FINAL
CONTRACT REPORT

INVESTIGATION OF TRANSFER LENGTH, DEVELOPMENT LENGTH, FLEXURAL STRENGTH, AND PRESTRESS LOSSES IN LIGHTWEIGHT PRESTRESSED CONCRETE GIRDERS

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16. Abstract

Encouraged by the performance of high performance normal weight composite girders, the Virginia Department of Transportation has sought to exploit the use of high performance lightweight composite concrete (HPLWC) girders to achieve economies brought about by the reduction of dead loads in bridges. Transfer length measurements (conducted on two AASHTO Type IV HPLWC prestressed girders) indicated an average transfer length of 17 inches, well below the AASHTO and ACI requirements.

Two HPLWC AASHTO Type II girders and a 48 x 8 inch normal weight 4000-psi concrete deck were fabricated. The girders were cast of concretes with a compressive strength of 6380 psi and a unit weight of 114 pcf. Full scale testing of the girders was conducted to evaluate development length and flexural strength in HPLWC composite girders. Embedment lengths of five, six, and eight feet were evaluated. Tests indicated a development length of about 72 inches, marginally below the ACI and AASHTO requirements. All tested girders exceeded their theoretical flexural capacity by 24% to 30%.

A third composite Type II girder was cast of high performance normal weight concrete and topped with a 48 x 8 inch normal weight 4000-psi concrete deck. This girder was intended as a control specimen.

Prestress losses in the HPLWC AASHTO Type IV girders monitored over a nine-month period were found to be less than those calculated using the ACI and PCI models.

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ABSTRACT

Encouraged by the performance of high performance normal weight composite girders, the Virginia Department of Transportation has sought to exploit the use of high performance lightweight composite concrete (HPLWC) girders to achieve economies brought about by the reduction of dead loads in bridges. Transfer length measurements (conducted on two AASHTO Type IV HPLWC prestressed girders) indicated an average transfer length of 17 inches, well below the AASHTO and ACI requirements.

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INTRODUCTION

Recent years have seen the successful use of high performance concretes (HPCs) in bridge design and construction.[1,2] These bridges were built with normal weight HPCs (HPNWC) of a density of about 145 pcf. HPCs are characteristically higher in strength and more durable than their regular counterparts due to their lower water to cement ratio and denser cement matrix as a result of the mineral admixtures used in their makeup. These attributes contribute directly to desired structural and economic efficiencies in the design and construction of highway bridges.

Encouraged by the successful implementation of HPC technology in demonstration bridges built by the Virginia Department of Transportation (VDOT), the Virginia Transportation Research Council (VTRC) is seeking to exploit HPC technology further by using lighter weight concretes.

PURPOSE AND SCOPE

An experimental program was planned and executed with the general objectives of demonstrating the capability of producing prestressed American Association of State Highway and Transportation Officials (AASHTO) type girders made of high performance lightweight concrete (HPLWC) with a density of about 120 pcf and a 28-day compressive strength of 8000 psi and further studying their behavior. This program was a necessary prelude to a planned HPLWC demonstration bridge to be built over the Chickahominy River on Route 106 in Charles City County, Virginia.

The experimental program included the proportioning and testing of lightweight and normal weight concrete mixes and was chiefly concerned with investigating the effects of using lightweight concretes in prestressed girders on transfer length, development length, and flexural strength. An additional objective was to monitor prestress losses in HPLWC girders and compare those losses to estimates that were calculated from various models available in the literature.

To accomplish the objectives, five precast, prestressed AASHTO type bridge girders (three AASHTO Type II girders and two AASHTO Type IV girders) were cast and tested. Transfer lengths were measured on all girders, and the AASHTO Type II girders were tested to failure with a composite deck section added. The AASHTO Type VI girders were monitored for 9 months to determine prestress loss.

METHODS

Overview

Three prestressed test girders were produced to evaluate transfer length, development length, and flexural capacity of composite HPLWC beams. Two prestressed Type II AASHTO girders were cast of HPLWC, and the third one was made of HPNWC to serve as a control and comparison test girder. The composite decks were made of normal weight 4000-psi concrete. Two additional AASHTO Type IV girders were produced to investigate transfer length and prestress losses in HPLWC beams. The nomenclature shown in Figure 1 is used throughout this report to delineate the different test specimens used in the research investigation. Table 1 details the general properties of the test specimens and the corresponding test parameters investigated, and Table 2 contains the design specifications for each beam.

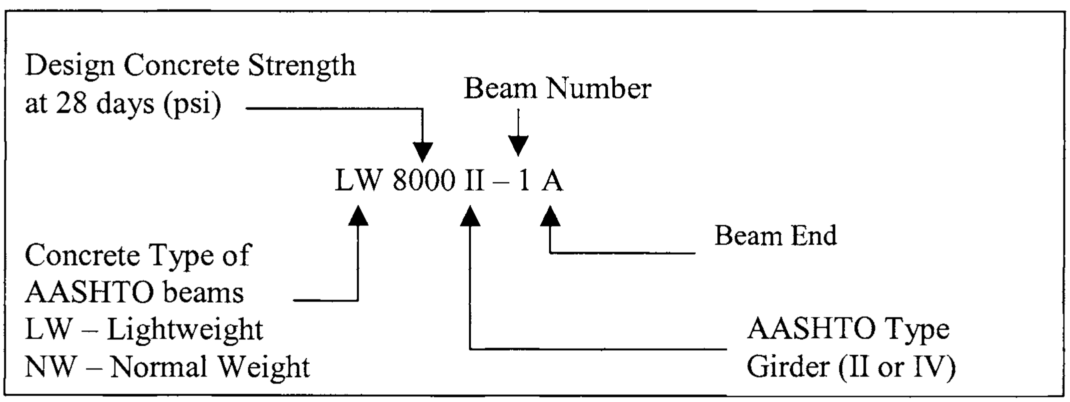

Figure 1: Test Beam Nomenclature

Table 1: General Properties of the Prestressed Test Girders

Test Girder ID	Cross Section	Beam Type (AASHTO)	Concrete Type in Beams	Deck	Length (feet)	Investigated Parameters
LW8000II-1A/B	T-Beam	Type II	Lightweight	Yes	36	Development Length, Transfer Length and Flexural Capacity
LW8000II-2C/D	T-Beam	Type II	Lightweight	Yes	36	Development Length, Transfer Length and Flexural Capacity
NW8000II-3A/B	T-Beam	Type II	Normal weight	Yes	36	Development Length, Transfer Length and Flexural Capacity
LW8000IV-4A/B	I-Beam	Type IV	Lightweight	No	84	Transfer and Prestress Losses
LW8000IV-5C/D	I-Beam	Type IV	Lightweight	No	84	Transfer and Prestress Losses

Table 2: General Design Specifications for the Prestressed Test Girders

Test Girder ID	Beam Strength at Transfer (psi)	Beam Strength at 28 days (psi)	Deck Strength at 28 days (psi)	Strand Prestress at Transfer (psi)
LW8000II-1A/B LW8000II-2C/D	5600	8000	4000	202.5
NW8000II-3A/B	5600	8000	4000	202.5
LW8000IV-4A/B LW8000IV-5C/D	5600	8000	No deck added	202.5

General Definitions and Testing Scheme

The end region of a member over which the effective prestressing stresses, f_{se}, are transmitted from the strands to the concrete is referred to as the transfer length, L_t. A generally accepted model of the variation of prestress forces in a concentrically prestressed member is shown in Figure 2.[3-5]

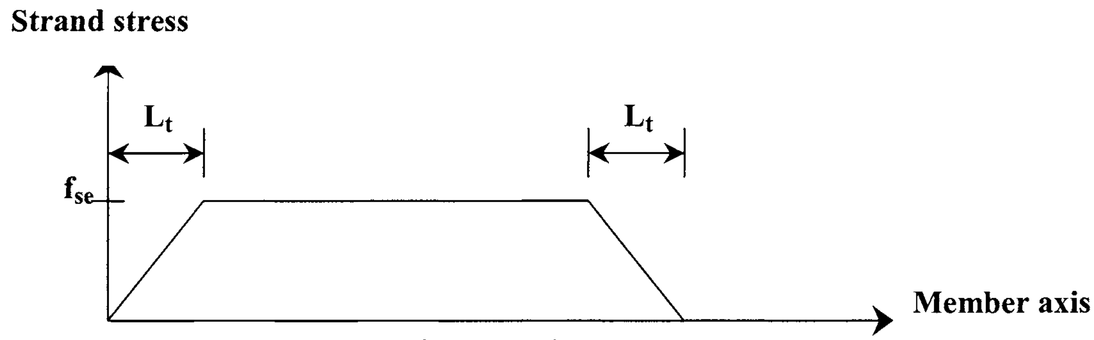

Figure 2: Variation of Stresses in a Prestressed Member

The transfer lengths in this study were established by measuring concrete surface strains in the end regions of five test specimens designed and built for the purpose of this experimental research. Two of these specimens were T-girders made of HPLWC Type II AASHTO girders and a normal weight composite concrete deck. A third specimen similar in every way to the aforementioned two was made with HPNWC. This specimen was intended to serve as a control and comparison specimen. The other two specimens were Type IV AASHTO girders made of HPLWC.

The transfer length in members prestressed with strands is to be taken as 50 strand diameters per the American Concrete Institute (ACI) building code provisions [6] and the AASHTO standard specifications for highway bridges.[7] This provision is stipulated in sections of the code addressing shear design of prestressed members. The ACI commentary R12.9, in Chapter 12, which covers "Development and Splices of Reinforcement," gives the equation for transfer length as $(f_{se}/3)\ d_b$, where f_{se} is the effective prestress and d_b is the strand diameter. An average effective prestress of 150 ksi lends the approximation of 50 strand diameters. The AASHTO Load and Resistance Factor Design (LRFD)[8] acknowledges and allows for a higher effective prestress as is normally applied in current practice by increasing the transfer length to 60 strand diameters.

ACI defines development length as the "length of embedded reinforcement required to develop the desired strength of reinforcement at a critical section."[6] In prestressed concrete members, development length is composed of two bond regions. The first bond region is the one characterized previously as the transfer length, and the second bond region is termed the flexural bond length. This flexural bond region is the additional length required beyond the transfer length for the strands to develop their ultimate flexural stress, f_{ps}. Figure 3[6] illustrates a qualitative variation of strand stress along the development length.

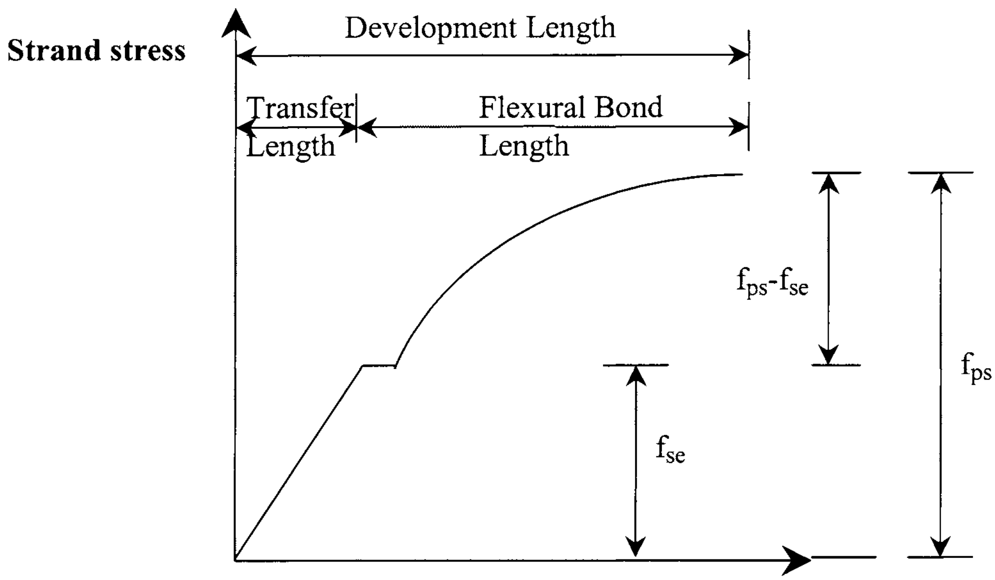

Figure 3: Variation of Steel Stress with Distance from Free End of Strand[6]

The three AASHTO Type II T-girders were loaded to failure at various embedment lengths, L_e, to establish the development length, L_d, and the flexural strength of the specimens. In theory, the development length is established when a test specimen has attained its ultimate flexural strength at the shortest tested embedment length. Figure 4 demonstrates the testing scheme adopted to arrive at the development length.

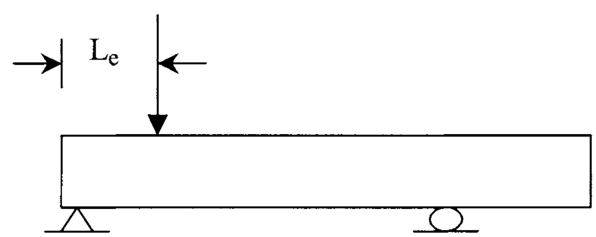

Figure 4: Development Length and Flexural Strength Testing Scheme

Section 12.9, "Development of Prestressing Strand," in the ACI code[6] and Section 9.28, "Embedment of Prestressed Strand," of the AASHTO standard code[7] stipulate a development length for girders prestressed with three- or seven-wire strands to be not less than

$$(f_{ps} - 2/3\ f_{se})\ d_b \qquad \text{Eq. 1}$$

where d_b is the nominal strand diameter in inches, and f_{ps} and f_{se} are the ultimate and effective strand stresses, respectively, in kips per square inch. The resulting data from the transfer length, development length, and flexural strength tests were compared to the relevant design provisions available in the ACI and AASHTO codes to verify their validity for HPLWC prestressed members.

Moreover, in order to monitor prestress losses in HPLWC members, the two Type IV specimens were fitted with internal vibrating wire gages that recorded concrete strains to a digital read-out over a period of about nine months. Data from these gages were compared to theoretically calculated strains that incorporated the long-term effects of creep, shrinkage, and steel relaxation.

Composite Type II Prestressed HPLWC and HPNWC Test Girders

Figure 5 shows the cross section of the composite specimens used in this investigation. An AASHTO Type II girder and a 48 x 8 inch normal weight concrete slab comprised the cross section of the test girders. Each girder was prestressed with eight ½-inch-diameter Grade 270 low-relaxation strands. Six strands were straight throughout the girder, and two were harped at 14 feet from each beam end.

