**Transfer Length in Bulb-Tee Girders Constructed with**
**Self-Consolidating Concrete**

by

Emily Louise Dunham

A thesis submitted to the Graduate Faculty of
Auburn University
in partial fulfillment of the
requirements for the Degree of
Master of Science

Auburn, Alabama
August 6, 2011

Keywords: anchorage, bond, debonding, prestressed concrete, prestressing steel



Approved by

Robert W. Barnes, Chair, James J. Mallett Associate Professor of Civil Engineering
Anton K. Schindler, Associate Professor of Civil Engineering
James S. Davidson, Associate Professor of Civil Engineering

Abstract

This thesis is focused on the transfer bond behavior of self-consolidating concrete (SCC) relative to conventionally vibrated concrete (CVC) in full-scale, plant-cast, prestressed concrete bulb-tee girders of implementation in Alabama bridges. The study involved twelve girders: six BT-54 girders—three SCC and three CVC—and six BT-72 girders—three SCC and three CVC. All of the concrete used in the study was considered high-strength with values of approximately 8,300 psi at the time of prestress transfer. Unlike the previous phases, for the first time, this study also considered partially debonded strands and their performance in SCC. Two different standard strand diameters were investigated.

It was found that the results from the girders cast with SCC produced longer transfer lengths than those cast with CVC. The larger cross-sections and less violent methods of prestress release resulted in shorter transfer lengths. Transfer lengths grew over time.

Transfer length expressions given by both ACI and AASHTO standards greatly overestimated the transfer lengths in comparison to the measured values. In general the predicted values were roughly twice those measured in the study primarily because of the high-strength concretes used. In addition, the expressions did not take into account several of the parameters that can have a significant effect on the transfer length, particularly concrete strength. Consequently, an expression proposed by Levy (2007)

produced the best overall representation of the trends seen in these girders and in prior related studies.

Acknowledgements

I would like to thank Dr. Barnes for so many of his hours used in teaching, guiding, and sharing of his wisdom. Not only did it have a tremendous effect on quality of this thesis, but he has also taught me invaluable things about being a structural engineer. I would also like to thank Graduate Research Assistants Tom Hadzor, Brandon Johnson, Brian Rhett, Sam Keske, Wes Bullock, and Morgan Ellis for the time and portions of their lives they dedicated in helping to produce the test specimens, as well as the reading and recording of countless data points. Not only did their help make this project feasible, but it became an adventure.

This project would also have been impossible without the generous funding from the Alabama Department of Transportation and the help from the men at the Hanson Prestress Plant in Pelham, Alabama. Their patience to allow us to interrupt their work routines in order to teach us the fabrication process will be valuable for years to come.

Many thanks are also due to my wonderful family: Mom and Dad in Texas, Christian, and my Auburn family as they encouraged and supported me throughout this process. Finally, may all the praise and glory go to the Lord who makes all things possible and brought me to Auburn for such a time as this.

Table of Contents

List of Tables

List of Figures

## List of Abbreviations

| | |
|---|---|
| AASHTO | American Association of State Highway and Transportation Officials |
| ACI | American Concrete Institute |
| ALDOT | Alabama Department of Transportation |
| AMS | Average maximum strain |
| BT-54 | Bulb-tee girder 54 in. in height |
| BT-72 | Bulb-tee girder 72 in. in height |
| DEMEC | Demountable, mechanical (points and gauges) |
| GGBF | Ground-granulated blast-furnace (slag) |
| HSC | High-strength concrete |
| HRWR | High-range water reducing (admixture) |
| NASP | North American Strand Producers |
| PCA | Portland Cement Association |
| SCC | Self-consolidating concrete |
| SCM | Supplementary cementing material |
| VDOT | Virginia Department of Transportation |
| VMA | Viscosity-modifying admixture |
| VSI | Visual stability index |

# Chapter 1 Introduction

## 1.1 Background

The precast/prestressed concrete industry is one that has grown rapidly in the past several decades, from playing its first major role in rebuilding after World War II to being one of the leading participants in the construction industry today. This type of construction has show great durability, along with fairly easy construction, reduced construction time, and cost efficiency as a result of the repeatability that is the key to the industry's nature. Any number of members and shapes can be reliably cast in large quantities that can then be used on the construction site. One of the most common shapes are girders, typically used in bridge design. Figure 1-1 shows a typical bulb-tee bridge girder being moved from a prestressing bed to the storage yard. As research continues to find new ways of improving this process, the industry becomes even more advanced. One new development in the precast industry is the use of self-consolidating concrete (SCC).


