FHWA RESEARCH PROGRAM ON LIGHTWEIGHT HIGH-PERFORMANCE CONCRETE – TRANSFER LENGTH

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ABSTRACT

FHWA's Structural Testing Laboratory recently completed the fabrication of 27 prestressed lightweight high-performance concrete (LWHPC) girders designed to investigate the use of this type of concrete in highway bridges. The girders used specified density concrete mixes with expanded shale or expanded slate coarse aggregate, granite coarse aggregate, and sand fine aggregate. The concrete mixes had 28-day compressive strengths that ranged between 7,400 psi and 10,500 psi. The research program has the purpose of (i) investigating the performance of LWHPC produced using aggregates representative of those available in North America, (ii) investigating the transfer length, development length, and shear strength of precast/prestressed LWHPC members, (iii) studying development and splice length of mild steel reinforcement used in LWHPC, and (iv) investigating prestress losses in LWHPC girders.

This paper describes the results of prestress transfer length measurements made on 18 AASHTO Type II and BT-54 girders using concrete surface strain method. An accurate estimation of the transfer length is necessary for the calculation of the concrete stresses at transfer and under service loads. The results have been compared to the AASHTO Specifications, ACI 318 Code, and several models available in the literature. The AASHTO Specifications and ACI 318 Code made conservative predictions of transfer length for all 18 girders.

Keywords: Lightweight concrete, High-performance concrete, Specified density concrete, Transfer length

INTRODUCTION

There are many advantages to using lightweight concrete, such as reduced transportation costs, longer spans, and/or smaller and potentially less expensive members. The current AASHTO LRFD Bridge Design Specifications[1] define lightweight concrete as having an equilibrium density less than or equal to 120 pcf. Normal weight concrete is defined as having an equilibrium density from 135-155 pcf. Concretes in the gap of densities between 120-135 pcf are commonly referred to as “specified density concrete” and are not directly addressed by the AASHTO specifications. Specified density concrete typically contains a mixture of normal weight and lightweight coarse aggregate and has benefits similar to lightweight concrete; however the AASHTO specifications do not take full advantage of the potential use of LWHPC.

There has been considerable research in recent years on the behavior of high-performance concrete (HPC) containing normal weight aggregate. These research efforts could expand the applicability of the AASHTO LRFD Bridge Design Specification for normal weight concrete to compressive strengths up to 18 ksi. However, there has been considerably less research on HPC containing lightweight aggregates, especially on structural members with compressive strengths in excess of 6 ksi. In addition, the limited research on lightweight HPC (LWHPC) has been focused on equilibrium densities less than 125 pcf. This leaves a gap in experimental data for specified density concretes.

This paper gives the preliminary results of transfer length measurements on 18 prestressed/prestressed bridge girders. These results are one portion of an ongoing research program conducted by the Federal Highway Administration (FHWA) at the Turner-Fairbank Highway Research Center (TFHRC). The purpose of the research program is to investigate the performance of LWHPC with concrete compressive strengths in the range of 6 to 10 ksi and equilibrium densities between 125 pcf to 135 pcf. The research program will use LWHPC with three different lightweight aggregates that are intended to be representative of those available in North America. The program will use the tests from 27 precast/prestressed LWHPC girders to investigate transfer length and development length of prestressing strand, the time-dependent prestress losses, and shear strength of LWHPC. The development and splice length of mild steel reinforcement used in girders and decks made with LWHPC will be investigated using 40 reinforced concrete (RC) beams. While the FHWA program is focused on structural behavior, it will also have a material characterization component that will include tests to determine compressive strength, elastic modulus, splitting tensile strength, creep, shrinkage, coefficient of thermal expansion, permeability, resistance to scaling, and resistance to rapid freezing and thawing of the concrete mixes used in the structural testing program. The result of the research program will be to recommend changes to the AASHTO LRFD Bridge Design Specifications relevant to LWHPC.

TRANSFER LENGTH OF PRESTRESSING STRANDS – THEORY

The transfer length of prestressing strands is defined as the embedment length required to transfer the effective prestressing force in the strands to the surrounding concrete. An accurate estimation of the transfer length is important for several reasons: calculation of the concrete stresses at transfer and under service loads, design of anchorage zone reinforcement for strut-and-tie models, and design of shear reinforcement which requires knowledge of the level of precompression in the concrete[2].

The two most significant mechanisms that contribute to prestress transfer bond are friction and mechanical resistance[3]. Radial compressive stress, commonly attributed to the Hoyer Effect, is required to develop frictional bond stresses. In the short region of the transfer length where the concrete remains elastic, the radial compressive stress depends directly on the elastic modulus of the concrete. In the inelastic region, the radial compressive stress depends on both the elastic modulus and the tensile capacity of concrete.

Both the elastic modulus and tensile capacity of LWHPC are less than normal weight concrete of the same compressive strength. Previous test specimens using LWHPC have had varied results as to the whether the AASHTO Specifications gave a conservative prediction of the transfer length[4,6].

