matched cure	1 d	2	65.3[a]	67.0
Compressive strength (MPa)	1 d	2	67.7	66.1
	28 d	3	72.5	75.2
	56 d	3	73.9[b]	72.6
	1 yr	3	75.3	76.9
Elastic modulus (GPa)	1 yr	3	43.4	40.7
Empirical E (GPa)	---	---	43.7	44.1
Splitting tensile (MPa)	28 d	3	5.66	5.71
Permeability (coulombs)	28 d	2	122	127

[a] One of the specimens had a corner break and had a strength of 57.6 MPa.
[b]Average of two specimens.

Table 16. Chloride Content for Bridge Concrete (kg/m^3)

Batch	Element	13 mm	25 mm	38 mm	51mm
1	Beam	1.28	0.26	0.25	0.22
2	Beam	2.15	0.31	0.29	0.26
1	Deck	6.40	0.65	0.56	0.17
2	Deck	6.98	0.71	0.21	0.18

The freezing and thawing data are given in Table 17 and indicate that batch 1 had a low durability factor and batch 2 had a satisfactory value. Batch 1 had 4.0 percent air, which is the lowest acceptable limit. The length change data are shown in Figure 28. At 1 year, the length change for 75 x 75 x 285 mm shrinkage test specimens was 333 µm/m and 450 µm/m for specimens cast with batch 1 and batch 2, respectively.

Table 17. Freezing and Thawing Data at 300 cycles for Bridge

Batch	Element	Weight Loss (%)	Durability Factor	Surface Rating
1	Beam	1.31	26	0.69
2	Beam	0.20	91	0.35
1	Deck	0.74	109	0.75
2	Deck	0.88	107	0.69

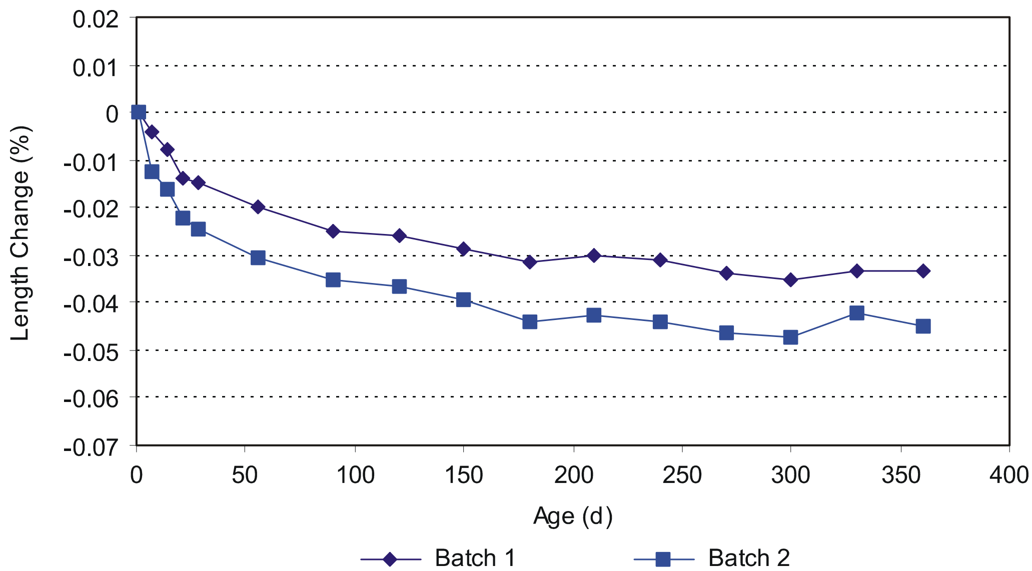

Figure 28. Length Change Data

Performance of Beams

Beam Strains and Temperatures

Plots of temperature versus time are shown in Figures 29 through 31 for the instrumented girders. The temperatures were taken from the middle of the beam at the designated depths. Temperatures are shown from time of concrete placement until just after detensioning. The temperatures were well within acceptable limits. A plot of strain versus time for the same period

