Development Length of Prestressing Strand in Bridge Members

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FHWA is undertaking a research study on the development length of prestressing strand. The objective is to investigate the validity of AASHTO Equation 9-32 for predicting both the transfer length and flexural bond length components of development length for fully bonded, straight, uncoated, and epoxy-coated prestressing strand. Toward this end, the effects of strand diameter [9.5, 12.7, and 15.2 mm (3/8, 0.5, and 0.6 in.)], concrete strength, strand coating (uncoated or epoxy coated), and strand spacing on development length will be evaluated. The phase of the research study involves full-size prestressed concrete bridge members. A total of 32 AASHTO Type II prestressed concrete bridge girders and 32 prestressed concrete deck panels were fabricated at a precast concrete plant in Winchester, Virginia. All of the members will undergo transfer and development length experimentation. The fabrication, instrumentation, and experimentation procedures, as well as partial results, are described. Experimentation is scheduled to be finished in the spring of 1995.

The bond of prestressing strands in pretensioned concrete members has been studied by many researchers over the last few years. Much of this research was initiated as a result of FHWA's October 1988 memorandum. The memorandum increased the required development length for fully bonded uncoated strand by 1.6 times the development length specified by AASHTO in Equation 9-32 (*1*). For debonded strand, this factor was specified as 2.0. The memorandum also disallowed the use of a strand 15.2 mm (0.6 in.) in diameter in a pretensioned application and restricted the minimum strand spacing (center-to-center of strand) to four times the nominal strand diameter. The FHWA memorandum indicated that its restrictions were adopted only as an interim measure, until research results indicate otherwise and AASHTO adopts the results.

The advent of epoxy-coated prestressing strand has also prompted research on bond of prestressing strand. Epoxy-coated strand was developed by a manufacturer to provide corrosion protection for prestressing strand. This epoxy coating has a chemical formulation that is different from that of the epoxy coating found on reinforcing bars, and it is also much thicker. The thickness of the epoxy coating on prestressing strands is 0.64 to 1.14 mm (0.025 to 0.45 in.) thick, whereas the epoxy coating on reinforcing bars is 0.18 to 0.30 mm (0.007 to 0.012 in.) thick (ASTM A 882-92 and ASTM A 775-93a). For pretensioned applications, particles of grit are embedded in the surface of the epoxy coating to enhance the bond of the coated strand to the concrete. Questions arose as to the applicability of the AASHTO equation for development length to pretensioned members containing epoxy-coated strand.

FHWA initiated its research effort in the spring of 1990 in an effort to answer questions concerning bond for both uncoated and epoxy-coated prestressing strand. The research effort is in two phases: Phase 1 involves rectangular prestressed concrete specimens, whereas Phase 2 uses full-size prestressed concrete deck panels and girders.

Rectangular prestressed concrete specimens for Phase 1 were fabricated and evaluated at the FHWA Structures Laboratory in the Turner-Fairbank Highway Research Center in McLean, Virginia. A total of 50 rectangular specimens were fabricated: 24 had concentric uncoated or epoxy-coated strands, 24 had eccentric uncoated or epoxy-coated strands, and 2 were used to monitor shrinkage of the concrete. The specimens ranged in size from 102 × 102 × 3658 mm (4 in. × 4 in. × 12 ft) to 356 × 356 × 8534 mm (14 in. × 14 in. × 28 ft). Three strand sizes were used in the following diameters, namely 9.5, 12.7, and 15.2 mm (3/8, 0.5, and 0.6 in.), and the specimens contained either one strand or four strands. The details of this phase of the study are provided elsewhere (*2–4*). The rectangular specimens containing concentric strands underwent transfer length experimentation only. Both transfer and development length experimentation were performed for the rectangular specimens containing eccentric strands. The broad conclusions from the first phase of the study were as follows:

- The AASHTO transfer and development length expressions were unconservative for specimens with multiple uncoated strands of all diameters;
- The AASHTO transfer length expression was conservative for specimens containing epoxy-coated strands, except for the specimens containing four strands 12.7 mm (0.5 in.) in diameter; and
- The AASHTO development length expression was conservative for all specimens containing epoxy-coated strands.

Full-size members for Phase 2 of the study were fabricated at Shockey Bros., Inc., precast concrete plant in Winchester, Virginia. A total of 32 AASHTO Type II prestressed concrete I-girders and 32 prestressed concrete subdeck panels (hereafter called deck panels) were fabricated from February through May 1994. Development length experimentation began in May 1994 and is still ongoing at the time this paper was written.

This paper describes the fabrication, instrumentation, and methods of experimentation for Phase 2 of the study. Because the experimentation is ongoing as of the writing of this paper, limited results will be presented here.

Background

In a pretensioned member, the prestressing force in a prestressing strand is transferred from the strand to the concrete by bond. A certain distance is needed to achieve bonding at ultimate load between the steel strand and the concrete to attain the capacity of the member. This distance, measured from the end of the member, is called the development length.