There are many variables that affect transfer length. Transfer length has been shown by previous research to be proportional to strand diameter[3,7-10]. Transfer length is also strongly influence by the stress level in the strand[7,9]. Other variables that can affect the transfer length include surface condition of the steel (clean, oiled, rusted), time-dependent effects (concrete creep and shrinkage, strand relaxation), method of release (flame cut, gradual release), and concrete properties (compressive strength, tensile strength, and modulus of elasticity) [2,9,11,12]. In many previous investigations the transfer length was measured at release of the prestress. Previous research has shown that the transfer length does not change significantly after release[11].

Research on small specimens with only a few strands has shown that strands that were flame cut had longer transfer lengths than strands that were released gradually[9]. Research has shown that flame cutting the strands of large AASHTO-type girders with multiple strands causes less of an increase in transfer length than flame cutting the strands of small, single-strand members[12]. This is due to the greater mass of concrete being more capable of distributing the energy and stress induced by flame cutting.

TRANSFER LENGTH OF PRESTRESSING STRANDS – PREDICTION

There are many different methods to predict the transfer length of a prestressing strand. Several common methods are described in this paper.

AASHTO SPECIFICATIONS

The Standard Specifications for Highway Bridges, 16th Edition[13] (AASHTO 16th) recommends the expression in Eq. (1) for transfer length. Eq. (2) is recommended by the AASHTO LRFD Bridge Design Specifications, 4th Edition[1] (AASHTO LRFD), and will give a calculated transfer length that is 20% longer than the one calculated by Eq. (1). In both specifications, the recommendation is located in the shear provisions and not in the provisions on development of reinforcement.

$$l_t = 50 d_b \tag{1}$$

$$l_t = 60 d_b \tag{2}$$

ACI 318-08

The expression for transfer length in the ACI 318-08 Building Code[14] is given by Eq. (3). This expression was derived by Mattock[15] who assumed a uniform bond stress of 400 psi based on the research of Hanson and Kaar[7]. Eq. (3) was developed for Grade 250 prestressing strands (250 ksi ultimate strength). Assuming a 150 ksi effective stress (f_{se}), then Eq. (3) simplifies to the expression in Eq. (1).

Since the development of Eq. (1), construction practice has changed and Grade 270 strands (270 ksi ultimate strength) are currently used. If a 180 ksi effective stress is assumed for the Grade 270 strands, then this represents a 20% increase in the effective stress over the stress assumed for the Grade 250 strand. Assuming the same uniform bond stress of 400 psi, Eq. (2) incorporates the 20% increase in effective stress over Eq. (1).

$$l_t = \frac{f_{se} d_b}{3} \tag{3}$$

MITCHELL ET AL.

Equation (4) was the result of research by Mitchell, Cook, Khan and Tham[10] on 22 precast, pretensioned normal weight concrete beams to investigate the affect of the compressive strength and strand diameter on transfer and development length. The beams had a small cross section with a single strand and the prestress was released gradually. The compressive strength at release varied from 3000 psi to 7310 psi, and the nominal strand diameters varied from 3/8 in. to 0.62 in.

$$l_t = 0.33 f_{si} d_b \sqrt{\frac{3}{f'_{ci}}} \tag{4}$$

ZIA AND MOSTAFA

The empirical expression for transfer length developed by Zia and Mostafa[9] is given by Eq. (5) and was based on data available in the literature. The data was from normal weight concrete specimens with nominal strand diameters that ranged from 1/4 in. to 3/4 in. The investigators stated that their expression was applicable to concrete strengths ranging from 2000 to 8000 psi.

$$l_t = 1.5 \frac{f_{si} d_b}{f'_{ci}} - 4.6 \quad (5)$$

BUCKNER

Bucker performed a review of the literature related to transfer and development length and he analyzed the data from several studies that were published in the early 1990s[8]. As part of his analysis, he developed Eq. (6) based on the data from normal weight specimens that had only one ½ in. nominal diameter fully bonded strand. Buckner's study indicated an influence of the modulus of elasticity of concrete (E_{ci}) on transfer length.

$$l_t = \frac{1250 f_{si} d_b}{E_{ci}} \quad (6)$$

THATCHER ET AL.

The study by Thatcher, Heffington, Lolozs, Sylva, Breen, and Burns[16] also indicated an influence of the modulus of elasticity on transfer length. They studied the transfer length of AASHTO Type II girders made with lightweight concrete. Their expression for transfer length is given by Eq. (7) and is 72% of the value calculated by Eq. (6).

$$l_t = \frac{900 f_{si} d_b}{E_{ci}} \quad (7)$$

EXPERIMENTAL PROGRAM

An important aspect of this study is using LWHPC produced from aggregates representative of those available in North America. This is important because the mechanical properties of the lightweight aggregate are mostly responsible for the differences between concrete made from normal weight and lightweight aggregate. The Expanded Shale, Clay, and Slate Institute (ESCSI) assisted FHWA in finding three girder mix designs that are already in use at precasting plants in the United States. The three mix designs used are shown in
Table 1 and include the following lightweight aggregates: Haydite, an expanded shale produced in Ohio; Utelite, an expanded shale produced in Utah; and Stalite, an expanded slate produced in North Carolina. The mix designs use a combination of lightweight coarse

aggregate and normal weight coarse aggregate to produce their target equilibrium densities. None of the mix designs use silica fume.