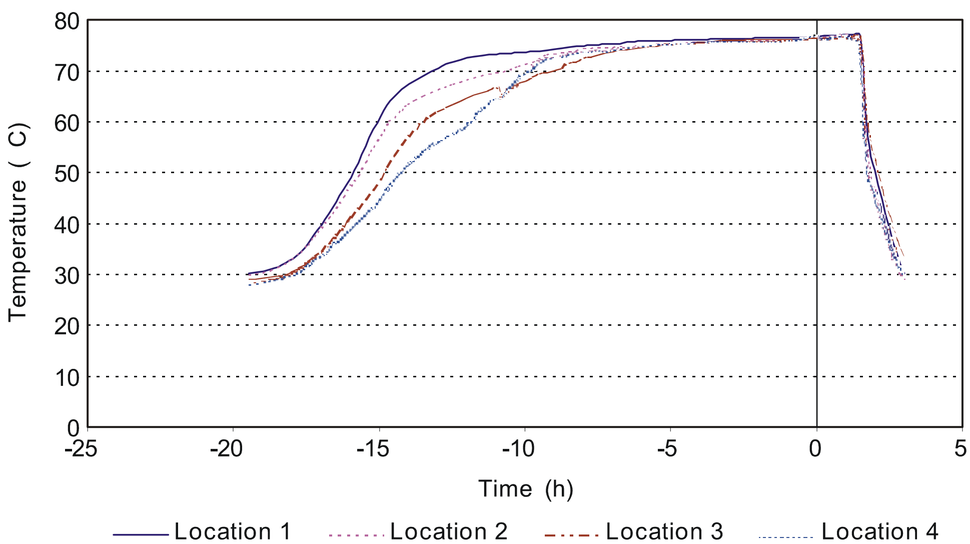

Figure 29. Temperature Versus Time for Beam 1

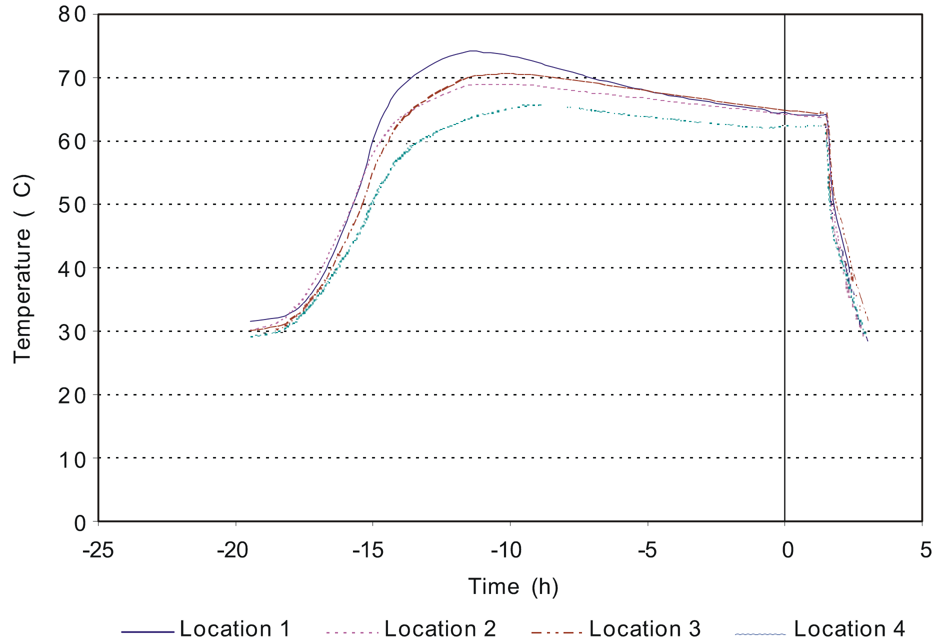

Figure 30. Temperature Versus Time for Beam 2

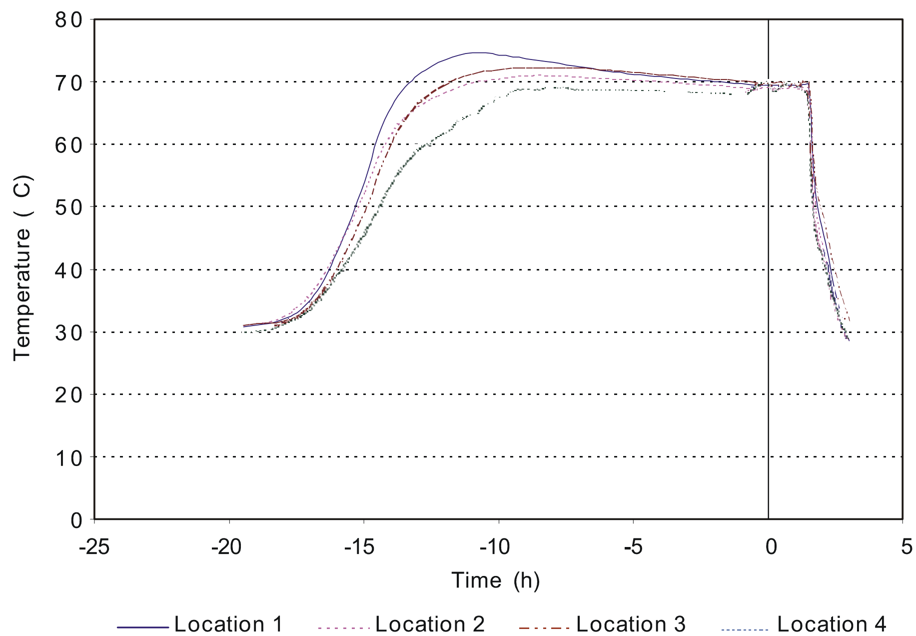

Figure 31. Temperature Versus Time for Beam 3

is shown in Figures 32 through 34 for each instrumented beam. It is evident when detensioning occurred by the instantaneous jump in compressive strain. Strain and temperature monitoring was stopped after detensioning while the beams were moved to the holding yard at the fabrication plant. Monitoring was discontinued again while the beams were in transit to the