**Figure 1-1: Precast/Prestressed Concrete Bulb-Tee Bridge Girder**

Originating in Japan in the 1980s, SCC was created to be more workable and more convenient when placing concrete while providing a more durable concrete (Okamura and Ouchi 1999). Due to its fluid nature and the fact that it can deform under its own weight when wet, the concrete can flow into small areas and through reinforcement without the aid of vibration or other human interference. This not only leads to easier placement of the concrete, but it reduces the amount of labor involved, as well as increasing the level of safety for the workers. As a result, SCC is very attractive to the precast/prestressed concrete industry.

However, as a newer product, many entities are wary of using SCC until a full understanding of its behavior and structural integrity is understood. One particular area of concern is the interaction between the prestressing strand and the concrete, as this is a vital aspect of any precast/prestressed member. Alleviation of this concern will speed widespread implementation of SCC.

In order to address some of these issues, the Alabama Department of Transportation (ALDOT) sponsored an investigation performed by the Auburn University Highway Research Center. The study explored the use of SCC in precast/prestressed bridge girders by first looking at many different aspects of the issue at a small scale before applying the results to larger projects with increasingly narrow scopes until a thorough understanding of the topic was achieved. As the last phase of the investigation, information found from earlier investigations was applied to the production of full-scale, bulb-tee bridge girders for actual bridge construction. In this case, a bridge will be constructed on State Route 22 over Hillabee Creek in Tallapoosa County, Alabama, two spans of which will be supported by SCC girders. The remaining two spans will be supported by conventionally vibrated concrete (CVC) girders.

## 1.2 Research Objectives

The focus of this thesis was on the prestress bond behavior of SCC girders relative to that of CVC girders as expressed in the transfer length of the prestressing strand. This relationship was determined by monitoring the girders and assessing their performance, which was then compared to previous investigations as well as to requirements set forth by the American Association of State Highway and Transportation Officials (AASHTO).

Five characteristics, previously not addressed in other studies, were included when establishing the defining aspects of the project. First, the bulb-tee girder shape was used at is a very common shape used for bridge construction. Second, a large cross section was examined as many test specimens and small full-size girders have been examined, but nothing as large as the BT-72 girders. Third, the amount of prestressing in the cross section was maximized as previous projects only included minimal prestressing in order for the specimen to be tested in a laboratory environment. Also, the presence of debonded strands wanted to be examined. Finally, the girders were to be produced using standard plant production methods with minimal research influence in order to mimic girders that would be created for the construction of any bridge.

### 1.3 Research Scope

Twelve of the twenty-eight bulb-tee girders cast for use within the bridge over Hillabee Creek were selected for evaluation. The girders were PCI bulb-tee girders 54 in. tall (BT-54) and PCI bulb-tee girders 72 in. (BT-72) tall, which were 97 ft 2 in. and 134 ft and 2 in. long, respectively. Six of the girders, three of each size, were cast using SCC. The remaining six, which were also composed of three girders of each size, were cast with CVC. After the girders were fabricated, transfer lengths were measured after release of the prestressing strand, as well as at 7 days and 28 days after release.

### 1.4 Organization of Thesis

The results of a literature review performed in relation to transfer length and SCC are summarized in Chapter 2. The pertinent definitions and code provisions dictating the calculations used to predict the transfer lengths based upon ACI's *Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary,* AASHTO's *Standard Specifications for Highway Bridges* (2002), and AASHTO's *LRFD Bridge Design Specifications* (2010) are outlined. Bond theory is also discussed in relation to transfer length, describing the mechanisms through which the strand anchorage is provided, along with various different factors that can influence the transfer length. In addition, results from previous studies looking at transfer lengths in both CVC and SCC are presented to support and further the knowledge base on this subject.

Information that describes the specimens in this experimental study is outlined in Chapter 3. Detailed descriptions of the girders are presented, including sizes, dimensions, and reinforcement configuration. The material properties of the concretes, prestressing strand, and mild steel are given, along with information on the components and manufacturers of the products. Finally, a detailed description is given of the process of girder production and preparation for testing.