The development length consists of two components: transfer length and flexural bond length. The transfer length is the distance from the end of the member needed to fully transfer the effective prestress (f_{se}) by bond from the steel strand to the concrete. The flexural bond length is the additional embedment length needed beyond the transfer length to achieve bonding between the strand and the concrete to attain the stress in the strand at the ultimate load of the member (f^*_{su}) (*5–7*).

In Article 9.27 of the AASHTO specifications (*1*), the development length of a member is given by Equation 9-32 as

$$(f^*_{su} - \frac{2}{3} f_{se})D \qquad (1)$$

where D equals the nominal diameter of the strand in inches.

This expression can be rewritten in terms of its constituent parts (*6*) as

$$L_d = \frac{f_{se}D}{3} + (f^*_{su} - f_{se})D \qquad (2)$$

where

L_d = development length,

$\frac{f_{se}D}{3}$ = transfer length, and

$(f^*_{su} - f_{se})D$ = flexural bond length.

In Article 9.20.2.4 of the AASHTO specifications (*1*), it is stated that the transfer length component of the development length may be assumed to be equal to 50 times the nominal diameter of the strand. The multipliers specified in the FHWA memorandum referred to previously would be applied to equation 1 or 2 mentioned earlier.

Description of Members

Girders

A total of 32 AASHTO Type II prestressed concrete I-girders were fabricated as part of this study. Half of the girders contained epoxy-coated strand, whereas the other half contained uncoated strand. Three different strand patterns were used in the girders, as shown in Figure 1. Strand Pattern A contained eight strands 12.7

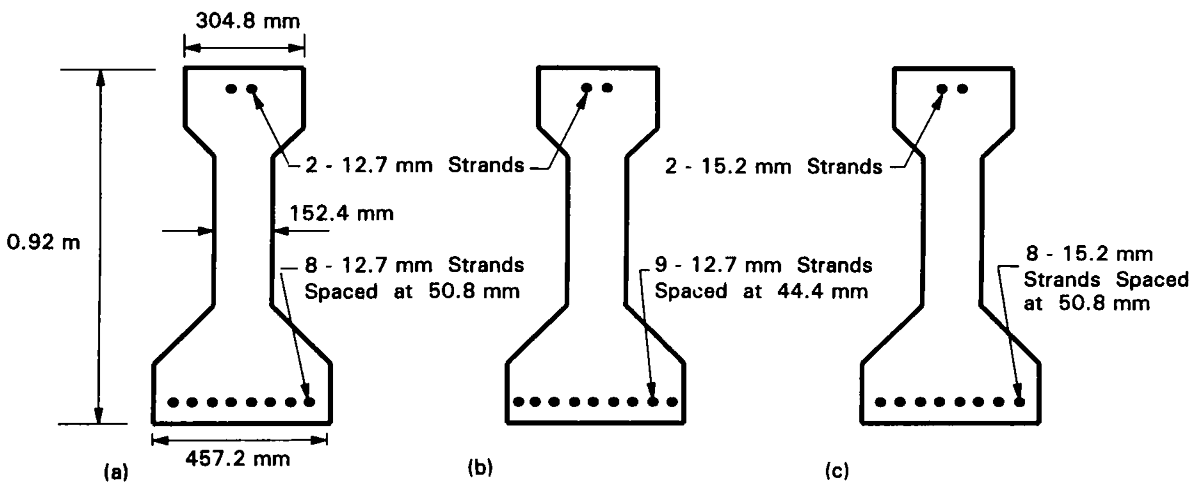

FIGURE 1 AASHTO Type II girder with various strand patterns: (*a*) Strand Pattern A; (*b*) Strand Pattern B; and (*c*) Strand Pattern C. (1 m = 3.28 ft; 1 mm = 0.039 in.)

mm (0.5 in.) in diameter spaced at 50.8 mm (2 in.) in one row in the bottom flange and two strands 12.7 mm (0.5 in.) in diameter in the top flange (Figure 1*a*). Strand Pattern B is shown in Figure 1*b*; it contained nine strands 12.7 mm (0.5 in.) in diameter spaced at 44.4 mm (1.75 in.) in one row in the bottom flange and two strands 12.7 mm (0.5 in.) in diameter in the top flange. Strand Pattern C contained eight strands 15.2 mm (0.6 in.) in diameter spaced at 50.8 mm (2 in.) in one row in the bottom flange and two strands 15.2 mm (0.6 in.) in diameter in the top flange (Figure 1*c*). All of the strands were fully stressed.