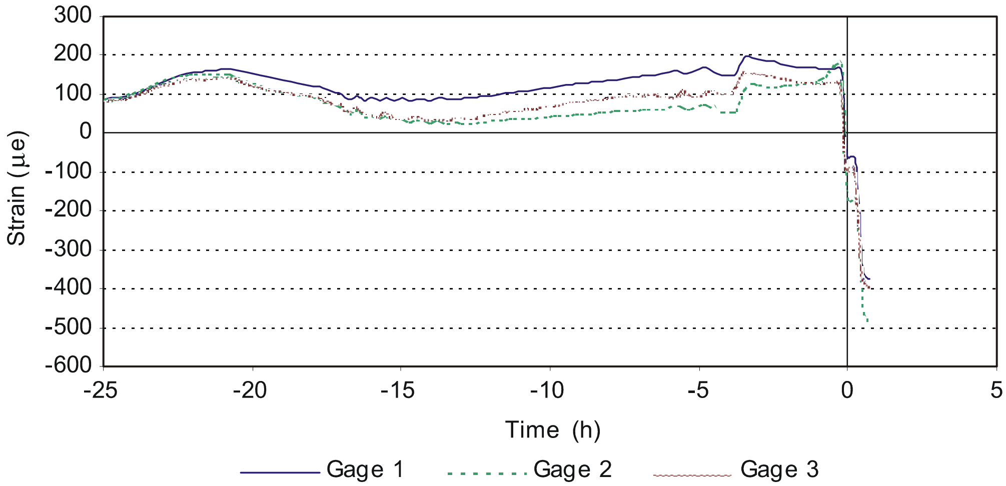

Figure 32. Strain Versus Time for Beam 1

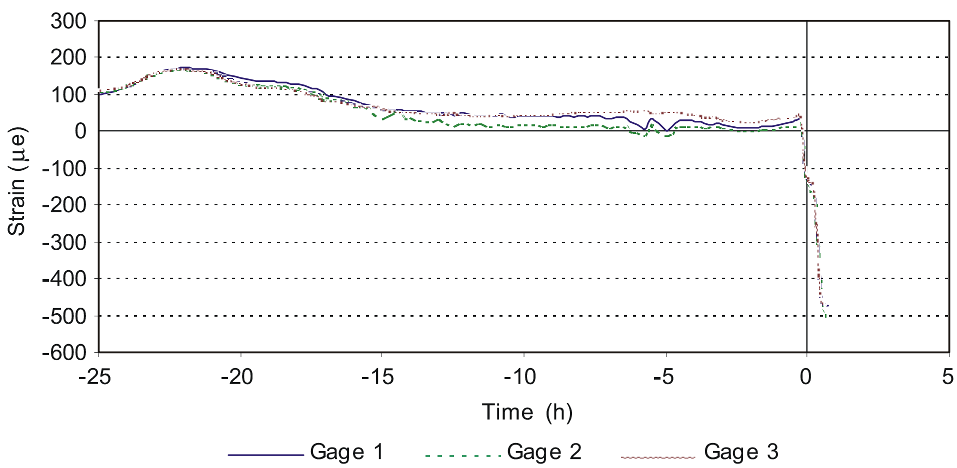

Figure 33. Strain Versus Time for Beam 2

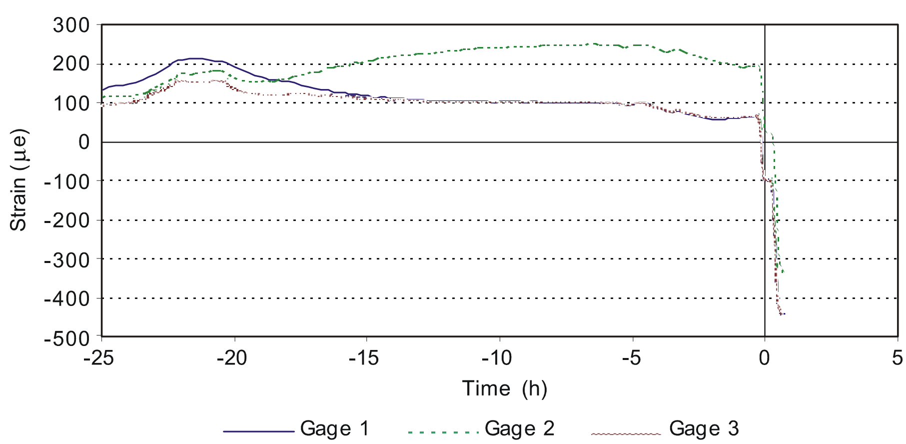

Figure 34. Strain Versus Time for Beam 3

bridge site. The data acquisition system was again activated once erection of the beams was complete. The strains and temperatures have been continuously monitored since that time.

Camber

Camber readings were taken immediately before and just after detensioning and again at 1 and 14 days. Camber readings were taken again just after the girders were erected (day 77). The camber at midspan for these periods is shown for all three girders in Figure 35. The day 77 midspan camber for the three instrumented beams were approximately 40, 43, and 46 mm, respectively. The maximum predicted camber was 30 mm. Some of this difference can be attributed to the higher prestress force that was applied at the fabrication yard. Camber readings will be taken twice a year for the duration of the monitoring period and will be used in determining prestress losses. The results will be reported at the completion of the monitoring period.