Table 1: Lightweight Concrete Mix Designs for Girders (Quantities per Cubic Yard)

Concrete Mix		Haydite Mix	Utelite Mix	Stalite Mix
Specified 28-Day Strength	(psi)	6,000	7,000	10,000
Specified Release Strength	(psi)	3,500	4,200	7,500
Lightweight Course Aggregate †	(lb)	800	740	880
Normal Weight Course Aggregate ‡	(lb)	520	385	250
Normal Weight Sand	(lb)	1,185	1,267	1,221
Class F Fly Ash	(lb)	-	600	-
Type III Portland Cement	(lb)	750	150	800
Water	(lb)	267	259	250
Water Reducer #	(oz)	19	19	19
Air Entrainer #	(oz)	2	2	2
HR Water Reducer #	(oz)	34	34	34
Water / Cementitious Materials Ratio		0.36	0.34	0.31
% Air		6	6	2
Fresh Concrete Unit Weight	(pcf)	130	126	126

† Maximum aggregate size: 3/4 in. Haydite, 1/2 in. Utelite and Stalite
‡ Nova Scotia granite
# Sika admixtures: Plastiment (water reducer), AEA 14 (air entrainer), HRWR/R V2100 (HR water reducer)

General details of the nine different prestressed girder designs are listed in Table 2. Girder Designs 1-4 were AASHTO Type II cross sections designed for the purpose of investigating development length (L_d). Girder Design 1 is the control girder for development length. The number of strands was increased to produce a higher total prestressing force in Girder Design 2. The amount of shear reinforcement was reduced in Girder Design 3. Girder Design 4 utilized 0.6 in. diameter strands. The amount of shear reinforcement was designed to give a constant ratio of shear capacity to moment capacity at sections located the theoretical development length, where the development length is calculated using AASHTO LRFD[1].

Girder Designs 5-7 were also AASHTO Type II cross sections and Girder Designs 8 and 9 were AASHTO-PCI BT-54 cross sections. The girders were designed for the purpose of investigating shear strength. The dead ends of Girder Designs 5 and 8 had the AASHTO Specifications' required minimum amount of shear reinforcement at the maximum spacing, and the live end had slightly more than the minimum amount. Girder Design 6 had draped strands and moderate amounts of shear reinforcement. Girder Design 7 and 9 were designed to have approximately 80% of the maximum amount of shear reinforcement implied by Article 5.8.3.3 (Equation 2) in AASHTO LRFD. Fig. 1a shows a typical cross section for Girder Design 7, and Fig. 1b shows a typical cross section for Girder Design 9.

Each of the nine girder designs was fabricated using the three mix designs given in Table 1, for a total of 27 LWHPC girders. The girders were manufactured by the Standard Concrete Products (SCP) plant in Mobile, Alabama. SCP is a company that specializes in precast and prestressed fabrication. After delivery to TFHRC, normal weight concrete decks will be cast on the girders.

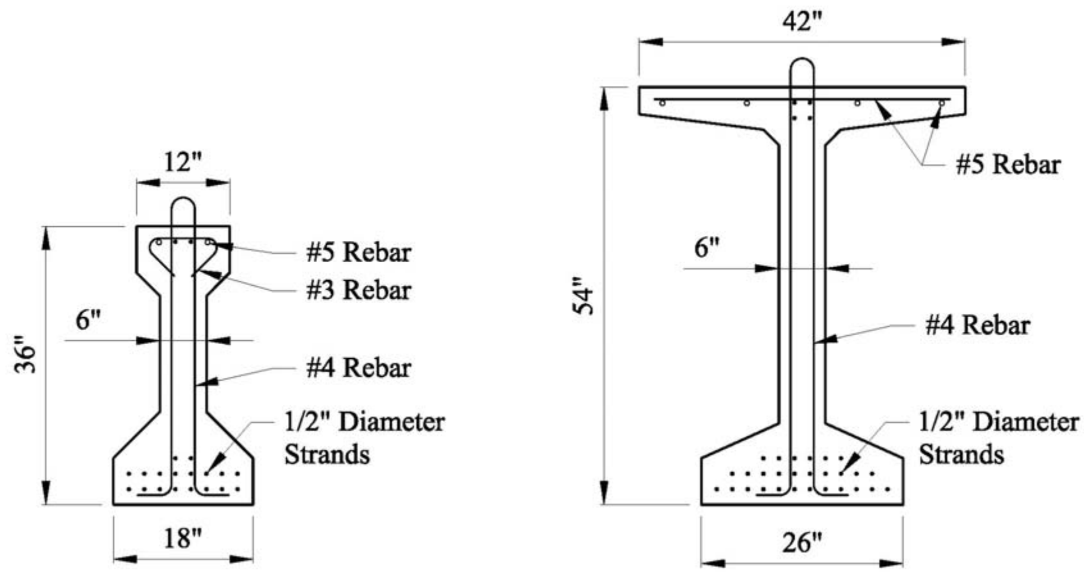
Prestress transfer data was measured on a total of 18 members from Girder Designs 1-4, 8, and 9. Analysis of the transfer data and comparisons of the measured transfer length to the transfer lengths predicted by the AASHTO Specifications, ACI 318-08 Code, and three other models available in the literature are given later in this paper. The remaining girders will be used to investigate time-dependant prestress losses and then loaded to failure to investigate development length (Girder Designs 1-4) and shear behavior (Girder Designs 5-9).