The transfer length testing program is detailed in Chapter 4. An improved method for measuring the transfer length is explained in full detail, including how the materials were fabricated that were then used in testing, along with how the measurements were taken. The method used to analyze the raw measurements and determine the transfer lengths is described.

The outcomes of the study are described in Chapter 5. Comparisons and conclusions are drawn from the results collected, and related to the three previous phases of research. In addition, many of the factors influencing the transfer length described in Chapter 2 are addressed in light of the results found in the investigation.

Finally, an overview of the study and the important and noteworthy results gathered from the research is given in Chapter 6.

### 1.5 Notation

AASHTO's *LRFD Bridge Design Specifications* (2010) has become the governing specification used in association with precast/prestressed concrete bridge girders. Consequently, the notation used within the specification was also used in association with this thesis. A description of the notation used is listed in Appendix A.

# Chapter 2 Review of Transfer Bond Behavior and Self-Consolidating Concrete in Precast/Prestressed Concrete Members

## 2.1 Introduction

Prestressed concrete members rely on adequate anchorage of the prestressing strand within the concrete. If this anchorage fails, the integrity of the member will be compromised with the possibility of a catastrophic failure. This loss of anchorage is likely to result in significantly reduced shear or flexural resistance. Consequently, it is important to not only understand anchorage, but it is also necessary to understand its different aspects, the various factors than can affect the quality of the anchorage, and consequently, understand how to use this information to properly design and produce safe members.

This chapter addresses anchorage of prestressing strand by means of bond. First, definitions and descriptions are given for key terms used throughout the study. The history and implications of current code provisions dictating aspects of strand anchorage are provided, as well as the presentation of different mechanisms and factors that could influence the quality of strand anchorage. In addition, the findings of recent research regarding transfer bond in SCC are summarized.

## 2.2 Definitions

In order to establish a basis of understanding, the following definitions are given based on current code provisions to describe key terms that are used throughout, and are essential to, this study.

### 2.2.1 Transfer Length

In order to produce an effectively prestressed member, the force applied to the prestressing strands prior to casting must be transferred to the concrete after it has hardened by releasing the strands, which is referred to as release or transfer. The resulting stress in the prestressing strands after transfer, $f_{pt}$, is less than the original jacking force due to losses from strand chuck seating, steel relaxation, and elastic shortening of the concrete. The entirety of the prestress force cannot be transferred to the concrete at the end surface of the member; it is transferred gradually over a finite distance of embedded strand. The measurable distance from the end of the girder to the location at which the full effective prestress force is developed in the concrete is considered the transfer length, $l_t$ (ACI 318-08, Section R12.9). The mechanisms by which the force is transferred through bond are discussed in Section 2.6.

### 2.2.2 Flexural Bond Length

As a prestressed member undergoes external loading, additional force must be transferred into the prestressing steel to counteract the forces being exerted upon the member. The maximum steel stress that must be resisted to achieve the nominal flexural strength, $M_n$, is denoted $f_{ps}$. The flexural bond length, $l_{fb}$, is the additional distance beyond the transfer length required for $f_{ps}$ to be developed in the member (ACI 318-08, Section R12.9). Since external loading was not applied to the specimens in this thesis, flexural bond lengths were not examined.

### 2.2.3 Development Length

The development length, $l_d$, is the total distance from the end of the beam to the location at which the design strength of the reinforcement, $f_{ps}$, is developed (ACI 318-08, Section 2.2). In terms of prestressed members, this is the sum of the transfer length and the flexural bond length. Figure 2-1graphically displays this quantity, along with transfer length and the flexural bond length. It also charts the changes in the prestressing stresses as the strand becomes further embedded. While development length is not investigated directly as part of this study, it is of importance because if the length of bonded tendon at a specific section is less than the development length, the member will never be able to achieve $f_{ps}$ and the corresponding nominal flexural strength (Barnes, Burns, and Kreger 1999).