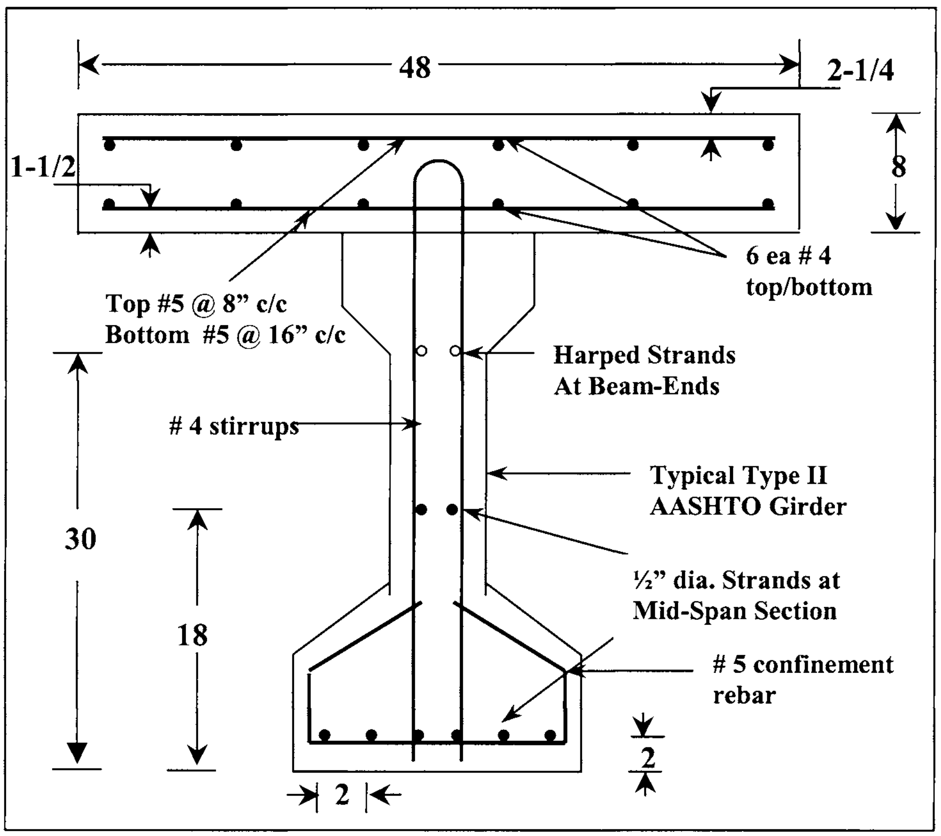

Figure 5: Prestressed Composite Test Girders Cross-Section (all dimensions in inches)

An allowable tensile stress of 100 psi at transfer of prestress was used for the beam design to ensure the absence of tensile cracks upon transfer, since lighter weight concretes often have proportionally lower tensile strength. The deck reinforcement is typical of that used throughout Virginia. The web reinforcement was detailed to guarantee composite action and sufficient shear capacity to prevent shear failure.

Mill certificates from Florida Wire and Cable, Inc., indicate that Grade 270 low-relaxation strands per ASTM A-416-93, with a 28,600-ksi modulus of elasticity, were used in the girders. All other reinforcement was Grade 60 reinforcing steel.

All decks were cast of normal weight concrete, with a design 28-day compressive strength of 4000 psi. Table 3 presents the mixture proportions.

Three batches of HPLWC were used to produce the Type II and Type IV girders. Results for the Type IV non-composite girders are discussed here since these girders were cast at the same time as the Type II composite test girders. The Type II girders were cast out of the first batch and the Type IV girders were cast out of the second and third batches. Even though the concrete mixture proportions in the three batches was the same, the 28-day compressive strength recorded for the first, second, and third batches was, 6660 psi, 6320 psi, and 6140 psi, respectively.

In all succeeding calculations and representations of strength in this report, an average 28-day compressive strength of 6375 psi is used for both the Type II and Type IV HPLWC prestressed girders.

The mixture proportions for the Type II and Type IV HPLWC girders were used to achieve a compressive strength of at least 8000 psi at 28 days with a unit weight of about 120 lb/ft^3. The concrete mixture, however, did not develop the desired 28-day strength. In fact, the highest average strength recorded at the time of testing (180 days of age) was 6680 psi and the actual average unit weight of the mixture was 114 lb/ft^3. At transfer, the compressive strength was 4780 psi, 15% lower than the specified strength of 5600 psi. Tables 3 and 4 present the HPLWC mixture proportions and the strength gain over time, respectively. It is suspected that the undesired high water-cementitious materials ratio in the mixture resulted in its lower strength. However, a subsequent mixture developed for use in the actual lightweight prestressed concrete girders for the Chickahominy River crossing incorporated the desired water-cementitious materials ratio and normal weight fine and coarse aggregates to increase its density to 120 lb/ft^3. Moreover, the lightweight aggregates used in the mixture were limited to a ½-inch maximum diameter size instead of the ¾-inch aggregate used in the first mixture.

A single composite Type II test girder was made with HPNWC. The mixture for the HPNWC was also designed to develop a 28-day compressive strength of 8000 psi. Included in Tables 3 and 4 are the mixture proportions and strength gain for the concrete. As observed, the mixture attained a 28-day compressive strength of 7800 psi, less than the design strength.

The modulus of elasticity and the split tensile strength of the concrete were tested at 28 days of age. The values recorded for the three batches of the lightweight concrete used to cast the Type II and Type IV lightweight girders mirrored the trend seen in the recorded values of the concrete mix strength. The modulus of elasticity and split tensile strength recorded for the first and second batches were higher than that recorded for the third batch, even though the same mixture proportions were used for all three batches. Table 5 lists the 28-day average modulus of elasticity and split tensile strength obtained from testing.

Table 3: Concrete Mixture: Prestressed Composite Test Girders

Material	LW8000II-1 LW8000II-2	LW8000IV-4 LW8000IV-5	NW8000II-3	Deck
Water (lb)	250	250	255	285
Type II Cement (lb)	451	451	451	381
Fly Ash (lb)	301	301	301	254
NW Coarse Aggregate (lb) (SSD)	0	0	1873	1873
Lightweight Coarse Aggregate (lb) STALITE (SSD)[a]	800	800	0	0
Natural Sand (lb)	1419	1419	1208	1278
Superplasticizer (oz)	56	56	56	---
Retarder (oz)	22	22	23	50
Air Entraining Agent (oz)	12	12	12	16
Unit Weight (lb/ft^3)	114	114	145	145

[a]Stalite is lightweight aggregate produced from expanded slate. Carolina Stalite Company supplied the material for this research.

Table 4: Concrete Strength Gain

Test Girder	Concrete Strength (psi)			Specification (psi)
	1 day	28 days	180 days	28 days
LW8000II-1 LW8000II-2	4775	6375	6675	8000
LW8000IV-4 LW8000IV-5	4775	6375	6675	8000
NW7000II-3	6040	7800	8990	8000
Deck	N/A	4500	N/A	4000

Table 5: Tested Modulus of Elasticity and Split Tensile Strength

'est Girder	MOE x 10^6 (psi)	MOE (test) / $\gamma^{1.5}\sqrt{f_c'}$	Split Tensile Strength, f_{ct} (psi)	$\frac{f_{ct}}{\sqrt{f_c'}}$
LW8000II-1 LW8000II-2	2.82	29	537	6.72
LW8000IV-4 LW8000IV-5	2.82	29	537	6.72
NW7000II-3	4.90	31.8	845	9.57

As shown in Table 5, the modulus of elasticity for the lightweight concrete was about 12% lower than the theoretical value calculated per section 8.5.1 of the ACI code.[6] The average split tensile strength was about $6.7\sqrt{f_c'}$ for the lightweight concrete and $9.6\sqrt{f_c'}$ for the normal weight concrete. Naaman[9] lists observed ranges for split tensile strength of lightweight and normal weight concretes as $4\text{-}5\sqrt{f_c'}$ and $6\text{-}7\sqrt{f_c'}$, respectively, both of which are considerably lower than was found in this research investigation.

Beams LW8000II-1 and LW8000II-2 were cast with the lightweight concrete mixture, and beam NW8000II-3 was cast with the normal weight concrete mixture. One day after concrete placement and steam curing, the side forms were stripped. Detensioning of the strands then proceeded immediately through torching each of the eight strands.

The concrete mixture used for producing the HPLWC beams failed to reach the design strength of 5600 psi required before detensioning of the strands could occur. The strength attained was 4780 psi, approximately 15% lower than the design strength. A decision was made to detension the strands despite the lower concrete strength recorded. The transfer operation was successfully completed, and no visible splitting cracks were observed at the transfer zone(s).

The normal weight concrete decks topping each of the three girders were placed one month later, and shored construction was used. The shear reinforcement used in the AASHTO girders extended upward into the slab, providing composite action of the girder and deck.

Non-Composite Type IV Prestressed HPLWC Test Girders

The main purpose of this research was to evaluate the behavior of HPLWC prestressed girders prior to using the material in the Chickahominy River crossing. Two Type IV test girders of similar design to the ones specified for the Chickahominy Bridge were used to investigate transfer length and prestress losses in the girders. The design and detailing of the girders were performed by VDOT. Figure 6 shows the cross section of the test girders and the detailing of the prestressing strands and reinforcing steel.

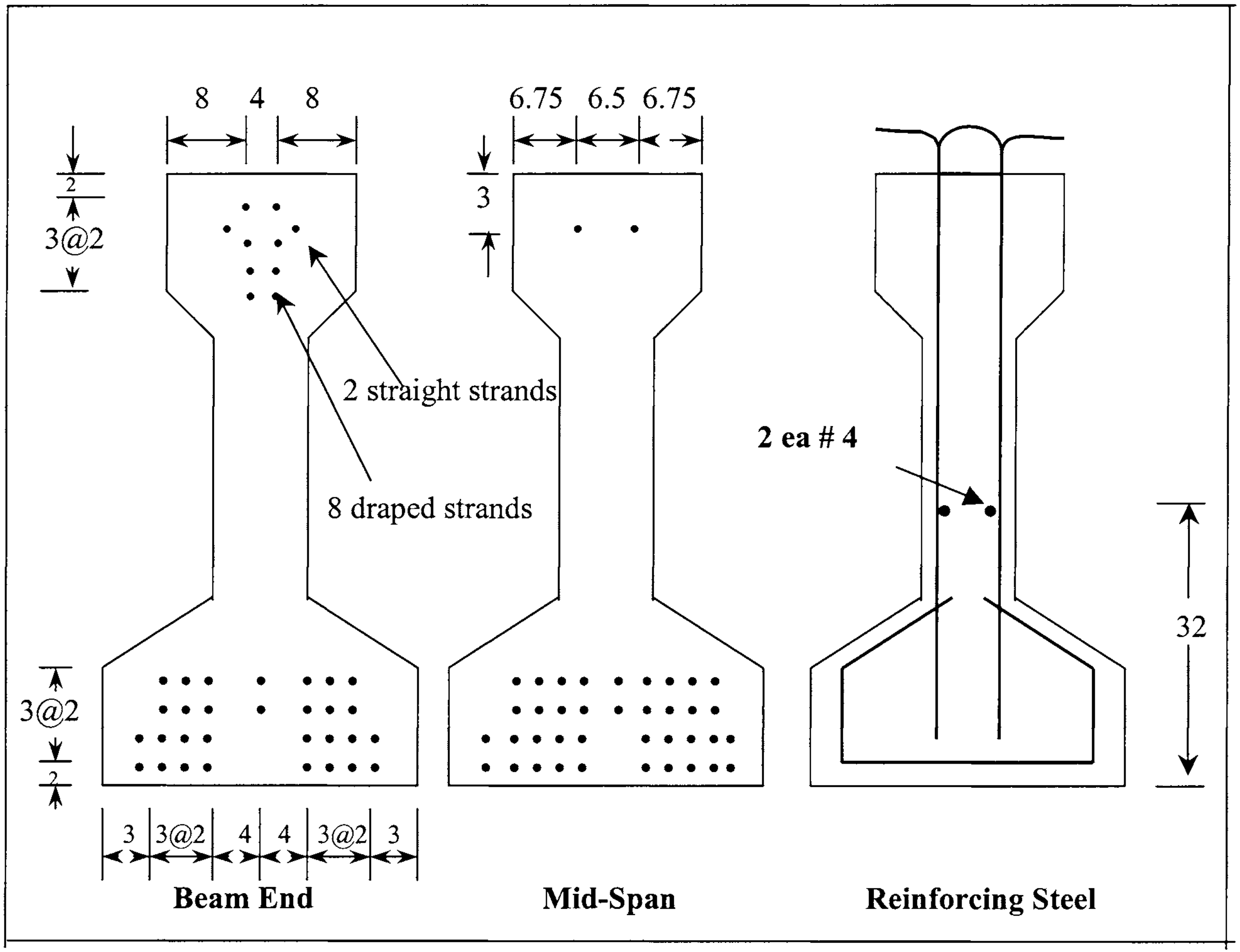

Figure 6: Non-Composite Type IV Prestressed Test Girders

All reinforcement and strands used for the Type IV HPLWC prestressed test girders were of the same specification as those used in the AASHTO Type II test girders. The same mixture proportions used for the Type II test girders was used for the Type IV HPLWC girders. The forming, prestressing, and placement operation for the Type IV test beams was as described for the Type II girders.

Testing and Instrumentation

This section describes the testing procedures adopted and the instrumentation used in the test beams to investigate transfer length, development length, flexural strength, and prestress losses.

Transfer Length

The transfer length is the distance over which the prestressing force is fully transmitted from the strands to the concrete. Assuming that the prestress force varies linearly from zero at the end of the beam to the effective prestress force at the end of the transfer length,[3] a means of measuring the transfer length is to measure the concrete surface strains along the transfer zone.

To that end, metallic strips fitted with threaded inserts spaced four inches apart were attached to the side forms on both sides of each end of each girder prior to casting of the concrete. After stripping of the side forms, the day after concrete placement, the metallic strips were removed from the sides of each girder to result in flush embedment of the threaded inserts along both sides of each end of the girders. The inserts spaced four inches apart, stretched a distance of six feet from the end of the beam. The first inserts were placed two inches from each end of the test girder.

A Whittemore strain measurement device, with an eight-inch gauge length, was used to measure the distance between the embedded inserts immediately after the stripping of the side forms. This records the zero or reference state of the concrete surface strain. A second set of measurements was taken after detensioning to record the strain state on the concrete surface after prestressing. The difference between the first and second Whittemore readings divided by the 8-inch gauge length provides the strains induced in the concrete surface by the prestressing. Another set of measurements was also taken one week after detensioning. Figures 7 indicates the layout of the Whittemore inserts in the girders.