All of the girders were 9.46 m (31 ft) long. A total of 24 of the girders had a 28-day design concrete compressive strength of 34.4 MPa (5 ksi), with a specified maximum 28-day compressive strength of 44.8 MPa (6.5 ksi). The remaining eight had a 28-day design compressive strength of 68.9 MPa (10 ksi), with a specified maximum 28-day compressive strength of 89.6 MPa (13 ksi). These limits, or windows, of concrete strength were specified to differentiate between the two strengths of concrete. A matrix of the makeup of the girders is given in Table 1.

As can be seen from Table 1, 12 of the girders will be made composite with a cast-in-place slab. This will be done in an attempt to increase the strain in the bottom strands at failure. In a report for FHWA (*8*, pp. 56–57) Buckner stated the following:

> . . . specimens proportioned to have strains at failure near yield (0.010) have usually achieved their predicted moment strengths at strand embedments calculated by the current ACI/AASHTO [development length] expression. Specimens proportioned to achieve strains at failure of about 0.035 have typically failed to reach their expected moment capacities due to premature bond failure.

In an attempt to verify this observation, the FHWA research program has included girders with and without composite slabs. This will provide a range of strand strain values at failure of the girders.

All of the girders contained single-leg stirrups, spaced at 76.2 mm (3 in.), which alternated sides of the cross section at each spacing. Confinement reinforcement was placed in the top and bottom flanges for the first 0.92 m (3 ft) on each end of a girder.

Deck Panels

A total of 32 prestressed concrete deck panels were fabricated as part of this study. Half of the deck panels contained epoxy-coated strand, whereas the other half contained uncoated strand. The deck panels were designed and fabricated in four different sizes, as shown in Figure 2.

Two different deck panel thicknesses were chosen, namely 76.2 mm (3 in.) and 88.9 mm (3.5 in.). This was done to determine whether there is any difference in bond behavior between deck panels of these thicknesses. Currently, FHWA specifies an 88.9-mm (3.5-in.) subdeck panel thickness for bridges built with federal-aid monies, whereas the Precast/Prestressed Concrete Institute (PCI) recommends a minimum deck panel thickness of 76.2 mm (3 in.) (*9*).

Two different panel lengths were selected. Deck panel types A and B were 2.52 m (8.25 ft) long, whereas deck panel Types C and D were 3.05 m (10 ft) long. This was done to provide varying embedment lengths

TABLE 1 AASHTO Type II Girders

Girder No.	Design f'_c (MPa)	Strand Pattern	Strand Epoxy Coated (E) or Uncoated (U)	Made Composite With Slab?
33, 34	34.4	A	U	No
35, 36	34.4	A	E	No
37, 38	68.9	A	U	No
89, 90	68.9	A	E	No
41, 42	34.4	A	U	Yes
43, 44	34.4	A	E	Yes
57, 58	34.4	B	U	No
59, 60	34.4	B	E	No
61, 62	34.4	B	U	Yes
63, 64	34.4	B	E	Yes
45, 46	34.4	C	U	No
47, 48	34.4	C	E	No
49, 50	68.9	C	U	No
51, 52	68.9	C	E	No
53, 54	34.4	C	U	Yes
55, 56	34.4	C	E	Yes

1 MPa = 0.145 ksi

for the development length experimentation. The 2.52-m (8.25-ft) length represents an overall length less than twice the calculated AASHTO development length; the 3.05-m (10-ft) length represents an overall length greater than twice the calculated AASHTO development length.

All of the deck panels had a 28-day design concrete compressive strength of 34.3 MPa (5 ksi), and they all contained strands 9.5 mm (3/8 in.) in diameter strands. A matrix of the makeup of the deck panels is given in Table 2. As can be seen from Table 2, half of the deck panels will be made composite with a cast-in-place slab. This will be done in an attempt to increase the strain in the strands at failure. All deck panels contained a layer of intermittently spaced reinforcing bars on top of the strands and perpendicular to them. The reinforcing bars functioned as a means of distributing the load and met the minimum reinforcement requirement specified in Article 9.23.2 of the AASHTO specifications (*1*).

MATERIALS

Prestressing Strand

All of the uncoated prestressing steel was seven-wire, Grade 270 [1,860 MPa (270 ksi) guaranteed ultimate tensile strength], low-relaxation strand, conforming to ASTM Standard A 416-90a. The strand was used in the as-received condition, with occasional surface rust visible but no pitting. Tests were run to determine any phosphate residue left on the strand surface. Also, concrete blocks containing untensioned, uncoated, and epoxy-coated strands were cast with samples from the as-