Table 2: Girder Design Summary

Girder Design	Description	Strand Size (in.)	No. of Strands		Length (feet)	Stirrups †
			Top	Bottom		
AASHTO Type II						
1	L_d – Control	0.5	2	10	45	Double #5 at 8”
2	L_d – High Prestress Force	0.5	4	18	45	Double #5 at 5”
3	L_d – Low Shear Reinforcement	0.5	2	10	45	Double #5 at 10”
4	L_d – 0.6” Diameter Strand	0.6	2	8	45	Double #5 at 9”
5	Shear – Minimum Shear Reinforcement	0.5	2	10	42	#3 at 22”, #3 at 15”
6	Shear – Draped Strands	0.5	6	10	42	#4 at 15”, #4 at 12”
7	Shear – High Shear Reinforcement	0.5	4	18	42	#4 at 8”, #4 at 12”
AASHTO-PCI BT-54						
8	Shear – Minimum Shear Reinforcement	0.5	2	16	50	#3 at 22”, #3 at 14”
9	Shear – High Shear Reinforcement	0.5	4	30	50	#4 at 8”, #4 at 10”

† Stirrups and spacing at each end (dead end, live end)

Girder fabrication occurred over a seven week period between May 8th, 2008 and June 23rd, 2008. SCP fabricated the girders on two different casting beds, one bed for the AASHTO Type II girders, and other bed for the AASHTO-PCI BT-54 girders. SCP organized the girders into casting production runs as shown in Table 3. Girders with the same strand pattern were cast together. Two sets of Girder Designs had the same strand pattern: 1, 3, and 5 had 12 strands; and 2 and 7 had 22 strands. Run 2 and Run 3 each had six girders; however SCP only had enough length of AASHTO Type II side forms to cast three girders at one time. The girders were located near the “dead end” (non-jacking end) of the 400-500 ft long prestressing beds. This meant that there was several hundred feet of free strand between the last girder and the jacking or “live end” of the prestressing bed.



(a) Girder Design 7 (AASHTO Type II)

(b) Girder Design 9 (AASHTO-PCI BT-54)

Fig. 1 Typical Cross Sections: Girder Design 7 and 9

Table 3: Girder Production Runs

Run	Cast Date	Release Date	Release Method	Girders Design and LW Aggregate †
AASHTO Type II				
1	5/21/2008	5/22/2008	Flame Cut	5, 1, 3 all Stalite
2 - Cast 1	5/30/2008	6/4/2008	Detension	5, 1, 3 all Utelite
2 - Cast 2	6/3/2008	6/4/2008	Detension	5, 1, 3 all Haydite
3 - Cast 1	6/9/2008	6/11/2008	Detension	7, 2 (Stalite); 2 (Utelite)
3 - Cast 2	6/10/2008	6/11/2008	Detension	7 (Utelite); 2, 7 (Haydite)
4	6/14/2008	6/16/2008	Detension	4 (Utelite, Haydite, Stalite)
5	6/20/2008	6/23/2008	Detension	6 (Stalite, Utelite, Haydite)
AASHTO-PCI BT-54				
6	5/14/2008	5/17/2008	Flame Cut	8 (Stalite, Haydite, Utelite)
7	5/29/2008	5/31/2008	Flame Cut	9 (Stalite, Haydite, Utelite)

† In casting order, beginning with "Dead End"

Sets of 4 x 8 in. and 6 x 12 in. cylinders for material property testing were made for each different concrete mix in a casting. For example, Run 1 had one set made because all three girders were cast using the Stalite Mix, and Run 4 had three sets made because concrete mixes using all three lightweight aggregate that were part of the study were cast. Compressive strength, spitting tensile strength, and modulus of elasticity was tested using 4 x 8 in. cylinders at release of prestressing and "28 days" after casting. Due to time constraints at SCP and truck availability, the 28-day tests were made between 27 and 32 days after casting. The results of material tests are shown in Table 4 and represent the average of three cylinders for compression and splitting tension tests, one cylinder for modulus of elasticity, and four cylinders for nominal unit weight.