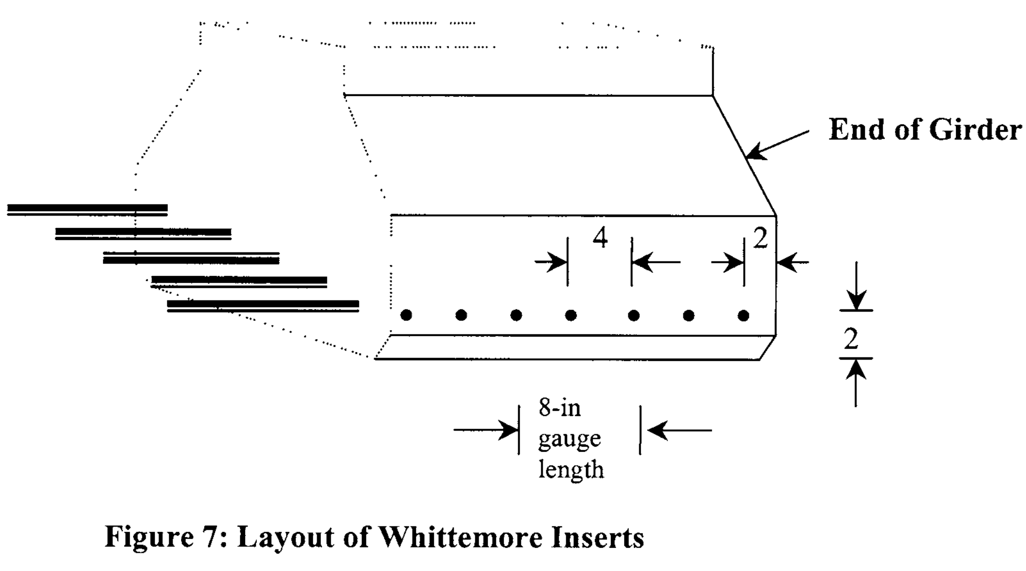

Figure 7: Layout of Whittemore Inserts

Development Length and Flexural Strength

Full-scale flexural testing was conducted to evaluate the development length and flexural capacity of the composite test girders. Should the reinforcement embedded in the concrete be shorter than is required, a bond slip occurs before the flexural capacity of the beam is reached.

In order to arrive at the development length and flexural capacity of the beams, a point load was applied to failure at varying embedment lengths. A 24-foot-long test span was chosen to allow for two flexural tests per test beam.

By loading the beam to failure, assuming sufficient shear strength, a flexural failure would indicate that the tested embedment length was longer than the development length. However, strand slippage and/or failure of the beam prior to development of its flexural capacity would indicate that the tested embedment length is shorter than the development length. As such, the next test setup should examine a longer embedment length to arrive at the development length. This iterative process was the methodology adopted to arrive at the development length. Four overall tests were carried out on the two Type II HPLWC composite beams to evaluate their development length and flexural capacity.

Development length and flexural strength testing were conducted at Bayshore Concrete Products in Cape Charles, Virginia. Two concrete blocks two feet high were used as supports for the test beams. Elastomeric pads were placed on top of each concrete block (24 feet apart, center-to-center). The tested end of the beam was lined up such that the centerline of the elastomeric pad was six inches from the end of the beam. The pad extended the full width of the flange and 12 inches along the beam's axis. Figure 8 shows the load frame and the test girder during one of the tests.

In order to monitor beam behavior during loading and at failure, several data collection devices (DCDs) were used to record beam deflections at regular intervals along the length of the beam, strand slippage at the beam end, load intensity at the ram, and internal strains in the flanges and deck. These devices were in turn connected via cabling to a computerized data acquisition system. The DCDs for the internal strain were connected to a separate digital read-out. This section describes the function and property of these devices.

A 300-kip-capacity load cell placed underneath the hydraulic ram and on top of a swivel placed on the deck, was connected to the data acquisition system to record the load intensity applied to the beam during the testing. The load cell was calibrated to a plus or minus 1% precision.

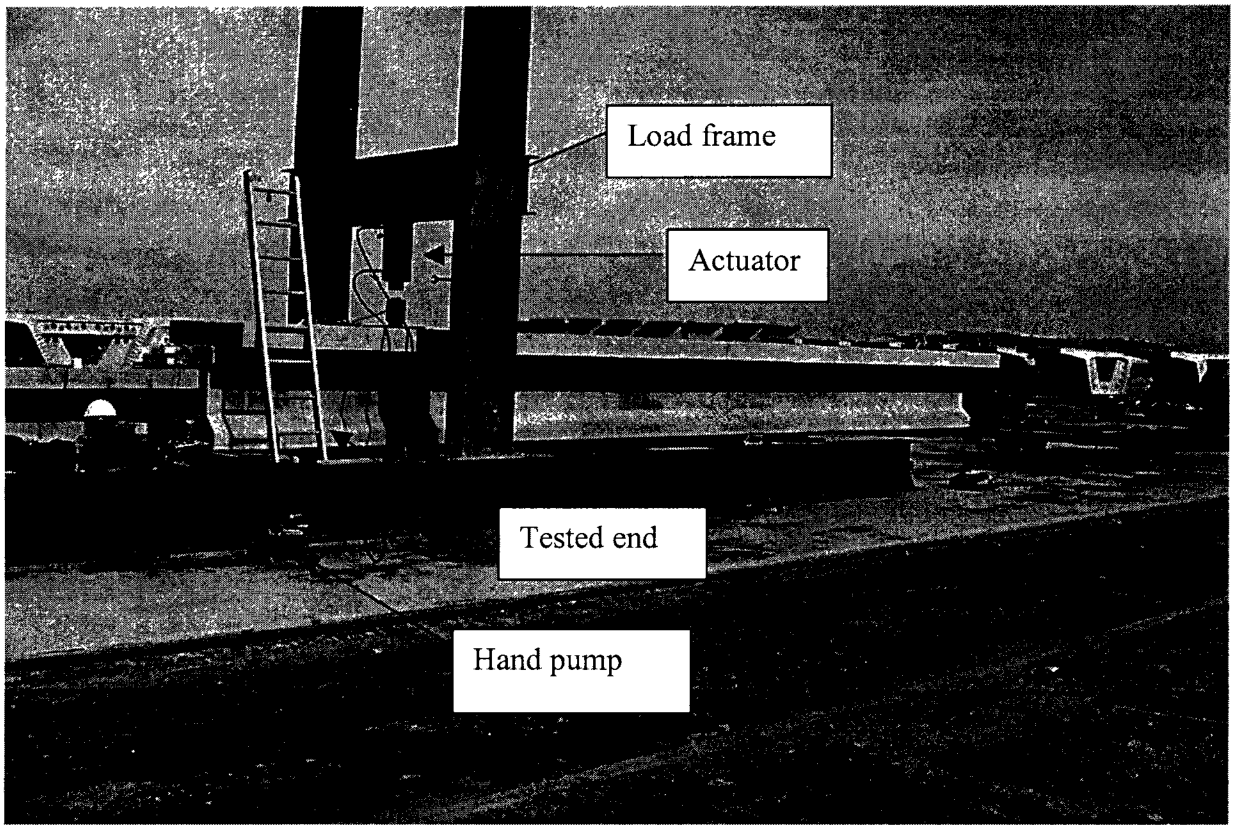

Figure 8: Load Frame and Test Setup

Wire pots or vertical displacement measuring potentiometers were placed six feet from the beam end, and at nine, 12, and 18 feet from the support to record the displacement profile of the beam. The data accuracy of these wire pots was on the order of plus or minus 0.01 inch. As in the case of the load cell, these wire pots were interfaced to the data acquisition system for data collection.

To monitor strand slippage while loading the beam to failure, linear variable differential transformer (LVDT) devices were fastened to the protruding stands at the tested end of the beam. The LVDTs recorded slippage in the strands with an accuracy of plus or minus 0.003 inch. A slip of 0.01 inch was considered indicative of bond failure.

Three vibrating wire strain gages, Model VCE 4200, by Geokon Inc., were embedded at eight feet from one end of each composite test girder, at the centroids of the lower flange, upper flange and deck. Vibration frequencies, of a pretensioned steel wire between two metallic casings that made up the strain gages, denoted the movement and the corresponding strains induced by the relative displacement of one metallic casing with respect to the other. A companion digital read-out was interfaced to the vibrating wire gauges to record the detected internal strains. Internal strain readings were manually recorded at 25-kip load increments to failure.

The data acquisition system consisted of the following:

1. notebook with 8MB RAM, 500 MB hard disk, and a Windows 95 operating system

2. multi-port MEGADAC System Series 3100, for data collection. Each port accommodated four-channel bridging cartridges that interfaced with the DCDs

3. TCS software interfaced the operating system and the MEGADAC, which allowed for continuous recording of data from the DCDs.

After all the DCDs were connected to the test beam, load was applied in increments of 25 kips. A remote control operated the 300-kip hydraulic ram. Internal strain readings were recorded manually at the end of each load increment for tested beam ends, which were fitted with internal strain gauges. At cracking, the load was held constant for a short period to allow the study team to visually inspect and map the emerging cracks in the beam. Loading was then resumed to failure. In some of the tests, the load approached 290 kips, which was deemed the highest load that could safely be supported by the test assembly, and the test was terminated. In one instance, the beam failed by yielding, followed by rupture of a strand.

The data set denoting beam deflections, strand slippage, and load intensity was continuously recorded by the MEGADAC during the testing at the rate of 10 scans per second. At the end of each test, the cracks were mapped and pictures were taken to record the final crack patterns.

Prestress Losses

Three vibrating wire gauges were embedded at the centroid of the prestressing strands of the Type IV test girders, at mid-span, and at six inches on either end of the mid-span. Internal strains were recorded at two-hour increments on a digital read-out device for nine months after detensioning. Thermocouples to monitor change in temperature in the concrete were placed adjacent to each of the three vibrating wire gauges.

RESULTS

Transfer Length Results

Concrete surface strain profiles produced from the Type II test beams were not decipherable. Low prestressing forces on the Type II beams led to strains on the order of 90 to 180 micro strains at the transfer ends. The Whittemore gauge could not precisely record such low strains. Moreover, some of the Whittemore inserts were damaged during the removal of the metallic strip that attached them to the side forms. Therefore, evaluation of the transfer lengths of Type II girders was not possible. The following discusses results for the Type IV girders.

Sixteen concrete surface strain readings were taken on each side of each end of each beam. Each strain reading represented the average strain over the eight-inch Whittemore gauge length. These strain data are referred to here as raw strain data. Strain profiles (variation of strains along the beam ends) produced from these raw strain data were quite irregular, a fact that could be attributable to high variations in the modulus of elasticity along the concrete surface of the lightweight beams and to human inaccuracies in measurement data.

Because the inserts were spaced four inches apart, successive strain readings overlapped four inches with the preceding ones. By averaging three successive strain data readings at a time, the effect of the irregularities are reduced and a smoother strain profile is produced. This technique is referred to as smoothing.[9] Figure 9 illustrates raw and smoothed strain profiles produced from strain readings taken on one side of beam end LW8000IV-4A.

Further reduction of the data was done by averaging the "smoothed" strain profiles on both sides of each end of a test beam for both beam-ends and for all beam ends. These averaged results (labeled ASSP) are shown in Figure 10. The 95 % average maximum strain (AMS) method[10] was used to determine the transfer length for the HPLWC Type IV test beams. The procedure for the 95 % AMS method was as follows:

1. Determine the point at which the strain profile begins to plateau.

2. Compute the average strain for all points within the plateau. This represents the 100 % average maximum strain.

3. Calculate 95% of the AMS and plot its line.

The intersection of the 95% AMS line with the sloping smoothed strain profile produces the transfer length. Figure 10 demonstrates the use of the 95 % AMS method.

The transfer length determined from this strain profile was 17 inches, which is less than that calculated using the ACI/AASHTO code (50 d_b, or 25 inches in this case). Consistent with findings from other research investigations, the live beam ends (beam ends closest to the prestressing jack) had longer transfer lengths than did the dead beam ends.