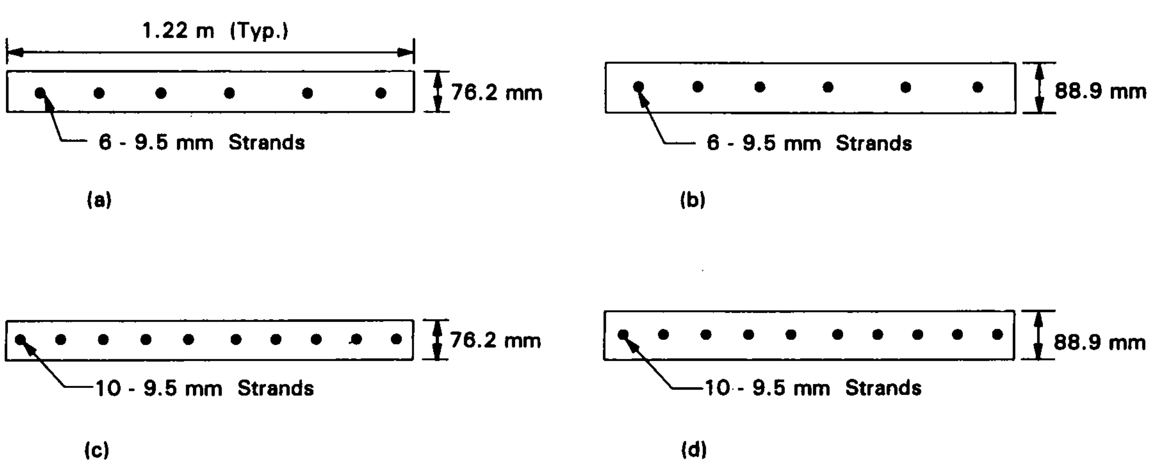

FIGURE 2 Deck panels (*a*) Type A; (*b*) Type B; (*c*) Type C; and (*d*) Type D. (1 m = 3.28 ft; 1 mm = 0.039 in.)

composite decks for 12 of the girders and 16 of the deck panels will be cast there. Experimentation was also performed there.

Transfer Length

The transfer length of each end of each member was determined by plotting the strains calculated from the Whittemore gauge readings. The transfer length can be defined as the distance from the end of the beam to the point on the curve equal to the average value of strain for the plateau portion of the plot. Transfer lengths were determined for average values of strain equal to 100 percent of the plateau value and for average values of strain equal to 95 percent of the plateau value. This was done in an effort to compare transfer length values with other researchers who use varying definitions of transfer length. Transfer lengths were determined for each of the intervals mentioned for the Whittemore gauge measurements.

Development Length

Development length experimentation will be performed for each end of each girder. The estimated development length is first calculated and is called the embedment length. A single point load is applied at a distance from the end of a girder equal to this embedment length. This point load is increased in intervals up to failure. Failure can be one of two types: flexural failure or bond failure. Flexural failure occurs when the concrete crushes in compression or the strands break in tension. Bond failure occurs when the strands lose their bond and slip in toward the member. In I-shaped members, bond failure is usually accompanied by shear cracks. A flexural failure signifies adequate embedment length, whereas a bond failure signifies inadequate embedment length.

The type of failure for one end of the girder dictates the embedment length for the opposite end of the girder. If a flexural failure occurs, the embedment length is decreased for the next test. If a bond failure occurs, the embedment length is increased for the next test. This iterative approach is employed until the development length is determined.

Point loads were applied by use of a hydraulic jack and were measured using a load cell. During the experimentation, vertical deflections under the load and at midspan were measured. Linear variable displacement transducers (LVDTs) were used to measure slip of each strand. All of these measuring devices were connected to a data acquisition system and were read continuously during the experiments.

Development length experimentation also will be performed for each deck panel. A line load will be applied along the full width of the panel at the midspan of the panel. This load is increased incrementally to failure, and is measured using a load cell. LVDTs attached to the end of each strand at both ends of the panel will be used to measure strand slip. Deflection at midspan will also be measured.

Results and Conclusions

Because the experimentation is ongoing as of this writing, only a limited number of results are presented here. Additional results and conclusions will be presented later.

TABLE 3 Transfer Length Results for Girders

Girder Description	Average Transfer Length (in)
0.5" Uncoated Strand @ 2", N.S.C.	46.5*
0.5" Uncoated Strand @ 2", H.S.C.	19.2
0.5" Epoxy-Coated Strand @ 2", N.S.C.	18.6
0.5" Epoxy-Coated Strand @ 2", H.S.C.	13.0
0.5" Uncoated Strand @ 1.75", N.S.C.	44.6
0.5" Epoxy-Coated Strand @ 1.75",N.S.C.	19.4
0.6" Uncoated Strand @ 2", N.S.C.	56.0
0.6" Uncoated Strand @ 2", H.S.C.	23.7
0.6" Epoxy-Coated Strand @ 2", N.S.C.	21.8
0.6" Epoxy-Coated Strand @ 2", H.S.C.	18.7

N.S.C. = Normal Strength Concrete
H.S.C. = High Strength Concrete

1 inch = 25.4 mm

* Approximate average; two values of transfer length exceeded 56 in (the limit of instrumentation), but were taken as 56 in to compute the average.