Table 4: Preliminary Material Test Data for Girder Mixes

LW Aggregate	Girder Design †	Release		28-Day		
		Compression Failure Stress (psi)	Elastic Modulus (ksi)	Compression Failure Stress (psi)	Splitting Tension Failure Stress (psi)	Nominal Unit Weight (pcf)
Stalite	4	6804	4069	9270	673	125
	5 (1, 3)	6355	3631	9480	659	125
	6	7732	3743	9710	732	125
	7 (2)	7119	3666	9640	674	123
	8	8199	3950	10510	716	127
	9	7716	3562	9630	622	126
Haydite	3 (1, 5)	6208	3551	8820	680	131
	4	7315	3783	9210	757	131
	6	7299	4017	9280	685	134
	7 (2)	6657	3728	9830	739	133
	8	7501	1538	10200	784	133
	9	7436	4109	10080	681	134
Utelite	2 (7)	6221	3789	9640	764	133
	4	5865	3412	8340	668	130
	5 (1, 3)	7110	3563	8730	640	131
	6	5156	3197	7370	608	129
	8	6077	3566	9100	669	132
	9	5802	3477	8400	744	129
Average	Stalite	7321	3770	9707	680	125
	Haydite	7069	3454	9570	721	133
	Utelite	6038	3500	8597	682	131

† Girder design number in parentheses were cast at the same time

In Runs 1, 6, and 7 the prestressing was released by "simultaneously" flame cutting the strands. This was accomplished by workers using acetylene torches to cut the strands between the girders and at each end of a line of girders at the same time. At a foreman's signal, the workers began cutting the same strand at each location. Each strand gave an audible "bang" after it was cut and all the cuts along a strand typically varied by several seconds. When only two strands remained, the girders would separate at the location where the first cut was made. This was due to the remaining strand being unable to carry the force of the two uncut strands remain at all the other girder ends. This caused the girders to slide away from the location without any connected strands. Release of prestress using the flame cutting technique was typically completed in 30 minutes.

Girder Runs 2 through 5 were detensioned in accordance with safety considerations. The strands were detensioned one at a time at the jacking end of the prestressing bed. Release of prestress using the detensioning technique was typically completed in 2-3 hours.

TRANSFER LENGTH – MEASURMENT, ANALYSIS, AND DISCUSSION

The transfer length measurements were made on a total of 18 girders: Girders 1 through 4 (AASHTO Type II development length girders) and on Girders 8 and 9 (AASHTO BT-54 shear girders). Two different types of transfer length measurement were made on the girders: concrete surface strain (CSS) and strand draw-in measurements. The method used to make the CSS measurement and the analysis of the resulting strain profiles is presented in this paper.