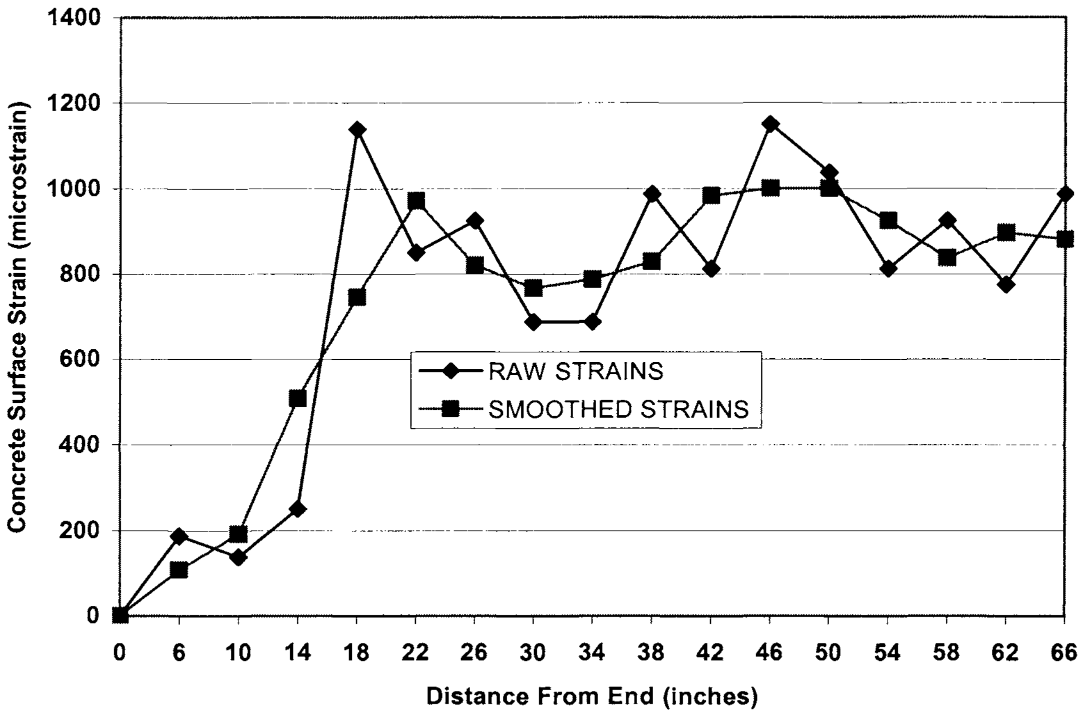

Figure 9: Raw and Smoothed Strain Profiles (SSP) for One Side of LW8000IV-5C

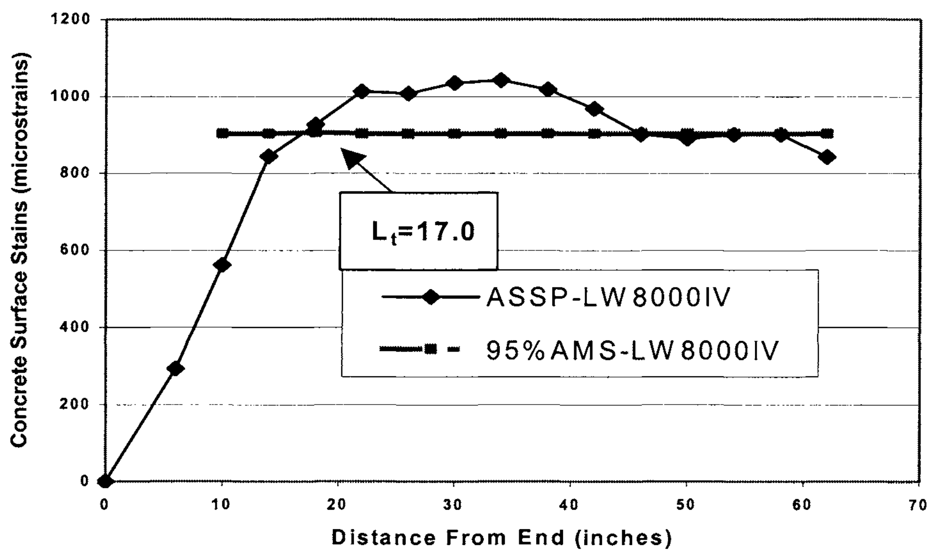

Figure 10: Averaged Strain Profile for Both HPLWC Type IV Test Beams

Development Length Tests Results and Analysis

Six full-scale flexural tests were undertaken to investigate the development length and flexural strength of the tested specimens. The first embedment length tested was six feet. The development length calculated from equation 1 per ACI/AASHTO was 78 inches.

There was a limitation on the intensity of loading that could be applied during the flexural tests due to the strength of the bolted connection between the header beam of the load assembly and the hydraulic ram. The testing load frame was designed to withstand the maximum anticipated load that could be carried by the test girders based on the girder's theoretical flexural capacity. Actual loads applied however exceeded the design loads by as much as 30 %, as will be seen in the succeeding discussion and results.

Two of the six tests (T2 and T6) were terminated at maximum loads of 280 kips and 290 kips, respectively. Table 7 lists the tests in the order in which they were conducted and the embedment length investigated.

Table 7: Flexural Tests Designation

Test No.	Specimen ID	Embedment Length (in)
T1	LW8000II-1A	72
T2	LW8000II-1B	60
T3	LW8000II-2C	72
T4	LW8000II-2D	96
T5	NW8000II-3B	96
T6	NW8000II-3A	72

From this point onward in the discussion of the test results, the delineation for the test specimens includes the embedment length. For example, the test specimen used in Test T1 will be referred to as LW8000II-1A-72 to denote a 72-inch embedment length. Deflections were monitored at 5.5, 9, 12, and 18 feet from the support nearer the tested end. Also, strand slip and load intensity were monitored and recorded continuously using a high-speed data acquisition system.

Due to a computer malfunction, data from the first test (T1) were lost, and hence load-deflection and load-slip curves were not reproducible for that specific test. However, the cracking and failure loads were manually recorded and are discussed in the following flexural strength and cracking moment discussions. Test T3 was conducted to reexamine a six-foot embedment length and to recapture the data lost from the first test conducted.

A gap in the deflection and slip data for Test T4 exists between 175 kips and 225 kips of loading. Test T4 was successfully carried out up to 175 kips, when another computer malfunction occurred. The test was then stopped, and the beam unloaded. The reloading of the test beam resulted in reproducible data for deflections and slip, which resumed at 225 kip of loading to failure. A combination of the test data from these two test attempts did provide intelligible load-deflection and load-slip graphs, albeit with a gap in the curves.

Prior to a discussion of the test results, the following sections briefly detail the test beams' engineering properties and the prestress loss models used to calculate the amount of prestress in the beams at the time of testing and at its end of service.

Composite Test Beam Engineering Properties

Table 8 lists the engineering properties for the HPLWC and HPNWC composite test beams. The modulus of elasticity for the beams and deck were calculated per Section 8.5.1 of the ACI code,[6] based on their 28-day compressive strength. The transformed composite sections were based on the modulus of elasticity of the concrete in the beams.

Table 8: Composite Test Beam Properties

Specimen ID	Beam MOE (ksi)	Deck MOE (ksi)	Moment of Inertia (in^4)	Unit Weight (plf)
LW8000II-1/2	3210	3820	173,000	692
NW8000II-3	5030	3820	148,000	772

The jacking force in a strand was determined from calibration tables, which translated pressure gauge readings on the jack to equivalent loads in pounds. A measurement of the strand elongation along the prestressing bed assisted in verifying the applied jacking forces. For the Type II beams, an anomaly in the pressure gauge readings dictated determining the jacking force from the recorded elongations. The strands were initially prestressed with a pre-load of about 4000 pounds to remove slack. The recorded elongation of the strand after preloading was about 9.5 inches. With a distance of approximately 130 feet between the prestressing abutments, this translated into a force of approximately 26,900 pounds and a total jacking force of 30,900 pounds per ½-inch strand, or 202 ksi (0.75 f_{pu}). Instantaneous losses due to slip of strands at the prestressing chucks, steel relaxation, and elastic shortening of the concrete at transfer were all calculated, and the initial prestress, f_{si}, was determined.

Effective prestress, f_{se}, at the time of testing (180 days after detensioning) was calculated in order to compare the theoretical cracking moments to the test cracking moments. The models used to calculate long-term losses (TL) due to steel relaxation (REL), creep (CR), and shrinkage (SH) were the ACI209R-92[11] model and the Prestressed Concrete Institute (PCI) model.[12] Table 9 is provided as an illustrative sample of the calculations undertaken to arrive at the effective prestresses in the HPLWC test beams.

Table 9: Effective Prestress for the HPLWC Test Beam (Per PCI Model)

Parameter (ksi)	Time Interval in Days			
	0 to 30	30 to 90	90 to 180	180 to End of Service
f_{si}	184.6	176.9	173.6	172.1
REL	1.2	0.3	0.2	1.0
CR	2.7	1.1	0.7	2.6
SH	4.1	2.0	0.6	3.1
TL	8.0	3.4	1.4	6.8
f_{se}	176.5	173.6	172.1	165.3

Ultimate Moments and Failure Modes

The flexural test results are presented in Table 10. Ultimate moments in the test beams were calculated from the ACI and AASHTO equations ($M_{ACI/AAHSTO}$) for strength and strand stress. Note that the ACI and AASHTO approximate methods for calculating strength yield identical results for the test beams. Ultimate moments were also calculated using the strain compatibility analysis method (M_{COMP}) for comparison purposes. The ACI/AASHTO equation provides an approximate value of the stress in the strand at failure. The strain compatibility analysis is a more accurate evaluation of the strand stress at failure. It is based on the generally accepted assumptions that plane sections remain plane, the compatibility of strains between the strand and the concrete (i.e., perfect bond), and equilibrium of forces. The ACI/AASHTO codes allow the use of either method. As shown in Table 10, there is close agreement between the two methods with a difference of about 1%. The ACI/AASHTO equation is generally more conservative.

Table 10: Development Test Results: Ultimate Moments

Test No.	Specimen ID	$M_{ACI/AAHSTO}$ (k-in)	M_{COMP} (k-in)	M_{TEST} (k-in)	Failure Mode	$\frac{M_{TEST}}{M_{ACI/AASHTO}}$
T1	LW8000II-1A-72	11500	11700	14700	Flexure[a]	1.27
T2	LW8000II-1B-60	11400	11600	12600	Bond/Shear[b]	1.11
T3	LW8000II-2C-72	11500	11700	14300	Flexure/Bond	1.24
T4	LW8000II-2D-96	11600	11800	15100	Flexure	1.30
T5	NW8000II-3A-72	11500	11700	15200	None[c]	1.32
T6	NW8000II-3B-96	11600	11800	15900	Flexure	1.37

[a]Slip data were not available. The data were lost due to a computer malfunction during testing.
[b]Testing was stopped at 280 kips. Higher loads were deemed unsafe to the load frame.
[c]Testing was stopped at 290 kips for the same reason indicated previously.

Generally, three modes of failure were observed: flexure, flexure/bond, and bond/shear failures. Flexural failure was characterized by excessive flexure and shear-flexure cracks, yielding of the strands, and subsequent crushing of the compression zone concrete in the deck. In one instance (NW8000II-3B-96), the flexural failure was characterized by a simultaneous crushing of the concrete and rupture of one prestressing strand.

Flexure/bond failure was similar to the flexural mode failure; however, significant bond slippage was recorded prior to the failure of the beam.

Bond/shear failure of beam LW8000II-1B-60 was characterized by a widening shear crack at the near support of the beam at about 250 kip of loading, which caused local disjointing at the lower flange and simultaneous inception of bond slippage, with increased loading. The theoretically computed ultimate load for that beam was about 256 kips. For this reason, the failure mode of this beam was classified as shear failure accompanied by general bond failure as observed from the recorded strand slippage. A reexamination of the shear capacity of the Type II composite beam calculated per ACI 318-99,[6] given the lower concrete strength (6780 psi) and the short development length tested (5 feet) in this specimen, shows a reduced shear strength of about 247 kips. This is consistent with the shear crack and disjointing exhibited by the specimen at around 250 kips. Formation of a tie in the lower flange as shown in Figure 11 accelerated the progression of the incipient web shear crack into the lower flange and hence the noted disjointing during the testing. It is this deep beam action that enabled the beam to carry additional loading beyond its shear capacity. The load applied to this beam reached 280 kips.

Cracking Moments

Cracking moments observed during flexural testing corresponded to moments in the beam at the first visible signs of cracking. Cracking moments based on the predicted effective prestresses (predicted by the ACI and PCI models at the time of testing) were calculated using the concrete strength at the time of testing. Table 11 presents the actual ($M_{CR\text{-}TEST}$) and theoretical cracking moments ($M_{CR\text{-}ACI}$ and $M_{CR\text{-}PCI}$) in the test beam.

Table 11: Cracking Moments

Specimen ID	$M_{CR\text{-}ACI}$ (k-in)	$\frac{M_{CR\text{-}TEST}}{M_{CR\text{-}ACI}}$	$M_{CR\text{-}PCI}$ (k-in)	$\frac{M_{CR\text{-}TEST}}{M_{CR\text{-}PCI}}$	$M_{CR\text{-}TEST}$ (k-in)
LW8000II-1A-72	9146	0.99	9436	0.96	9073
LW8000II-1B-60	9091	0.86	9377	0.83	7823
LW8000II-2C-72	9146	0.88	9436	0.85	8055
LW8000II-2D-96	9262	0.95	9561	0.92	8805
NW8000II-3A-72	9986	0.94	10033	0.94	9376
NW8000II-3B-96	10102	0.98	10155	0.97	9854

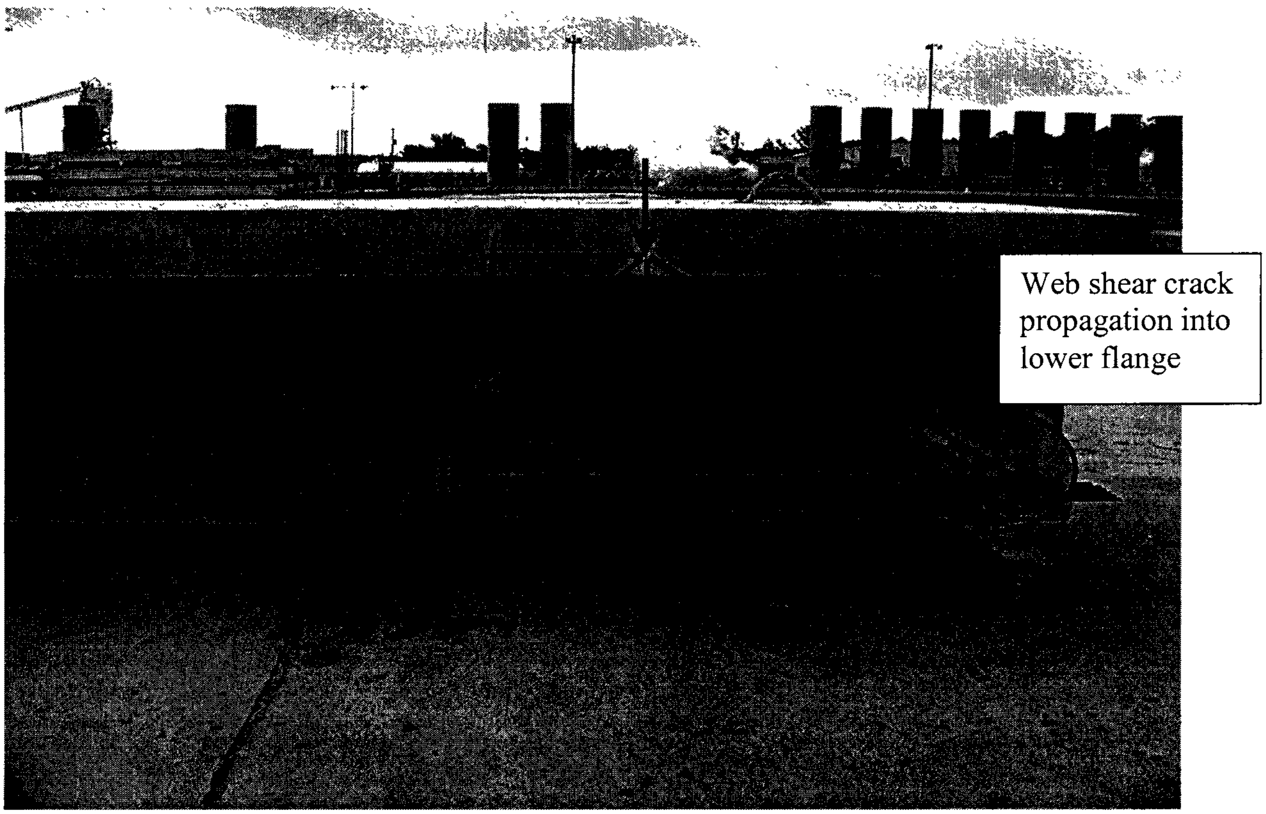

Figure 11: Strut and Tie Model: Shear Crack Propagation in Specimen LW8000II-1B-60

Load Deflection and Load Slip Behavior

Load deflection curves were obtained for each test. Wire pots located at 5.5, 9, 12, and 18 feet from the support recorded the beam deflections during the testing. Figures 12 and 13 depict the load deflection and load slip behavior for Test T3 on beam LW80000II-2C-72. Slip was recorded via LVDTs attached to the strands. Bond failure was defined as occurring when slippage greater than 0.01 inch occurred in any of the strands. This particular beam exhibited a flexural/bond failure.