As previously described, the transfer lengths for each end of each girder and deck panel were determined from the Whittemore readings at certain intervals. Table 3 gives some of these results. Specifically, it lists average transfer lengths for girders at a concrete age of 28 days. These transfer length values represent the average of all ends of particular groups of girders, such as girders containing strands 12.7 mm (0.5 in.) in diameter and fabricated with normal-strength concrete. The values listed are those determined using average values of strain equal to 95 percent of the plateau values.

Several conclusions can be drawn from the results shown in Table 3. First, the transfer lengths of epoxy-coated strands with grit were considerably shorter than those of uncoated strands for a given strand diameter and spacing. For normal strength concrete, the transfer lengths for epoxy-coated strands were less than half those of their respective uncoated strands.

Second, the use of high-strength concrete resulted in a reduction in transfer length. The reduction was much more pronounced for uncoated strand than it was for epoxy-coated strand.

Third, the transfer lengths for uncoated strands in normal-strength concrete were considerably longer than those predicted by the AASHTO approximation of 50 times the strand diameter. For uncoated strands 12.7 mm (0.5 in.) in diameter, the transfer lengths were approximately equal to 90 times the strand diameter. For uncoated strands 15.2 mm (0.6 in.) in diameter, the values were longer than 110 times the strand diameter.

Fourth and finally, the transfer lengths for strands 12.7 mm (0.5 in.) in diameter spaced at 44.4 mm (1.75 in.) were approximately equal to those for the same diameter strands spaced at 50.8 mm (2 in.). This conclusion held for both uncoated and epoxy-coated strands.

ACKNOWLEDGMENTS

The author would like to express her appreciation to the research teams of FHWA and of Construction Technology Laboratories, who worked together during the fabrication of the members. The teamwork of the staff of Shockey Bros. precast concrete plant, as well as the work of a review committee of prestressed concrete specialists for this study, is also gratefully acknowledged. It should be noted that the contents of this paper reflect the views of the author and do not necessarily reflect the official views or policies of FHWA.

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TABLE 2 Deck Panels

Deck Panel No.	Design f_c' (MPa)	Deck Panel Type	No. Of Strands	Length of Panel (m)	Strand Epoxy Coated (E) or Uncoated (U)	Made Composite With Slab?
1, 2	34.4	A	6	2.52	U	No
3, 4	34.4	A	6	2.52	U	Yes
5, 6	34.4	A	6	2.52	E	No
7, 8	34.4	A	6	2.52	E	Yes
9, 10	34.4	B	6	2.52	U	No
11, 12	34.4	B	6	2.52	U	Yes
13, 14	34.4	B	6	2.52	E	No
15, 16	34.4	B	6	2.52	E	Yes
17, 18	34.4	C	10	3.05	U	No
19, 20	34.4	C	10	3.05	U	Yes
21, 22	34.4	C	10	3.05	E	No
23, 24	34.4	C	10	3.05	E	Yes
25, 26	34.4	D	10	3.05	U	No
27, 28	34.4	D	10	3.05	U	Yes
29, 30	34.4	D	10	3.05	E	No
31, 32	34.4	D	10	3.05	E	Yes

1 MPa = 0.145 ksi
1 m = 3.28 ft

received strand reels. Pull-out tests were later conducted on the strands in these blocks in an attempt to identify a possible measure of strand surface condition. The pull-out tests will also be used to investigate a possible correlation between pull-out values and transfer length. The results and analysis from the phosphate residue test and pull-out tests are beyond the scope of this paper.

The epoxy-coated prestressing steel was seven-wire, Grade 270, low-relaxation strand, conforming to ASTM Standard A 882-92. This strand had small aluminum oxide particles called "grit" embedded in the surface of the epoxy coating.

Concrete

The same concrete mix was used for the deck panels and for the normal-strength [34.4-MPa (5-ksi)] girders. This concrete mix was designed to obtain a 28-day compressive strength (f_c'), which was greater than or equal to 34.4 MPa (5 ksi) and less than or equal to 44.8 MPa (6.5 ksi). The mixture was also designed to have a compressive strength of 27.6 MPa (4 ksi) for prestress release in approximately 24 hr. This concrete mix consisted of Type III portland cement, sand, crushed limestone aggregate, water, an air-entraining admixture, a retarder, and a superplasticizer. The average water/cement ratio for the 16 batches was 0.44.

The precast concrete plant used cylinders 101.6 × 203.2 mm (4 × 8 in.) to determine concrete compressive strength. These cylinders measuring 101.6 × 203.2 mm (4 × 8 in.) were cured in heated containers where the temperature matched that of the heat applied to the prestressing bed for curing. When the compressive strength of these cylinders met or exceeded 27.6 MPa (4 ksi), the strands were detensioned. At that time, researchers tested cylinders 152.4 × 304.8 mm (6 × 12 in.), cured at the prestressing bed, for a corresponding concrete compressive strength at the time of prestress release. The 28-day concrete compressive strengths were determined by testing two sets of cylinders 152.4 × 304.8 mm (6 × 12 in.): one set was cured in a moist room and one set was cured in air with the members.