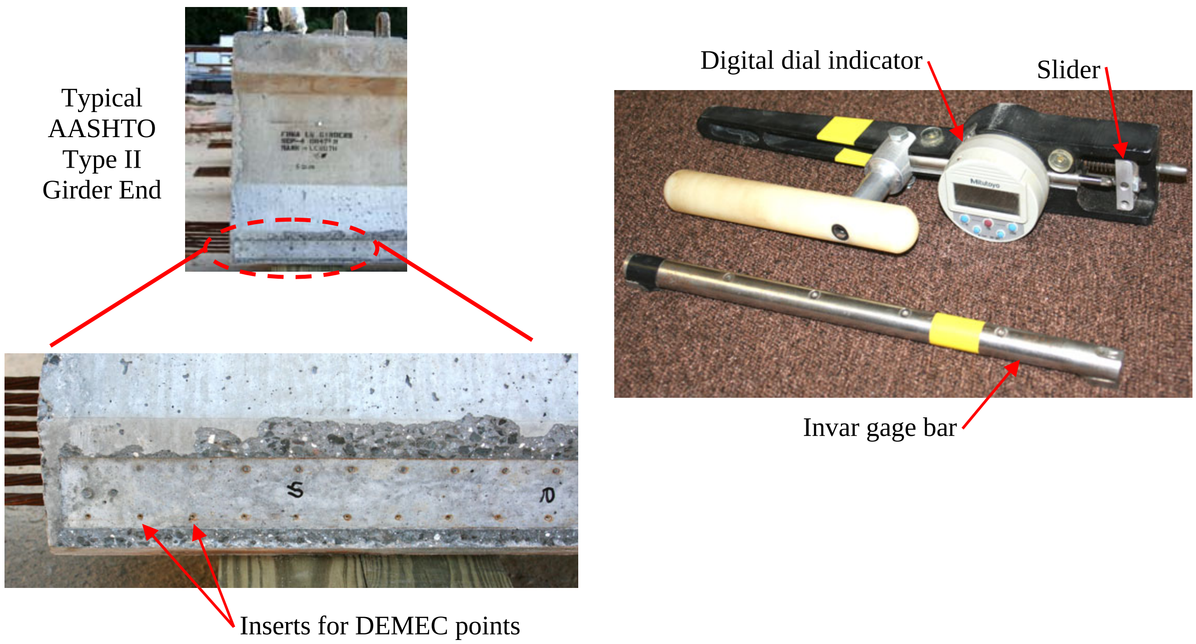
MEASUREMENTS

The transfer length was "measured" using the CSS method. This method uses a Detachable Mechanical Strain Gage (DEMEC) to read the distance between two target points (DEMEC points) in the concrete. The average strain at the surface of the concrete girders is calculated by taking the difference between readings made before and after the release of the prestressing. The CSS gives a reasonable estimate of the strain in the prestressing strand due to strain compatibility. Typically the strain measurements start near zero at the end of the girder and increase approximately linearly until they reach a constant value. A plot of the CSS with respect to the distance from the girder end is the strain profile. An ideal strain profile shows a plateau beginning at the theoretical transfer length.

The DEMEC points were brass inserts spaced at 2 in. that were screwed to a 1/4 in. thick steel strip that was bolted to the inside of the side forms. The points were located at the centroid of the bottom layer of prestressing strand. After the concrete was cast and set, the side forms and steel strips were removed to expose the brass inserts cast into the side of the girder. The DEMEC points on an AASHTO Type II girder are shown in Fig. 2a.

The DEMEC instrument is shown in Fig. 2b and consisted of a small hand-held frame that holds a fixed conical pin at one end and second conical pin on a slider at the other end. The slider was oriented to allow the second pin to travel along a linear path from the fixed pin. A spring pushed the slider away from the fixed pin. A Mitutoyo digital dial indicator model IDA-112ME with a reading to the nearest 0.0001 in. measured the movement of the slider. DEMEC reading are known to be very sensitive to the technique used by the operator to make the reading[6,12]. For this reason, measurements were made by two different operators, and the difference between two operator readings was limited to 0.0010 in, although typical readings had a difference that was less than 0.0005 in. Prior to the first reading and after every ten readings, the DEMEC instrument was calibrated using an 8 in. gage bar made from Invar (Fig. 2b). Invar is a nickel-iron alloy with low coefficient of thermal expansion.

Multiple measurements were made along each group of points. Each measurement spanned four DEMEC points and represents the average strain across the 8 in. gage length. The initial measurements were made for all the girders prior to release of the prestressing. It took approximately 30 minutes for two individuals to make measurements at all four ends of a single girder. The final measurements were started immediately following release of the strands.



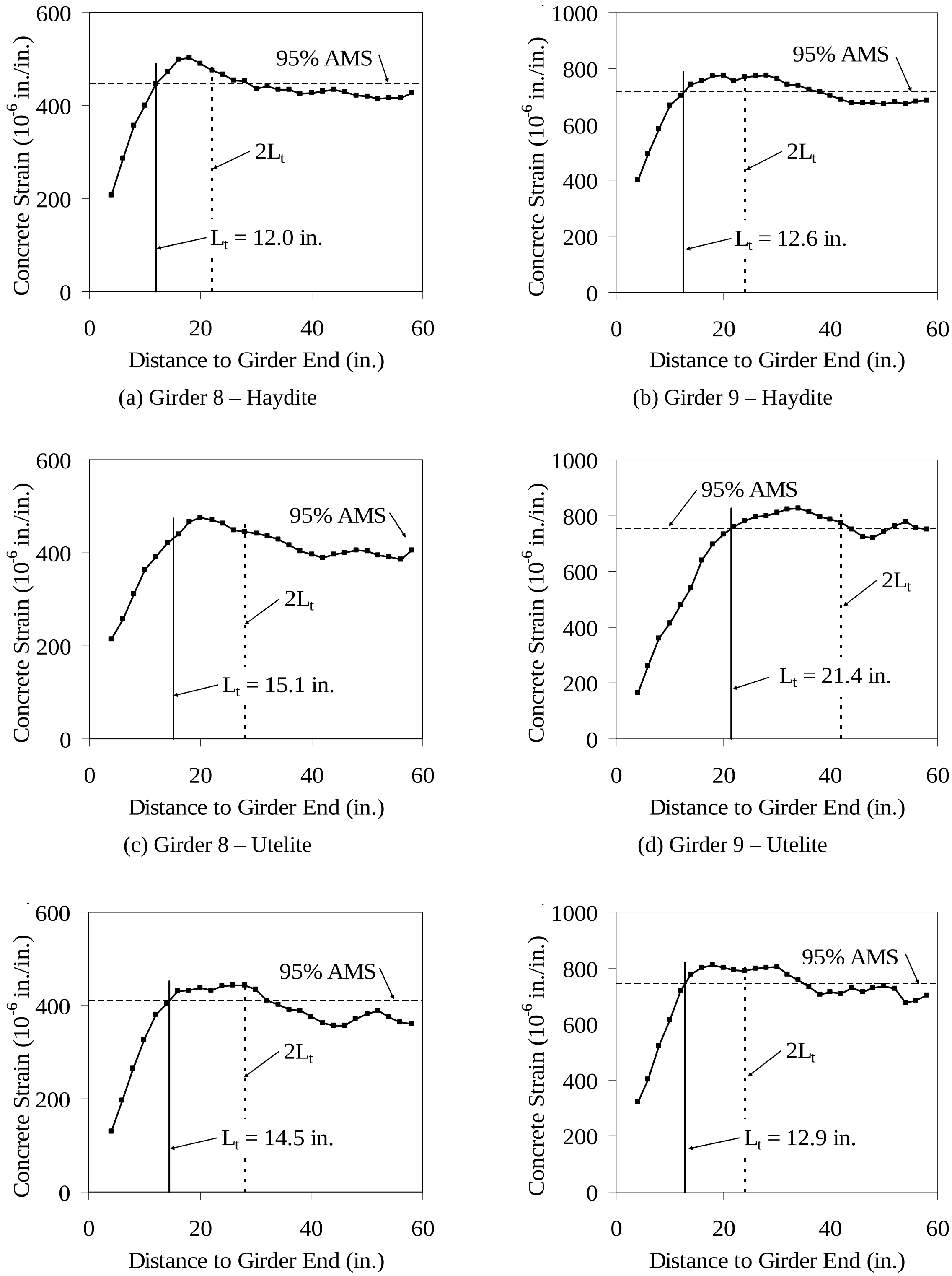
(a) DEMEC Points on Girder End (b) DEMEC and Gage Bar