In general, the load deflection curves for the lightweight beams were tri-linear. The curves varied linearly to cracking and then reflected an elasto-plastic behavior in the beams as steel yielded, followed by a plastic behavior characterized by increased deflections with little additional loads applied until concrete crushed at failure. The load deflection curves for the normal weight concrete beams were bi-linear in shape. The curves increased linearly to cracking and then slightly beyond cracking, after which yielding of steel induced increasing deflections in the beam followed by ultimate failure by crushing of the concrete.

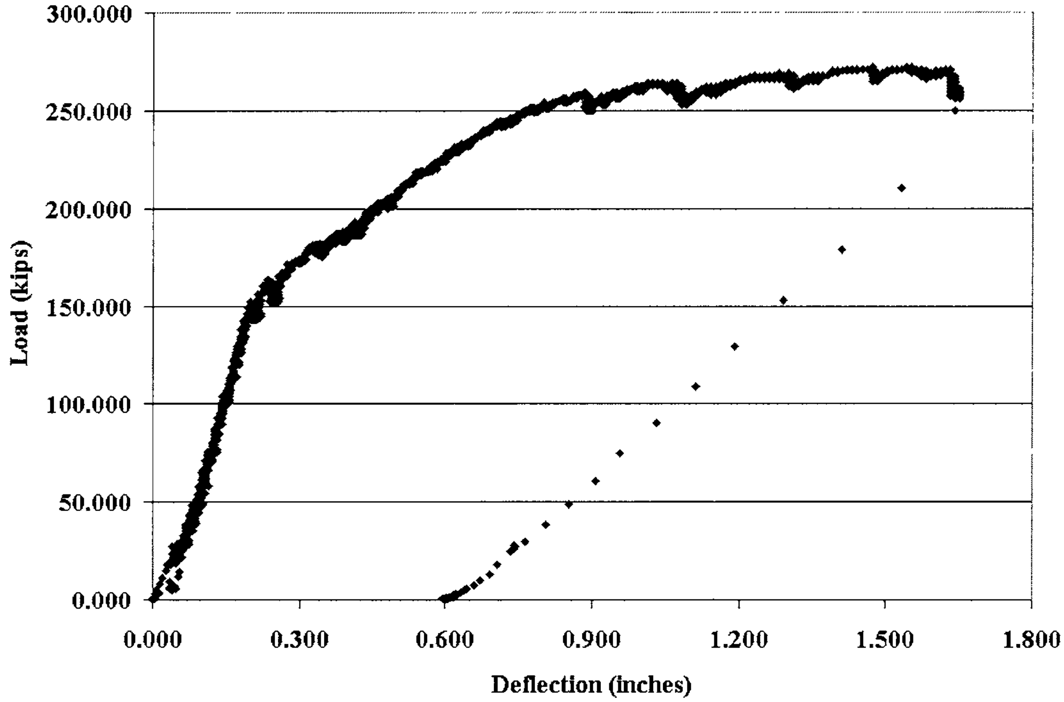

Figure 12: Load Deflection Curve: Beam LW8000II-2C-72

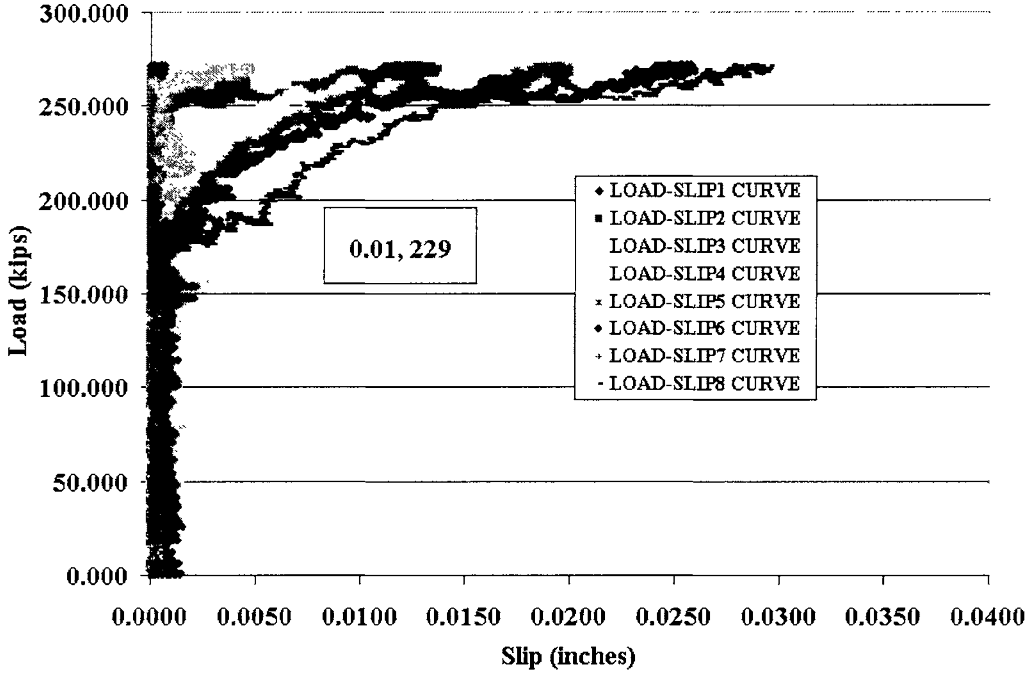

Figure 13: Load Slip Curve: Beam LW8000II-2C-72

Figure 14 illustrates superimposed load deflection curves for a lightweight and normal weight test beam for comparison purposes. The resulting linear segments of the curves are delineated with a dashed line. Note that, even though the lightweight and normal weight beams were both AASHTO Type II beams, the compressive strength for the normal weight beams was 8990 psi at the time of testing compared to 6680 psi for the lightweight beams, and the density of the lightweight concrete (114 pcf) was about 79% of the normal weight concrete's density (145 pcf).

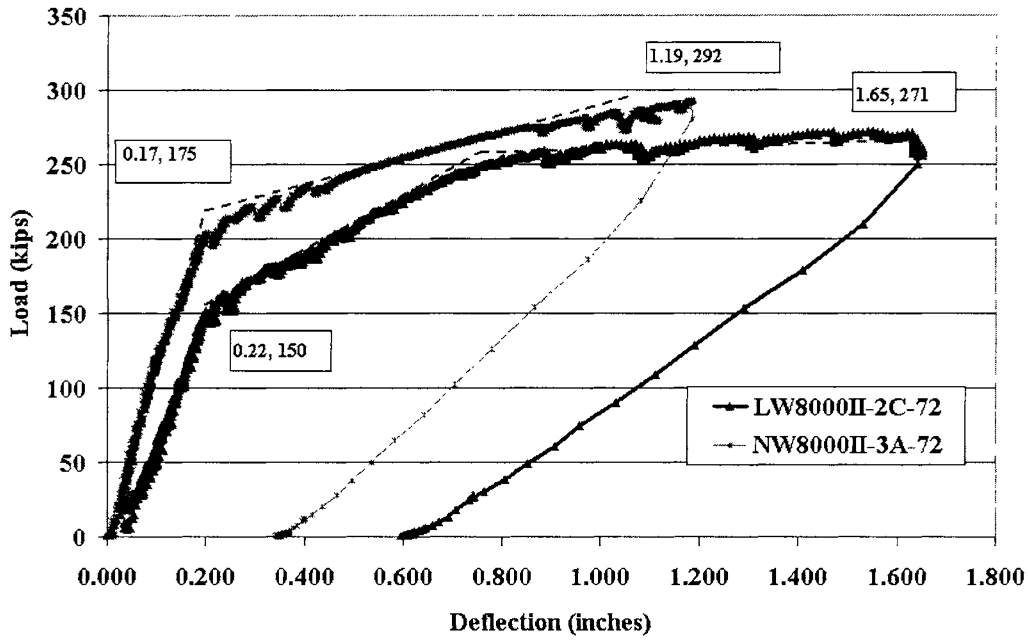

Figure 14: Typical Load Deflection Curve for HPLWC/HPNWC Beams

Strand Slip and Development Length

There are two types of bond failure in beams prestressed with multi-wire strands: General and ultimate bond failures.

General bond slip represents significant slip at the free end of the strands, considered to be a slip of 0.01 inch or greater in this research investigation. At ultimate bond failure, the strands continue to slip without an additional increase in load/stress. Mechanical resistance to slip between general and ultimate bond failure is provided by the interlock between the concrete and the helically shaped strands.

It is worthwhile indicating here that the development length equation in the ACI/AASHTO codes was intended to serve as a reasonable mean representing general bond failure.[13,14] Table 12 indicates the maximum slip in the strands recorded during testing.

Table 12: Maximum Strand Slip Recorded During Testing

est o.	pecimen ID	ax. Strand lip (in)	o. of Strands /slip > 0.01	ailure Mode	$_{TEST}$ / $_{ACI}$
T1	LW8000II-1A-72	N/A	N/A	Flexure[a]	1.27
T2	LW8000II-1B-60	0.04	6/8	Bond/Shear[b]	1.11
T3	LW8000II-2C-72	0.03	4/8	Flexure/Bond	1.24
T4	LW8000II-2D-96	<0.01	None	Flexure	1.30
T5	NW8000II-3A-72	<0.01	None	None[c]	1.32
T6	NW8000II-3B-96	<0.01	None	Flexure	1.37

[a]Slip data were not available. Data lost due to computer malfunction during testing.
[b]Test was stopped at 280 kips for safety purposes. Beam was not loaded to failure.
[b]Test stopped at 290 kips for safety purposes.

Crack Patterns

In general, lightweight beams exhibited cracks that were closely spaced, as indicated in Figure 15. At failure the widest crack was about 3/16 inch wide. Vertical flexural cracks dominated the zone under the load, and inclined flexure-shear cracks dominated the zones on either end of the load. Due to the weaker tensile strength of the lightweight concretes, web shear cracking was more prevalent than in the normal weight concrete beams. Prior to failure, most lightweight beams tested featured extensive flexure and flexure-shear cracks. Redistribution of increasing bending and shear stresses in the lightweight beams by progressive cracking led to increased elasto-plastic behavior and ductility in the lightweight beams. This is most evident from the tri-linear shape of the load-deflection curves of the HPLWC beams.

Cracks in the normal weight beams were further apart than in the lightweight concrete beams as can be seen in Figure 16. There were considerably fewer web shear cracks than in the lightweight beams due to the higher tensile strength of the normal weight beams. At failure the cracks were about ½ inch wide. As higher bending stresses were induced into the beams, the flexural cracks grew wider, and with increasing shear stresses, web shear cracks appeared on one side of the loading zone.