The average compressive strength for the normal-strength concrete mix at the time of prestress release was 31.5 MPa (4.57 ksi). The average 28-day compressive strength for this concrete was 44.2 MPa (6.42 ksi) for the set of cylinders that was air cured, and 49.2 MPa (7.15 ksi) for the set of cylinders that was moist cured.

The concrete mix used in the high-strength girders was designed to obtain a 28-day compressive strength that was greater than or equal to 68.9 MPa (10 ksi) and less than or equal to 89.6 MPa (13 ksi). The mixture was also designed to have a compressive strength of 48.2 MPa (7 ksi) for prestress release in approximately 24 hr. This concrete mix consisted of Type III portland cement, microsilica, sand, crushed traprock aggregate, water, an air-entraining admixture, a retarder, and a superplasticizer. The average water/cementitious material ratio for the four batches was 0.33.

The average compressive strength for the high-strength concrete mix at the time of prestress release was 54.3 MPa (7.88 ksi). The average 28-day compressive strength for this concrete was 71.3 MPa (10.35 ksi) for the set of cylinders that was air cured and 74.2 MPa (10.77 ksi) for the set of cylinders that was moist cured.

Fabrication

The fabrication process consisted of the following: stressing the strand, mixing the concrete at an on-site batch plant, transporting the concrete to the stressing bed via a ready-mix truck, casting the concrete, curing the concrete, and detensioning (releasing) the strand. This process was successfully repeated four times for the deck panels and 10 times for the girders.

After the stressing of the epoxy-coated strands, a few strands slipped out of their anchorages. Because of safety concerns and the lack of time available to obtain new anchorages, the decision was made by the precast plant to strip the epoxy coating off of all of the epoxy-coated strands in the anchorage areas. The tensioning was then completed as for uncoated strands.

Accelerated curing was used for all members. Steam pipes running underneath the stressing beds heated the members, which were covered with moisture retention covers. Curing continued until compression tests on concrete cylinders indicated the desired strength for prestress release. Detensioning was accomplished by flame cutting the strands for the girders and by flame cutting the strand or cutting the strand with wire cutters for the deck panels.

Instrumentation

Mechanical Gauge Points

Every member was instrumented with gauge points for measuring surface strains (called Whittemore points) at regularly spaced intervals on the concrete. Most girders had gauge points running along both sides of the top flange for the full length of the girder and along both sides of the bottom flange at the ends of the girder. A few girders had only gauge points along the bottom flange at the ends of the girders. The spacing of the gauge points was 100 mm (3.94 in.) at the girder ends and 200 mm (7.87 in.) along the midspan regions of the top flange. Each deck panel had gauge points spaced at 100 mm (3.94 in.) along the full length of both sides of the panel. The gauge points for all members were placed at the level of the strands. Because of the large number of gauge points involved, threaded brass inserts were preattached to thin steel strips. The strips were then attached with screws to the inside surface of the framework, and the points were embedded in the concrete as the concrete was cast.

Once the concrete attained a compressive strength of 27.6 MPa (4 ksi), the screws attaching the thin steel strips to the formwork were removed. The formwork was then stripped, and the mechanical strain gauge (Whittemore gauge) was used to measure the distances between the gauge points. Portable data loggers were used to collect the numerous Whittemore gauge measurements directly from the Whittemore gauge. The measurements were then downloaded into a computer. Immediately after detensioning, another set of readings was taken using the same equipment. The differences in values between the two sets of readings were used to determine the strains in the concrete after detensioning. A full set of readings was also taken at concrete ages of 7, 14, and 28 days and immediately before development length experimentation.

End Slip

The end slip of every strand at both ends of every member was measured. A small channel-shaped fixture was attached to a strand adjacent to the end of a member. Holes were bored in the legs of the fixture to accept a digital depth gauge, which was used to measure the distance from the outer leg of the fixture to the concrete surface. This distance was measured both before and after detensioning; the difference between these two values was the end slip of the strand. A full set of readings was taken at each of the intervals mentioned above.

Concrete Temperature

Thermocouples to measure concrete temperature were installed at each end and at midspan of one girder for each cast. A total of 15 thermocouples were used for a girder, spread around the cross section at each of the three aforementioned locations. The thermocouples were monitored during curing and throughout the period when the Whittemore gauge measurements were taken after detensioning.

Three thermocouples were installed at each end of one deck panel for each cast. These thermocouples were monitored in a fashion similar to that for the girders.

Other Measurements

Camber was measured for each of the girders using standard surveying equipment. Shrinkage of the concrete was monitored using cylinders 152.4 × 304.8 mm (6 × 12 in.) cast for each batch of concrete. Freeze-thaw testing was performed using prisms cast from the high-strength concrete. The coefficient of thermal expansion was also determined for both the normal and high-strength concretes.