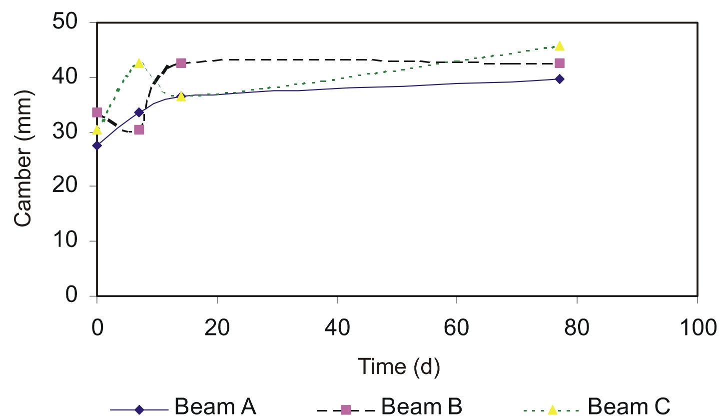

Figure 35. Midspan Camber Versus Time

Transfer Length and End Slip

A typical plot of concrete surface strain is shown in Figure 36. The measured transfer lengths varied from 310 mm (12.2 in) to 390 mm (15.4 in), as shown in Table 18. The predicted value, based on AASHTO Equation 9.2, is 840 mm (33.0 in).

Table 18. Transfer Lengths of the Three Instrumented Bridge Beams (mm)

Beam	End	Transfer Length
A	Live	360
A	Dead	390
B	Live	315
B	Dead	310
C	Live	375
C	Dead	310

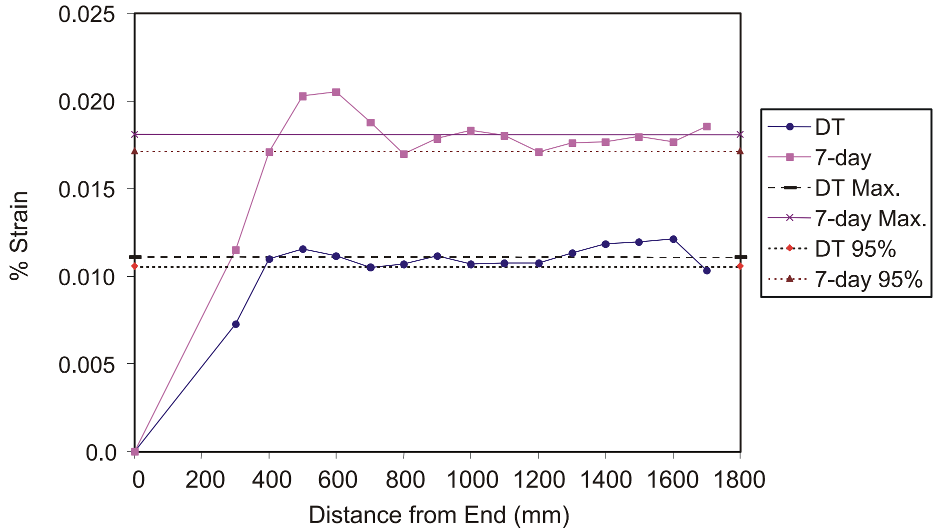

Figure 36. Concrete Surface Strain for Transfer length Determination of Beam 1

Detensioning was achieved by flame cutting. This method releases a tremendous amount of energy in a very short time. This shock resulted in damage to the end slip devices such that there was no confidence in the accuracy of the measurements. Hence, no end slip data are reported.

Material Testing of Deck Concrete

The characteristics of freshly mixed concrete for the deck are provided in Table 19. Workable concrete mixes were obtained. The air content for the first batch was 5.5 percent before and 3.5 percent after pumping. In the second batch, the air content was 6.0 percent before and 4.5 percent after pumping. The air content requirement of the bridge deck concrete was 6.5 ± 1.5 percent. Thus, the mixes met the requirement before pumping but not after pumping.

The results of freezing and thawing tests are given in Table 17. The specimens were made from concrete mixes obtained after pumping. At 300 cycles, the first batch had a loss in mass of 0.7 percent and a durability factor of 109, and the second batch had a loss in mass of 0.9 percent and a durability factor of 107. The acceptance criteria were met.