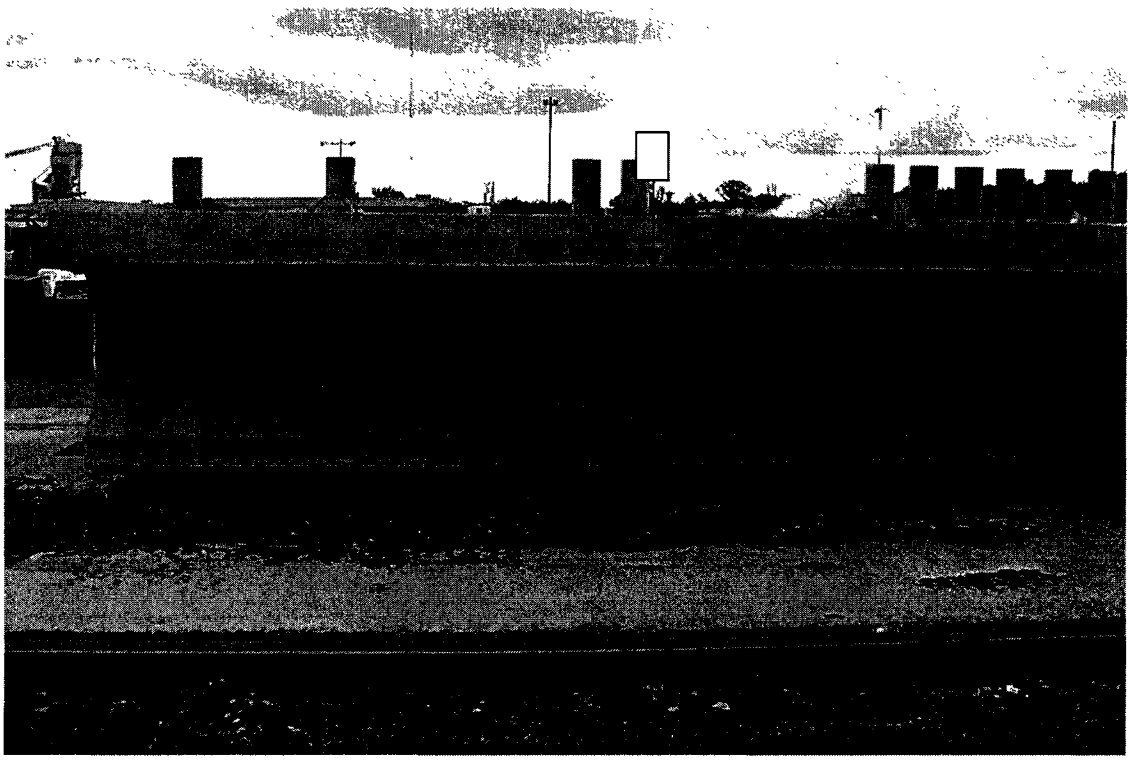

Figure 15: Crack Pattern for Beam LW8000II-1A-72

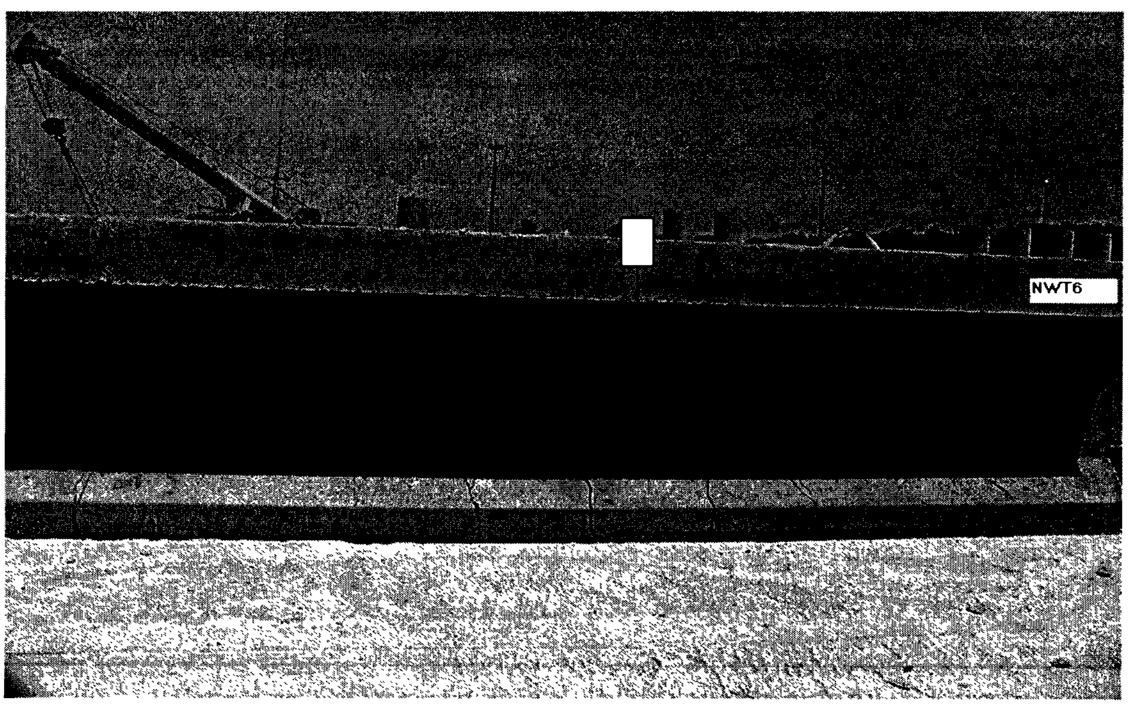

Figure 16: Crack Pattern for Beam NW8000II-3A-72

Prestress Loss Results

A secondary objective of this research investigation was to monitor and compare prestress losses in HSLWC AASHTO Type IV girders to estimates derived from existing loss prediction models.

The two AASHTO Type IV specimens, LW8000IV-4 and LW8000IV-5, produced for this purpose were fitted with internal vibrating wire gauges, which measured concrete strains at the center of gravity of the prestressing strands. Three vibrating wire gauges were installed in each beam: One at the center of the beam span and two placed six inches on each side of the center. A data logger recorded strain readings from the gauges at two-hour intervals. A temperature sensor was contained in each vibrating wire gage and these temperature readings were recorded by the data logger as well.

In order to assess directly the prestress losses in the beams, it would have been necessary to measure separately the amount of creep and shrinkage occurring in the beam during the testing. Since this was not the objective of the research, a qualitative assessment of the prestress loss that is encountered in the HPLWC Type IV prestressed girders is achieved through a comparative analysis of the concrete strains measured directly from the girders throughout a nine month period to theoretical strains derived from creep, shrinkage, and relaxation losses determined from the ACI and PCI models.

As indicated, concrete strain readings from three gauges in each girder were recorded bi-hourly. Since the gauges were six inches apart along the girder's axes, the significance of the differences in the girders self weight on the concrete stresses and strains at the level of the strands on the different gauges was neglected. Also, since both girders were cast out of the same HPLWC mixture, data from the six gauges internal to the girders were averaged on a daily basis to provide average concrete strains at the center of gravity of the strands in the test specimens after transfer. Averaging the data also smoothed the effects of the daily temperature changes in the concrete on the recorded concrete strains.

The resulting test data were then compared to theoretical strains determined using a time-step method[3] using the ACI and PCI prestress loss models.

Time dependent losses in prestressed concrete, namely creep, shrinkage, and steel relaxation are interdependent. Creep and steel relaxation are a function of the prestress force in the concrete, which is itself decreasing due to the effects of creep, shrinkage, and relaxation. This interdependency is accounted for by a summation procedure that takes into account all of these variables within a discrete time interval. For the purpose of this study, a weekly time interval was used.

The procedure was as follows:[3]

1. At the beginning of each time interval, the concrete strains at the bottom and top fibers ($\varepsilon_t, \varepsilon_b$) of the section are calculated and the strains at the level of the strands (ε_s) are then derived.

2. The creep strain ($\Delta\varepsilon_{CT}$) at the level of the strands is computed by multiplying the creep coefficient, from the PCI or ACI model during the time interval, by the strains at the level of the strand.

3. The total change of strain in the level of the strand is computed by adding the shrinkage strain increment ($\Delta\varepsilon_{sh}$) during the time interval to the creep strains calculated in Step 2.

4. The total change in strain computed in Step 3 is then multiplied by E_{ps} (Modulus of Elasticity of the strand) and added to the steel relaxation to obtain the total loss in prestress during the time interval.

5. Concrete strains ($\Delta\varepsilon_s$) corresponding to the decrease in the prestress losses during the time interval are calculated as Step 1.

6. The net strain (ε_{snet}) at the level of the strands is computed by adding the strains in Steps 1, 3, and 4.

The initial prestress used in the time stepped analysis was calculated by subtracting the instantaneous losses of elastic shortening, steel relaxation, and anchorage slippage from the jacking force.

Figure 17 shows the actual variation of strains over time in the HPLWC Type IV girders and the corresponding theoretical strains for the ACI and PCI models.

As seen from the curve, the PCI method closely models the HPLWC behavior over time. Generally, the actual concrete strains were less than those predicted by the ACI and PCI models.

Table 13 compares the actual and theoretical strains at 1, 8, 29, 57, 180, and 266 days, respectively.

Table 13: HPLWC Type IV Beams: Actual and Theoretical Strains[1]

DAY	TEST (uε)	PCI (uε)	ACI (uε)	PCI / TEST	ACI / TEST	ACI / PCI
1	-801	-634	-634	0.79	0.79	1.00
8	-683	-762	-858	1.12	1.26	1.13
29	-718	-786	-981	1.09	1.37	1.25
57	-722	-858	-1113	1.19	1.54	1.30
183	-840	-963	-1318	1.15	1.57	1.37
266	-905	-982	-1368	1.09	1.51	1.39

1 negative sign denotes compressive strains

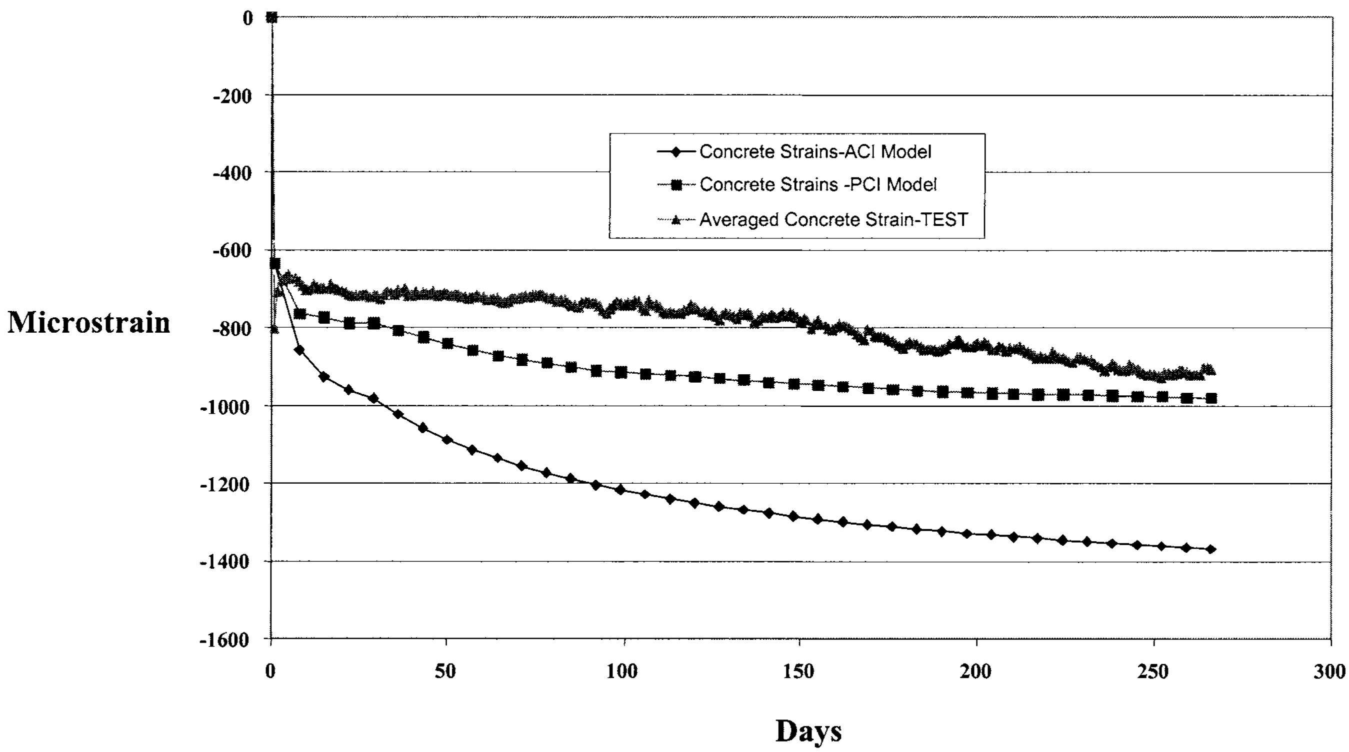

Figure 17: Variation of Concrete Strains vs. Time

DISCUSSION

Transfer Length Test Results

There are few research investigations that address the effect of using high-strength lightweight concretes on transfer length. Among these emerging studies are one performed at Purdue University by Peterman et al.,[15] and the University of Texas at Austin, by Kolzos.[16] Table 14 details the results from these two studies and those derived from this research study (labeled Virginia Tech).

It is evident that the transfer length tests presented in this report produced the shortest average transfer length (17 inches) in the available research findings. This is consistent with findings from work done by Russell and Burns,[10] which found that specimens with larger cross sections exhibit shorter transfer length. In particular their findings have shown that AASHTO type beams exhibit transfer length 28% to 40% shorter than is found in small rectangular test specimens, such as the ones used in Purdue's research. In this research study, the transfer length was about 21% shorter than that recorded at Purdue University.

Multi-strand prestressed AASHTO type girders simulate more closely the transfer length to be found in pretensioned members used in actual bridge construction. In this research investigation, the cross sections, and prestressing applied is typical of the pretensioned members that will be used in the construction of the planned demonstration bridge. Therefore, the transfer length to be expected in the pretensioned members to be used in the Chickahominy Bridge should closely mirror the findings of this research investigation.

The transfer length results from the University of Texas at Austin (UTA) are interesting. The specimens were AASHTO Type I beams. The concrete strength at transfer was higher than was recorded in our specimens and the concrete density of the elements was higher than that recorded for this research work. Yet, the average transfer length recorded was about 35.5 inches, which is about 200% of that recorded in our investigation. The standard deviation for UTA's test data sample was about 4.5 inches.

A report attributed to Logan[17] by Peterman et al.[15] in their investigation of the transfer length of semi-lightweight concrete concluded that there is a significant difference in the bond performance in prestressed concrete beams among strands supplied by different manufacturers, such as was the case between the strands used in the research study of UTA and this research study. Whether this could singularly explain the large disparity in the transfer lengths results between UTA's tests and this research investigation is unclear. As such, it is recommended that further investigation of transfer length is necessary in order to reach a more conclusive interpretation of transfer length results for HPLWC prestressed beams. It would be beneficial to use strands from different manufacturers and adopt different AASHTO cross sections as test specimens.

Table 14: Comparison of Transfer Length Results with Other Researchers' Results

Research Institution	Specimen Type	Concrete Density (lb/ft^3)	No of ½-inch strands	f_{si} (ksi)	f_{ci} (psi)	f_c (psi)	$L_{t\,(test)}$ (in)	No. Of Specimens
Purdue University	Rectangular 4 x 6 inch	130	2	190	5620	7960	21.5	2
University of Texas at Austin	AASHTO Type I	120	12	180	4900	8130	35.8	2
University of Texas at Austin	AASHTO Type I	120	12	180	5560	7850	34.8	3
Virginia Tech (i.e., current study)	AASHTO Type IV	114	40	180	4480	6680	17	2

Table 15: Comparison of Transfer Length to Codes and other Researchers' Recommendations

Research Institution	$L_{t\,(test)}$ (in)	f_{si} (ksi)	f_{ci} (psi)	E_{ci} (ksi)	Lt = 50 d_b ACI/AASHTO standard	Lt-test / Lt-code	Lt = 60 d_b AASHTO LRFD	Lt-test / Lt-code
Purdue University	21.5	190	5620	3670	25	0.86	30	0.72
UTA	35.8	180	4900	3040	25	1.43	30	1.19
UTA	34.8	180	5560	3240	25	1.39	30	1.16
Virginia Tech (i.e., current study	17	180	4480	2690	25	0.68	30	0.57

The findings from Purdue University and the current study are well within the code stipulations of the ACI, AASHTO standard, and AASHTO LRFD as shown in Table 15. UTA's results are well beyond the range of the current codes, including the AASHTO LRFD stipulation of 60 d_b. The average transfer length from UTA's study is about 20% (six inches) longer than the AASHTO LRFD code.

Pending further research, it is recommended for lightweight concrete that all codes be upgraded, including the ACI and AASHTO standard to 60d_b or (f_{si} .d_b)/3. The equations for transfer length were always intended as mean representations. Raising the ACI/AASHTO standards equation by 20% will still maintain that intent and should be re-evaluated when additional research data becomes available.