Experimentation

All girders and deck panels were transported to the FHWA Structures Laboratory in McLean, Virginia. The composite decks for 12 of the girders and 16 of the deck panels will be cast there. Experimentation was also performed there.

Transfer Length

The transfer length of each end of each member was determined by plotting the strains calculated from the Whittemore gauge readings. The transfer length can be defined as the distance from the end of the beam to the point on the curve equal to the average value of strain for the plateau portion of the plot. Transfer lengths were determined for average values of strain equal to 100 percent of the plateau value and for average values of strain equal to 95 percent of the plateau value. This was done in an effort to compare transfer length values with other researchers who use varying definitions of transfer length. Transfer lengths were determined for each of the intervals mentioned for the Whittemore gauge measurements.

Development Length

Development length experimentation will be performed for each end of each girder. The estimated development length is first calculated and is called the embedment length. A single point load is applied at a distance from the end of a girder equal to this embedment length. This point load is increased in intervals up to failure. Failure can be one of two types: flexural failure or bond failure. Flexural failure occurs when the concrete crushes in compression or the strands break in tension. Bond failure occurs when the strands lose their bond and slip in toward the member. In I-shaped members, bond failure is usually accompanied by shear cracks. A flexural failure signifies adequate embedment length, whereas a bond failure signifies inadequate embedment length.

The type of failure for one end of the girder dictates the embedment length for the opposite end of the girder. If a flexural failure occurs, the embedment length is decreased for the next test. If a bond failure occurs, the embedment length is increased for the next test. This iterative approach is employed until the development length is determined.

Point loads were applied by use of a hydraulic jack and were measured using a load cell. During the experimentation, vertical deflections under the load and at midspan were measured. Linear variable displacement transducers (LVDTs) were used to measure slip of each strand. All of these measuring devices were connected to a data acquisition system and were read continuously during the experiments.

Development length experimentation also will be performed for each deck panel. A line load will be applied along the full width of the panel at the midspan of the panel. This load is increased incrementally to failure, and is measured using a load cell. LVDTs attached to the end of each strand at both ends of the panel will be used to measure strand slip. Deflection at midspan will also be measured.

Results and Conclusions

Because the experimentation is ongoing as of this writing, only a limited number of results are presented here. Additional results and conclusions will be presented later.

TABLE 3 Transfer Length Results for Girders

Girder Description	Average Transfer Length (in)
0.5" Uncoated Strand @ 2", N.S.C.	46.5*
0.5" Uncoated Strand @ 2", H.S.C.	19.2
0.5" Epoxy-Coated Strand @ 2", N.S.C.	18.6
0.5" Epoxy-Coated Strand @ 2", H.S.C.	13.0
0.5" Uncoated Strand @ 1.75", N.S.C.	44.6
0.5" Epoxy-Coated Strand @ 1.75",N.S.C.	19.4
0.6" Uncoated Strand @ 2", N.S.C.	56.0
0.6" Uncoated Strand @ 2", H.S.C.	23.7
0.6" Epoxy-Coated Strand @ 2", N.S.C.	21.8
0.6" Epoxy-Coated Strand @ 2", H.S.C.	18.7

N.S.C. = Normal Strength Concrete
H.S.C. = High Strength Concrete

1 inch = 25.4 mm

* Approximate average; two values of transfer length exceeded 56 in (the limit of instrumentation), but were taken as 56 in to compute the average.

As previously described, the transfer lengths for each end of each girder and deck panel were determined from the Whittemore readings at certain intervals. Table 3 gives some of these results. Specifically, it lists average transfer lengths for girders at a concrete age of 28 days. These transfer length values represent the average of all ends of particular groups of girders, such as girders containing strands 12.7 mm (0.5 in.) in diameter and fabricated with normal-strength concrete. The values listed are those determined using average values of strain equal to 95 percent of the plateau values.

Several conclusions can be drawn from the results shown in Table 3. First, the transfer lengths of epoxy-coated strands with grit were considerably shorter than those of uncoated strands for a given strand diameter and spacing. For normal strength concrete, the transfer lengths for epoxy-coated strands were less than half those of their respective uncoated strands.

Second, the use of high-strength concrete resulted in a reduction in transfer length. The reduction was much more pronounced for uncoated strand than it was for epoxy-coated strand.

Third, the transfer lengths for uncoated strands in normal-strength concrete were considerably longer than those predicted by the AASHTO approximation of 50 times the strand diameter. For uncoated strands 12.7 mm (0.5 in.) in diameter, the transfer lengths were approximately equal to 90 times the strand diameter. For uncoated strands 15.2 mm (0.6 in.) in diameter, the values were longer than 110 times the strand diameter.