Fig. 2 DEMEC: Girder Points, Instrument, and Gage Bar

ANALYSIS

The measurements from the two operators taking the readings were averaged together. The strain was calculated by taking the difference between the initial and final measurements, and then dividing by the gage length adjusted for the initial measurement (8 in. plus the initial measurement). The average data for each end of a girder (dead or live) was calculated by averaging the strain data for each side. The average data for each girder was calculated by averaging the strain data for all four sides (two at each end). The data (individual end, averaged for end, averaged for girder) was "smoothed" by averaging the strain for three consecutive points and applying their average to the middle point.

The transfer length was calculated using the 95% Average Maximum Strain Method (95% AMS). This method was developed Russell[12] and has also been used by researchers in several recent investigations[5,6,8] to evaluate the CSS data for transfer length. The 95% AMS method involves calculating the average of all the strains on the strain plateau (the AMS), constructing a line on the strain profile at the strain equal to 95% of the AMS, then determining the transfer length at the intersection of the 95% AMS line and the smoothed strain profile. The strain profiles of the six AASHTO-PCI BT-54 girders (Girder Designs 8 and 9) are shown in Fig. 3. The 95% AMS line and the calculated transfer length are shown in the figure.



(a) Girder 8 – Haydite (b) Girder 9 – Haydite

(c) Girder 8 – Utelite (d) Girder 9 – Utelite

(e) Girder 8 – Stalite (f) Girder 9 – Stalite

Fig. 3 Strain Profiles (Four End Average) and Transfer Length for Girders 8 and 9

DISCUSSION

The strain profiles shown in Fig. 3 exhibit a linear increase in strain to a peak (or narrow plateau), followed by a slow decrease in strain. The decrease in strain is most likely due to the self weight of the girder. A calculation of the change in strain from 18 to 60 in. from the girder end is approximately 150×10^{-6} in./in. (flexural tensile strain due to self weight) for a BT-54 girder with an elastic modulus of 4×10^{6} psi. This is of similar magnitude to the drop in compressive strain shown in the strain profiles of Fig. 3. Research on transfer length using AASHTO girders did not report compensating for self weight[5,12,16]. Buckner used the strain profile data from previous tests on AASHTO-type girders[12,17] to show that compensating for self weight only slightly increases the transfer length (approximately 5%).

The preliminary strain profiles presented in this paper were not compensated for self weight to be consistent with several recent studies[5,12,16]. However, the decreasing trend of the strains after peak does affect the calculated AMS. An AMS that included all 60 in. of measured strains would be much lower that an AMS that included only the strains near the peak, and would result in a much smaller transfer length using the 95% AMS method. In this study, the length of the plateau was taken as equal to the calculated transfer length. As a result only the data over a region of $2l_t$ was used. This method consistently gave transfer lengths that were more conservative (longer) than when considering the data over the full length of 60 in. Lines at $2l_t$ are shown on the strain profiles of Fig. 3.

Table 5 gives the transfer lengths averaged for each lightweight aggregate. The transfer length for each girder was calculated using the 95% AMS method on the average strain profile from all four sides of a girder. The average transfer length for all of the AASHTO Type II Girders with 1/2 in. diameter strand (Girder Designs 1, 2, and 3) was 10.6 in. (column 3) as compared to 18.6 in. (column 2) for the girders with 0.6 in. diameter strand (Girder Design 4). As expected, this shows a clear increase in transfer length with an increase in strand diameter. For girders with 1/2 in. diameter strand, there was also an increase in the transfer length from the AASHTO Type II (36” depth) to the AASHTO-PCI BT-54 (54” depth) (Girder Designs 8 and 9). As shown in Table 3, Girder Designs 8 and 9 were flame cut to release the prestressing. Almost all the AASHTO Type II girders were detensioned, and previous research has shown that flame cutting tends to produce slightly longer measured transfer lengths[12].