A keen reading of the development of the transfer length guidance in the ACI/AASHTO standard code indicates that the 50 d_b stipulation for the transfer lengths was based on the premise that the effective prestress at the time of testing done by Hanson and Kaar[18] was 150 ksi, which when coupled with the formula derived by Mattock[19] {$Lt = f_{se}.d_b/3$}, lends the 50 d_b guidance. In today's practice, most transfer length data referred to in the literature are determined from measurements conducted immediately after detensioning. Hence, it is logical to use the initial prestress rather than the effective prestress in the formulation of the transfer length as has been suggested by Buckner[14] and Shahawy et al.[20] Moreover, as is readily observed in Tables 14 and 15, the initial prestress in today's practice falls within the range of 180 ksi and higher. As such, the 60d_b guidance for transfer lengths stipulated in AASHTO LRFD is more appropriate to today's practice. Whereas it may not be conservative for some transfer lengths results as seen from the study conducted by UTA, it is still a more conservative mean representation than the current ACI and AASHTO standard stipulation of 50 d_b.

Development Length Test Results

Ultimate Moments

Figure 18 shows that all the HPLWC prestressed test beams exhibited a flexural capacity exceeding the design strength predicted by the ACI and AASHTO codes by about 25% to 30%. Test T2, Specimen LW8000II-1B-5, was terminated prior to ultimate failure of the beam as stated. Despite that, the test moments reached have surpassed the ACI/AASHTO ultimate moments.

In general, the test failure moments of the HPLWC beams were lower than their HPNWC counterparts by about 5%. Lower stiffness in the beams resulted in higher curvatures for the same test load in the lightweight beams than in the normal weight beams. Lightweight concrete is weaker in tension, which expedited cracking and the migration of the neutral axis toward the upper compression fibers and, hence, failure of the beams.

Lightweight beams exhibited greater ductility and energy absorption capacity than the normal weight beams. This is evident from the recorded ultimate deflections and the areas under the load-deflection curves for similar development lengths tested.

Figure 19 presents the ratio of the ultimate to cracking moments recorded during testing and compares that to the same ratios derived from calculating the ultimate and cracking moments from the ACI/AASHTO codes. It is evident that the beams' capacity to withstand increased loading beyond cracking is higher than the code predictions.

Consistent with the observed increased ductility in the lightweight beams, it is again seen here that those beams exhibit higher flexural capacity beyond cracking than the normal weight ones.

It is necessary to note that AASHTO/ACI design strength equations produce the same ultimate strength for the HPNWC composite beam and the HPLWC beams, even though the prestressed beams used are of differing strength and density. This is because the neutral axis at failure falls within the normal weight concrete deck that tops both beams and therefore the strength of the composite HPNWC and HPLWC beams is similar to that of any equivalent rectangular prestressed normal weight beam with the same reinforcement.

Given this discussion, it is necessary to caution the reader that the test moments observed in this study seem considerably high (25% to 37%). This is attributed in part to the use of neoprene pads at both supports. The neoprene pads were used to simulate the condition of the test girder in the bridge. The friction provided by the pads provided some lateral restraint at both supports and hence added to the flexural capacity of the test girders. Moreover, the theoretical values for the flexural capacity of the girders were based on an ultimate strand stress of 270 ksi and ultimate concrete compression fiber strains of 0.003. It is generally accepted that contemporary prestressing strands could withstand higher ultimate stresses and that concrete compressive strains higher than 0.003 could occur.

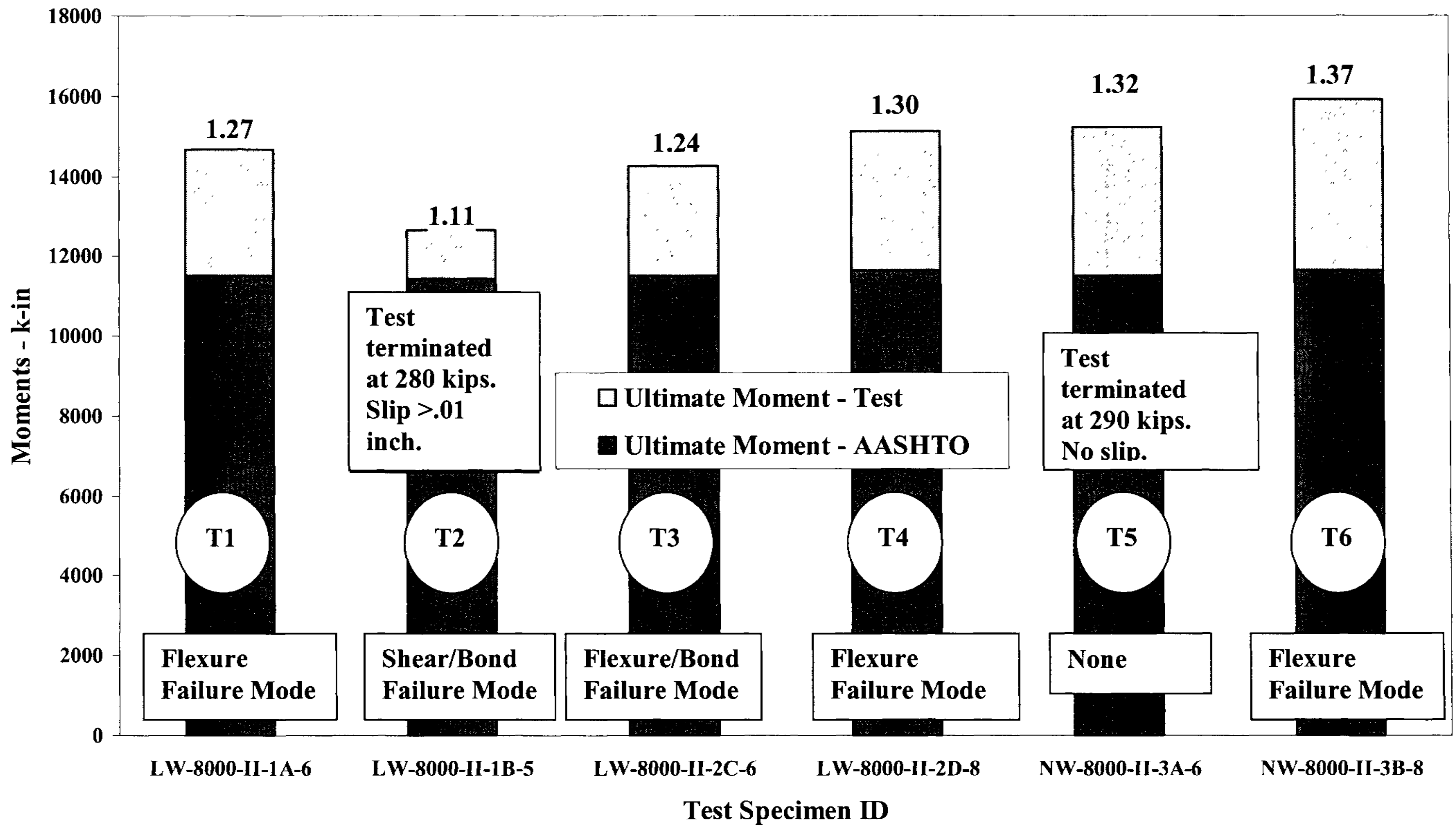

Figure 18: Ratio of Ultimate Moments at Test to Ultimate by AASHTO/ACI

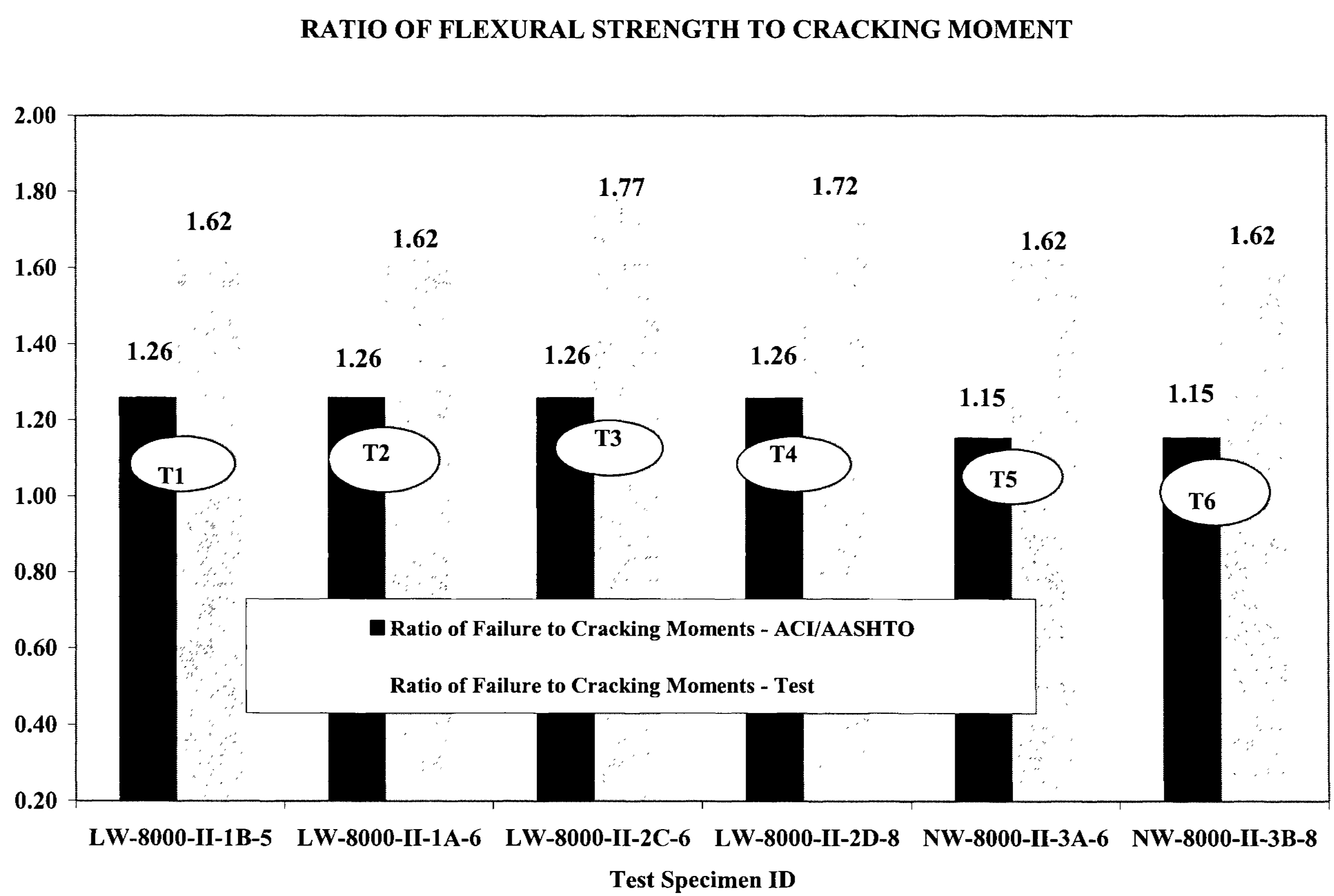

Figure 19: Ratio of Ultimate to Cracking Moments

Cracking Moments

Figures 20 and 21 are a comparison of the actual cracking moments recorded during testing and the theoretical cracking moments derived from elastic analysis of the sections using effective prestresses derived from the ACI and PCI models. The modulus of rupture, f_r, used in the computation of the theoretical cracking moments for the lightweight beams was calculated per the guidance of section 9.5.2.3 (b) of the ACI code, where f_r is equal to 0.85 ($7.5\sqrt{f_c'}$). Where the split tensile strength, f_{ct}, is specified the ACI allows substituting $f_{ct}/6.7$ for $\sqrt{f_c'}$. The average split tensile strength for this concrete mixture was 537 psi. The values of $f_{ct}/6.7$ and $\sqrt{f_c'}$ (f_c'= 6380 psi) were essentially equal, and hence no substitution was done to compute the modulus of rupture.

It can be readily seen from Figure 20 that the lightweight beams cracked at a modulus of rupture lower than that provided by the code. With the exception of Test T1 in which the emergence of first cracks may have been missed, the remaining beams cracked at moments 86% to 95% of the theoretical cracking moments. As will be seen in the prestress loss discussion, the ACI and PCI models predict higher prestress losses than are actually present in HPLWC beams. It is therefore unlikely that the higher theoretical cracking moments compared herein are due to prediction of higher effective prestress in the beams by the ACI/PCI prestress loss models.

Assuming prestress forces in the test beams are as predicted by the ACI model, the equivalent modulus of rupture for the cracking moments recorded during tests T2 to T4 ranges about 60% to 85% of the code provisions ($3.75\sqrt{f_c'}$- $5.45\sqrt{f_c'}$).

The cracking moments recorded during testing of the normal weight beam are closer to the theoretical cracking moments than were the lightweight ones, as shown in Figure 25. The test cracking moments were about 94% to 98% of the theoretical values of the cracking moments.

The modulus of rupture for the lightweight beams appears to be lower than the code stipulation. Naaman[9] quotes observed values for modulus of rupture ranging from $5\sqrt{f_c'}$ to $9\sqrt{f_c'}$. He further elaborates that the code stipulation on modulus of rupture 0.85 ($7.5\sqrt{f_c'}$) are based on the lower limit of these.

It is clear that the moduli of rupture observed in this research warrant further investigation and possible revision of the code with regard to the modulus of rupture of lightweight concretes of the density and strength used in this research investigation. Furthermore, it is interesting to note that the correlation between the average split tensile strength measured in the lab, 537 psi ($6.7\sqrt{f_c'}$), and the tensile strength recorded during flexural testing ($3.75\sqrt{f_c'}$- $5.45\sqrt{f_c'}$) is poor. This is one more reason for more conservative limits on the allowable tensile stresses and/or modulus of rupture used for lightweight concretes of the density and strength used in this research investigation. The difference in the curing conditions and the testing procedures combined add up to a wide variation between actual in-situ recorded tensile strength and lab-measured values.