Fourth and finally, the transfer lengths for strands 12.7 mm (0.5 in.) in diameter spaced at 44.4 mm (1.75 in.) were approximately equal to those for the same diameter strands spaced at 50.8 mm (2 in.). This conclusion held for both uncoated and epoxy-coated strands.

ACKNOWLEDGMENTS

The author would like to express her appreciation to the research teams of FHWA and of Construction Technology Laboratories, who worked together during the fabrication of the members. The teamwork of the staff of Shockey Bros. precast concrete plant, as well as the work of a review committee of prestressed concrete specialists for this study, is also gratefully acknowledged. It should be noted that the contents of this paper reflect the views of the author and do not necessarily reflect the official views or policies of FHWA.

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jective of the research is to define the limits at which the utilization of higher-strength concretes may no longer be structurally or cost effective. The paper then describes some solutions to overcome the limitations so that higher-strength concretes can be effectively utilized in bridge construction.

High-Strength Concrete

Concretes with compressive strengths in excess of 69 MPa (10,000 psi) have been produced commercially utilizing ready-mixed concrete at many geographic locations around the United States (*1*), including Illinois, Minnesota, New York, Ohio, South Carolina, Texas, and Washington. These concretes have been produced with a high decree of workability and pumpability. However in bridge design, a design strength in excess of 41 MPa (6,000 psi) at 28 days is hardly ever utilized. Rarely has concrete with a specified strength in excess of 69 MPa (10,000 psi) been utilized in a highway bridge structure. Consequently, here is a need to seek ways in which high-strength concrete can be effectively utilized.

Several investigations (*2–9*) have identified the advantages of using high-strength concrete in prestressed bridge girders. These advantages include fewer girders for the same width bridge, longer span lengths, or reduced dead load. The girders also will have increased durability. Studies (*5,7*) have also shown that these advantages more than offset the increased costs of high-strength concrete.

In addition to providing a higher compressive strength, high-strength concrete provides a higher modulus of elasticity, a higher tensile strength, reduced creep, and greater durability than normal-strength concrete. For the same cross section and span length, a high-strength concrete will result in less axial shortening and less short-term and long-term deflections. The higher tensile strength provides a small advantage where the allowable stress in tension controls the design. High tensile and compressive strength may be beneficial in reducing the transfer length at the ends of girders (*10*). The reduced creep will result in less prestress losses, which can be beneficial in reducing the number of strands and reducing the change in camber. Improved durability, particularly when silica fume is used, will result in a longer life for bridge girders.

Optimized Cross Sections

In the early applications of prestressed concrete, designers developed their own ideas of the "best" girder cross section to utilize. "As a result, each bridge utilized a different girder shape. Consequently, the reuse of girder formwork on subsequent contracts was not possible. As a result, girder shapes were standardized in the interest of improving economy of construction. This led to the development of the standard AASHTO-Prestressed Concrete Institute (PCI) sections for bridge girders. Types I through IV were developed in the late 1950s and Types V and VI were developed in the 1960s.

Adoption of the AASHTO standard bridge girders simplified design practice and led to wider use of prestressed concrete for bridges. Standardization resulted in considerable cost savings in the construction of bridges. However, following the original adoption of the standard AASHTO-PCI shapes, individual states again developed their own standard sections for improved efficiency and economy. In 1980, FHWA initiated an investigation to identify new optimized sections for major prestressed concrete girders.

In an FHWA study (*2,3*), Construction Technology Laboratories (CTL) found that the most structurally efficient sections were the Bulb-Tee, Washington, and Colorado girders. In an analysis for cost-effectiveness, the Bulb-Tee girder with a 152-mm (6-in.) web was recommended for use as a national standard for precast, prestressed concrete bridge girders in the United States for span lengths from 24 to 43 m (80 to 140 ft).

Subsequently, the PCI Committee on Concrete Bridges developed a modified section for use as a national standard. The modifications resulted in a slightly heavier section that was easier to produce and handle. This cross section was subsequently adopted by several states and is identified as the PCI Bulb-Tee in this paper. Several other versions of the Bulb-Tee also have been developed in various geographic locations (*11,12*).

Analyses of Existing Cross Sections

In the present analysis, the following specific cross sections were selected for analysis of their cost efficiency:

1. PCI Bulb-Tee BT-72, identified as BT-72.
2. Florida Bulb-Tee BT-72 (*11*), identified as FL BT-72.
3. AASHTO Type VI with a web 152 mm (6 in.) thick, identified as Type VI.
4. Washington Series 14/6, which is similar to a Washington Series 14 but with a web 152 mm (6 in.) thick and is identified as WA 14/6.
5. Colorado Series G68/6, which is a Colorado G68 but with a web 152 mm (6 in.) thick and is identified as CO G68/6.
6. Nebraska Section (*12*) with a depth of 1800 mm and a web thickness of 150 mm and is identified as NU 1800.