TRANSFER LENGTH – PREDICTIONS AND COMPARISONS

The measured transfer lengths for 18 girders that are part of this study are shown in Table 6. The transfer length was calculated using the 95% AMS method on the average data from four sides. The table also shows the transfer length predicted by Eqs. (1) through (5) and (7). The concrete material properties (E_{ci} and f'_{ci}) used in the predictions are based on cylinders tested on the day of detensioning. The strand stresses (f_{se} and f_{si}) were calculated using the AASHTO LRFD Specifications[1].

Table 5: Preliminary Average Measured Transfer Length by LW Aggregate

LW Aggregate	Average Measured Transfer Length †		
	AASHTO Type II, 0.6" strand (Girder Design 4) (in.)	AASHTO Type II, 0.5" strand (Girder Designs 1, 2, 3) (in.)	AASHTO-PCI BT-54, 0.5" strand (Girder Designs 8, 9) (in.)
Stalite	21.2	11.3	13.7
Hadite	15.3	10.3	12.3
Utelite	19.3	10.3	18.2
Average	18.6	10.6	14.7

† Calculated using 95% AMS method on average data from four sides (two at each end)

The expressions in the AASHTO Specifications[1,13] and ACI 318 Code[14] gave conservative predictions for the transfer length of all the specimens. The predictions made by the Mitchell et al.[10] and the Thatcher et al.[16] expressions were also conservative for all of the specimens. The transfer length predicted by the Zia and Mostafa[9] expression was also conservative for 15 of the 18 girders.

Table 6: Preliminary Measured and Predicted Transfer Length

LW Aggregate	Girder Design	Measured Transfer Length † (in.)	Predictions					
			AASHTO (16th) (in.)	AASHTO (LRFD) (in.)	ACI 318-08 (in.)	Mitchell et al. (in.)	Zia and Mostafa (in.)	Thatcher et al. ‡ (in.)
Stalite	1	9.4	25.0	30.0	29.2	21.8	18.1	23.9
	2	8.8	25.0	30.0	27.4	19.9	15.0	22.8
	3	15.6	25.0	30.0	29.2	21.8	18.1	23.9
	4	21.2	30.0	36.0	35.0	25.3	20.9	25.5
	8	14.5	25.0	30.0	30.8	19.6	13.4	22.4
	9	12.9	25.0	30.0	28.3	19.2	13.6	23.6
Hadite	1	11.0	25.0	30.0	29.1	22.1	18.6	24.4
	2	8.5	25.0	30.0	27.3	20.6	16.3	22.4
	3	11.5	25.0	30.0	29.1	22.1	18.6	24.4
	4	15.3	30.0	36.0	35.0	24.3	19.0	27.4
	8	12.0	25.0	30.0	29.3	19.7	14.3	55.2
	9	12.6	25.0	30.0	28.6	19.8	14.5	20.7
Utelite	1	7.9	25.0	30.0	29.4	20.6	15.7	24.3
	2	11.1	25.0	30.0	27.1	21.3	17.8	22.1
	3	11.9	25.0	30.0	29.4	20.6	15.7	24.3
	4	19.3	30.0	36.0	34.2	27.0	24.7	30.2
	8	15.1	25.0	30.0	30.2	22.7	19.6	24.8
	9	21.4	25.0	30.0	27.6	22.1	19.5	24.2

† Calculated using 95% AMS method on average data from four sides (two at each end)
‡ Thatcher et al.[16] prediction is 72% of Bucker[8] prediction

CONCLUDING REMARKS

CSS measurements were made on 18 HPLWC girders that are a part of this FHWA study. The specified density girder mixes used in the girders contained Utelite (an expanded shale), Haydite (an expanded shale), or Stalite (an expanded slate). The mixes also contained granite coarse aggregate to achieve the specified equilibrium density and sand. Twelve of the girders were AASHTO Type II and six were AASHTO-PCI BT-54.

The current provisions in the AASHTO Specifications and ACI 318 Code made conservative predictions of transfer length for all 18 girders.

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NOTATION

d_b	=	nominal reinforcing bar diameter (in.)
E_{ci}	=	modulus of elasticity at release of prestress (psi)
f'_{ci}	=	cylinder compressive strength at release of prestress (psi)
f_{se}	=	effective stress in the prestressing strands after long-term losses (elastic shortening at release, long-term shrinkage and creep of concrete, and relaxation of prestressing strand) (psi)
f_{si}	=	effective stress in the prestressing strands after release of prestress losses (elastic shortening at release) (psi)
l_t	=	transfer length (in.)

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