Table 19. Characteristics of Freshly Mixed Concrete for Deck

Characteristic	Batch 1	Batch 2
Before pumping		
Air (%)	5.5	6.0
Slump (mm)	95	----
After pumping		
Air (%)	3.5	4.5
Concrete temperature (°C)	22	26
Air temperature (°C)	14	16

The concrete samples were also subjected to petrographic analysis of the air-void system. For the first batch, the air content was 4.5 percent and the spacing factor 0.192 mm. For the second batch, these parameters were 3.4 percent and 0.218 mm, respectively. The spacing factors were about 0.200 mm (0.008 in), which is the generally accepted limit for satisfactory performance.[9] These mixes had acceptable performance even at a low air content since the loss of air was mainly associated with a loss in the larger bubbles that have a marginal affect on freezing and thawing resistance. However, the mixes should meet the specification requirements at the point of discharge after pumping. If air content does not meet the requirements after pumping, concrete can still be acceptable based on further testing (freezing and thawing, by ASTM C 666, Procedure A, or petrographic analysis, ASTM C 457) of the pumped concrete. The loss of air content is not detrimental to the concrete if resistance is satisfactory or if the air-void system is still satisfactory.

The compressive strength at 28 days exceeded 41.7 MPa (6,000 psi specified) as shown in Table 20. The splitting tensile strengths at 28 days were 9.3 and 10.1 percent of the compressive strength. The elastic modulus was determined from cylinders and also calculated using the empirical equation. The unit weight was assumed to be 2323 kg/m^3 (145 lb/ft^3). The values were close to the empirical elastic modulus values, as shown in Table 20. The permeability samples were moist cured for 1 week at 23 °C (73 °F) and for an additional 3 weeks at 38 °C (100 °F) and tested at 28 days. The values were less than the specified 2500 coulombs, as shown in Table 20.

Table 20. Testing of Hardened Concrete for Deck

Test	Age	No.	Batch 1	Batch 2
Compressive	7 d	3	28.7	30.1
strength (MPa)	28 d	3	42.4	44.0
	56 d	3	45.7	46.8
	1 yr	3	56.0	55.1
Elastic modulus (GPa)	1 yr	3	35.3	37.0
Empirical E (GPa)	---	---	35.8	35.5
Splitting tensile (MPa)	28 d	3	3.94	4.44
Permeability (coulombs)	28 d	2	1261	1375

Condition Assessment of Bridge

After exposure to two winters, the beams and the deck had no distress upon visual examination except that cracks were evident in the middle 2.5-m (8-ft) section of the deck. This section was cast last and is directly over the closure diaphragm at the pier. The closure diaphragm is a typical detail to provide moment continuity for live load. No other cracks were observed in the deck. The bridge will be monitored for another 2 years and a more complete assessment of its status will be included in a separate report.

CONCLUSIONS

- Air-entrained HPC with a high early release strength and a 28-day strength exceeding 69 MPa (10,000 psi) can be successfully produced with locally available material in Virginia.

- The development of high early strength adversely affects ultimate strength. However, with proper temperature management, both high early and high ultimate strength can be achieved.

- Temperature and strength development in steam-cured beams may be different than in cylinders kept in a steam enclosure. Temperature-matched cure specimens are more representative of the temperature and strength development of beams.

- T he large strands, 15 mm (0.6 in) in diameter, spaced 51 mm (2 in) apart can be used successfully for pretensioned applications.

- In addition to structural benefits, economic benefits can be achieved with the use of concrete with a higher compressive strength and low permeability and the large-diameter strands.

- Concrete mixes with very low permeability can be produced with the use of silica fume in the mixes.

- Since the transfer length for the bridge beams was less than half of that predicted by AASHTO Equation 9-2, modifications to the equation are warranted.

- A residual phosphate level of 4638 mg/m^2 (431 mg/ft^2) does not adversely affect the bond strength for prestressing strands.

RECOMMENDATIONS

1. Use high-strength concrete with 15-mm (0.6-in) strands in beams if economically feasible.

2. Use low-permeability concrete in bridges for longevity. Low permeability can be achieved with the use of silica fume, fly ash, or slag alone or in combination.

3. Use conventional concrete with a strength of 28 MPa (4,000 psi) in bridge decks. Economic benefits were not shown by the higher-strength (41 MPa, 6,000 psi) design.

4. Pay close attention to consolidation for quality concrete and for a strong bond between the strands and the concrete.

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