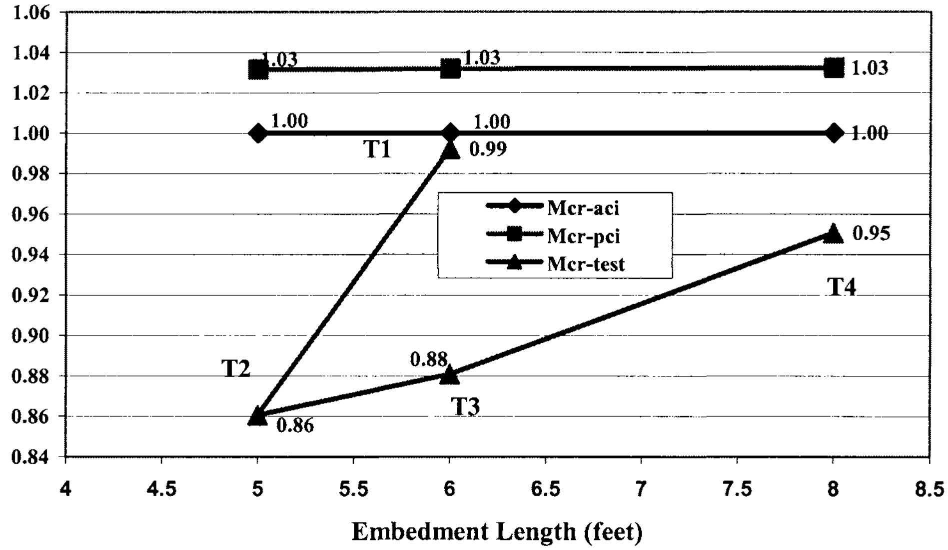

Figure 20: Ratio of Test and Theoretical Cracking Moments for the HPLWC Test Beams

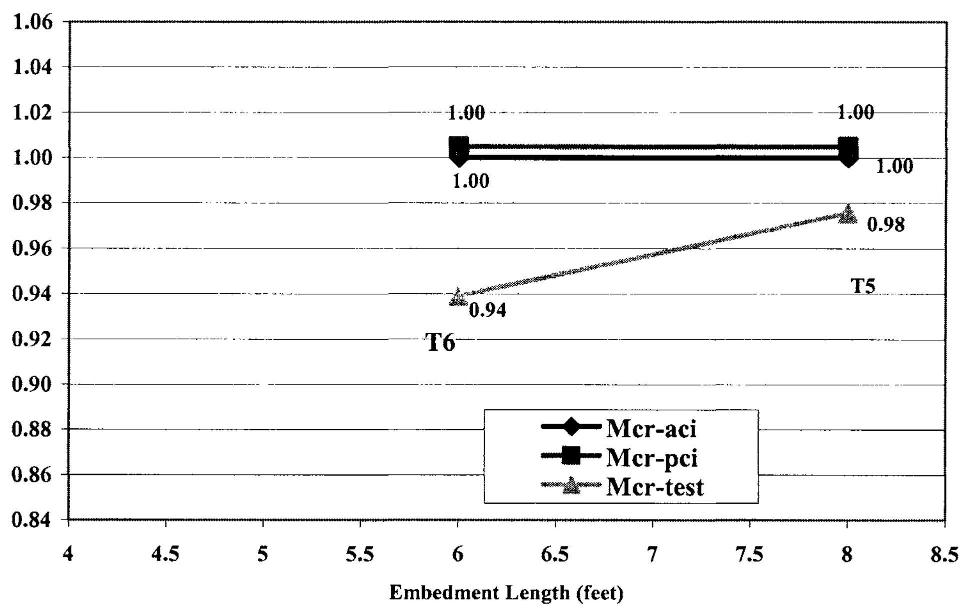

Figure 21: Ratio of Test and Theoretical Cracking Moments for the HPNWC Beams

Development Length Summary

Table 16 summarizes the findings of the development length testing. The theoretical development length, L_d, was calculated in accordance with the ACI/AASHTO codes formula, $(f_{ps}-2/3f_{se})\ d_b$. The ultimate strand stress, f_{ps}, was determined per equation 18-3 of the ACI code. The effective stress f_{se-180}, is computed from the PCI model. This suggests an upper bound value of development length for two reasons. First, the PCI model provides higher effective stresses than the ACI model. Second, the effective stresses at the time of testing are higher than at the end of service.

Table 16: Development Length Test Results

Specimen ID	f_{ps} (ksi)	f_{se180} (ksi)	L_d (in)	No. of Strands w/slip > 0.01 in	$L_{d\text{-test}}$ (in)	$\frac{M_{TEST}}{M_{ACI}}$
LW8000II-1A-72	266	172	76	N/A	N/A	1.27
LW8000II-1B-60	266	172	76	6/8	>60	1.11
LW8000II-2C-72	266	172	76	4/8	≅ 72	1.24
LW8000II-2D-96	266	172	76	None	<96	1.30
NW8000II-3A-72	266	175	75	None	<72	1.32
NW8000II-3B-96	266	175	75	None	<96	1.37

As shown in Table 16, all test specimens have exceeded the design strength predicted by the ACI equation, including lightweight specimen LW8000II-1B-60 and normal weight specimen NW8000II-3A-72. The loading reached during the testing of these two specimens was deemed unsafe for the load assembly, and therefore the beams were not tested to complete failure.

Accordingly, it could be argued that estimation of the "development length" as defined by the code to enable a beam to develop its "design strength" is conservative for both normal weight and lightweight prestressed concrete beams.

However, if development length is referred to as the shortest embedment length necessary for a beam to develop its "natural" flexural capacity as found from testing without general bond failure (<0.01 inch), then it is arguable that the code provisions for development length might not be conservative for high-performance lightweight beams. It can be seen from Table 16 that four of eight strands exhibited slip greater than 0.01 inch for a tested embedment length of 72 inches in specimen LW8000II-2C-72. The tested embedment length was approximately equal to the development length predicted by the ACI code at the time of testing.

As discussed previously, the ACI equation for development length was intended as a mean representation of data developed by Hanson and Kaar.[18] Data from testing in this research clearly show that beyond general bond failure, the mechanical bond mechanism between the strands and the concrete contributed towards additional flexural resistance in the HSLWC test beams.

Table 17 summarizes the loads at which general bond failure (defined as slip of 0.01 inch) occurred in specimens LW8000II-1B-60 and LW8000II-2C-72, and the failure loads predicted by the AASHTO/ACI design strength equations and the final test/failure loads. Note that at 280 kips, testing of specimen LW8000II-2C-72 was terminated for safety purposes as indicated previously.

Table 17: Comparison of Loads at Failure and Slip

Specimen ID	P_{SLIP} (k)	P_{ACI} (k)	P_{TEST} (k)	$\frac{P_{TEST}}{P_{SLIP}}$	$\frac{P_{TEST}}{P_{ACI}}$
LW8000II-1B-60	244	252	280	1.15	1.11
LW8000II-2C-72	230	218	272	1.18	1.25

As seen, the HPLWC test beams developed additional flexural resistance in excess of 15% beyond general bond failure.

The normal weight beams showed no slip in the tested embedment length of six feet and eight feet, respectively. Although testing of the six-foot embedment length in specimen NW8000II-3A-72 was terminated at 290 kips of load for safety purposes, the flexural moments reached were 32% greater than the design strength calculated from the ACI/AASHTO equations. As such it could be argued that the equation for the development length in ACI/AASHTO is more conservative for HPNWC members than for the HPLWC prestressed members.

Unfortunately because of the limited number of test samples available, an exact development length could not be established for the HPNWC prestressed beam to enable establishment of a factor of safety, vis-à-vis the code provisions. It is also necessary to note that a direct comparison between the HPNWC and HPLWC specimens could not be drawn because of the variation in the concrete strength of the specimens. The HPNWC beams were significantly higher in compressive strength than the lightweight counterparts.

In light of these findings, it is the authors' opinion that the development length equation in the ACI/AASHTO codes as a mean representation remains valid for prestressed HPLWC beams, even if with a low-to-no factor of safety, when compared to HPNWC prestressed members as could be qualitatively concluded here. At 72-inch embedment length, four of eight strands recorded a general bond failure, and yet the HPLWC beam (LW8000II-2C-72) developed a flexural capacity 24% higher than the code prediction. The ACI/AASHTO development length equation predicted a 75-inch development length for this test member (i.e., a factor of safety of approximately 1.0).

Further testing is necessary to develop a correlation in the factors of safety inherent in the current development length equation for high performance lightweight and normal weight prestressed concrete beams and to adjust the code accordingly for lightweight prestressed concrete members.

Until further test data are obtained, it is recommended that the development length equation in the code be modified by a factor of $1/\phi$, where ϕ is equal to 0.85 for lightweight prestressed concrete beams. This increases the factor of safety of the development length for lightweight beams by about 18 %, given the test results of this research investigation. The ϕ factor here is recommended for its ease of use. It is referenced in the AASHTO and ACI codes to modify shear strength and modulus of rupture for lightweight concrete beams.

Prestress Losses

As stated, the objective for monitoring the variation of strain in the prestressed Type IV girders was to assist in drawing qualitative deductions as to the rate and amount of prestress loss that occurred in the girders made with high performance lightweight concretes when compared to predictions derived from the ACI and PCI models. Correlation of the measured strains over time to strains calculated using the PCI and ACI models allows for a qualitative assessment of prestress losses in the girders.

Concrete strains at the level of the strands are a function of prestress, eccentricity of the strands, and the gravity loading at the critical section. Due to softness of the material (low modulus of elasticity) and its relatively lower developed strength at detensioning (4780 psi), the elastic shortening loss was relatively high, 20.5 ksi (i.e., 10 % of the initial jacking stress of 205 ksi). Due to this fact, it was recommended that a denser and stronger mixture be developed. The mixture used in the Chickahominy Bridge eventually incorporated normal weight coarse and fine aggregates to increase its unit weight and lightweight coarse aggregates of ½-inch maximum size to increase its strength.

The overall prestress losses estimated for the HPLWC Type IV girders over a nine-month period was 28.65 ksi using the ACI model and 16.20 ksi using the PCI model. Creep constituted 55% to 63% of the calculated losses, and shrinkage about 33% to 36% of the same as seen in Table 18.

Table 18: Prestress Loss Components in the Type IV Girders

MODEL	CREEP (ksi)	SHRINKAGE (ksi)	RELAXATION (ksi)	Total Loss (ksi)
ACI	17.99	9.46	1.20	28.65
PCI	8.99	5.81	1.40	16.20

Creep and shrinkage are the major components that contribute to the increase of compressive strains at the level of strands in prestressed beams. Creep is simply the property of concrete to continue to deform under sustained load, and shrinkage is the reduction in concrete volume due to loss of water by evaporation and continuing hydration.

As shown in Table 13, the actual compressive strains measured at the level of strands in the HPLWC Type IV girders are less than that estimated using either the PCI or ACI model.

It is conservative to use either of these models to estimate the prestress losses in HPLWC prestressed beams, assuming that the long-term trend is not unlike the short-term trend. The ACI model, which is the more conservative of the two and therefore forms a lower bound, predicts an effective prestress of about 143 ksi at the end of service (assumed as 10,000 days here) of the prestressed beams studied herein. That is to say, the final effective prestress would be about 53% of the ultimate strength of the strands. As such, it is safe to conclude that the designers of the Chickahominy Bridge could still use the ACI/AASHTO equations to determine the stress in the strands and subsequently the strength of the HPLWC prestressed bridge girders, since the retained prestress in the members can safely be assumed to be greater than 50% f_{pu} as stipulated in the ACI/AASHTO codes.

Given that, the reader is cautioned that thermal effects on the concrete were not accounted for and therefore constitutes a source of error in the test data. Moreover, the theoretical strains beyond the 28th day were calculated using the modulus of elasticity of concrete calculated using its 28th day strength per ACI Section 8.5.1. (i.e., the increase in strength of concrete and therefore its modulus of elasticity was not accounted for in calculating the increase in compressive strains due to shrinkage and creep over time). As such, these strains are understandably higher than if the increase of modulus of elasticity with time had been accounted for. However, this is counterbalanced by the fact that the modulus of elasticity calculated per Section 8.5.1 of the ACI code was about 14% higher than that recorded from testing done to measure the modulus of elasticity of the HPLWC mix on its 28th day.

The compressive strains at detensioning were recorded as 801 micro-strains, which is about 26% higher than the theoretically calculated strain of 634 micro-strains. Two main factors could be contributing to that. First, the actual modulus of elasticity of the concrete is lower than the theoretical one, and second, the initial prestress used in calculating the strains may be underestimated.

Overall, however, it is evident from the short-term trend observed that both the ACI and PCI models could be safely used to estimate the prestress losses in HPLWC prestressed girders, since both seem to be conservative with regard to actual losses observed during this research investigation.

CONCLUSIONS

- Composite girders made of HPLWC AASHTO girders and normal weight concrete (NWC) decks behave like their counterparts made with NWC AASHTO girders and NWC decks.

This is due to the fact that the neutral axis at failure invariably lies within the NWC deck. It is, therefore, safe to predict their nominal strength using the ACI/AASHTO equations for calculating stresses in the strands at nominal strength for strength calculations.

- Tests have shown that the modulus of rupture for HPLWC prestressed T-girders is about 60% to 85% of the ACI/AASHTO code stipulation of $\phi*7.5\sqrt{fc'}$, where ϕ is recommended to be 0.85 for lightweight concretes. This indicates that the allowable tensile stress at transfer of prestress force and at service load in lightweight, prestressed concrete members should be reduced to allow for the same safety factor against cracking as found in prestressed concrete members containing non-lightweight concrete.

- Prestress losses in HPLWC prestressed girders appear to be less than those predicted by the ACI and PCI models. Furthermore, the loss trend appears to validate the use of the ACI/AASHTO equations for predicting the stresses in the strands at nominal strength, which equations are predicated on the condition that the effective prestresses at the end of service should not be less than 50% of the strand strength.

RECOMMENDATIONS

1. It is recommended that all ACI/AASHTO code guidance for transfer length be raised to 60 d_b per AASHTO LRFD's stipulation and/or $(fsi.d_b)/3$ for lightweight, prestressed concrete bridge girders.

2. It is recommended that the equation for the development length of prestressing strands in prestressed concrete be modified by a factor of $1/\phi$ (where ϕ is equal to 0.85) when applied to prestressing strand in lightweight concrete. This should result in an 18% increase in the ACI/AASHTO code stipulation for development length.

3. Additional development length testing is required to validate these results for other test parameters. The parameters that need to be varied are differing AASHTO Type girders, prestressing forces, HPLWC strengths, strand manufacturers, and concrete density.

4. Additional testing is required to verify the findings of this research regarding the low modulus of ruptures that characterized the HPLWC prestressed Type II T-girders at cracking, but a reasonable and minimum reduction of ϕ is recommended.

5. Additional research is required to evaluate creep and shrinkage in HPLWC of differing strength and densities. This is necessary in order to better estimate long-term prestress losses in HPLWC prestressed girders.

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