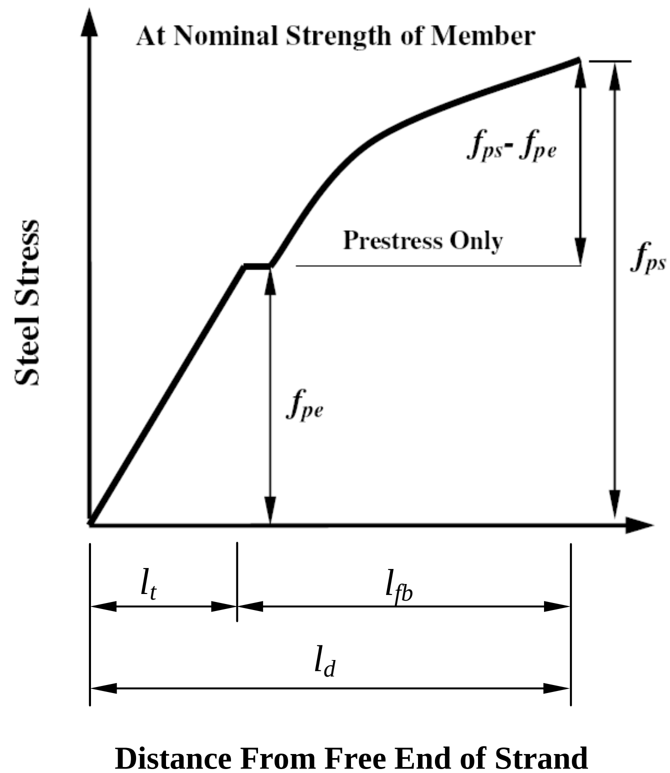



**Figure 2-1: Development of Steel Stress in a Pretensioned Member**
(Barnes, Burns, and Kreger 1999, from ACI 318R-99, Fig. R12.9)

### 2.2.4 Debonded Strands

To optimize a prestressed section, a large prestressing force is typically applied close to the bottom of the member. When these strands are released, a resulting tensile force is created on the top side of the member and large compressive stresses are induced at the bottom of the member. Near the middle of the member span, these stresses are counteracted by the self-weight of the girder. However, at the ends of the member, the tensile or compressive stresses can be so large that the concrete in these regions may be compromised. To prevent such an effect, some of the prestressing strand may be draped and anchored at a higher location towards the end of the beam while held at a lower location at the center of the beam, as seen in Figure 2-2. Other times, strands are prevented from bonding to the concrete at the end of a beam, which is referred to as partial debonding, which can be seen in Figure 2-3. To debond strands, plastic sheathing is placed around the strands for a prescribed distance from the end of the beam prior to casting, preventing a bond from being created between the strand and the concrete, resulting in reduced effective prestressing at the girder ends. In many cases, both methods are implemented to adequately reduce the prestressing effects in the concrete in the end regions. As the strand exits the sheathing and becomes bonded to the concrete, it too will exhibit transfer and flexural bond lengths, both of which are measureable quantities.


**Figure 2-2: Draped Strands**


**Figure 2-3: Debonded Strands**

### 2.2.5 Self-Consolidating Concrete

Self-consolidating concrete (SCC) is a type of concrete that is defined as “a highly flowable, yet stable concrete that can be spread readily into place and fill the formwork without any consolidation and without undergoing significant separation” (PCI 2003). In addition to this definition, Girgis and Tuan (2005) list three characteristics that further describe SCC: “flow ability (the ability to fill all spaces in formwork under its own weight), passing ability (ability to fill spaces around reinforcing bars and other reinforcement under its own weight), and resistance to segregation (composition remaining uniform throughout transportation and placement).” In light of these characteristics, the advantages of using this type of concrete are that typically it is of a better quality and has an increased production rate over conventional concrete (Naito, Parent, and Brunn 2006). This is achieved by the fact that the concrete can consolidate itself and does not require additional workers to vibrate the concrete, which also improves safety measures for the individuals working in concrete production facilities. However, there are also concerns about the performance of SCC, including concerns about the possibility of segregation, poor air-void system, shrinkage, and reduced bond strength between strands and concrete (Ozyildirim 2008). Consequently, the transfer bond performance of this type of concrete in prestressed girders was of particular interest and was the main focus of this research study.

## 2.3 Code Provisions for Anchorage of Fully Bonded Strands

Different design codes specify different methods on how the various lengths associated with prestressing anchorage are calculated. The following sections summarizes the current code provisions of ACI's *Building Code Requirements for Structural Concrete (ACI 318-08)*, AASHTO's *Standard Specifications for Highway Bridges* (2002), and AASHTO's *LRFD Bridge Design Specifications* (2010).

### 2.3.1 ACI 318-08

ACI's *Building Code Requirements for Structural Concrete (ACI 318-08)* directly address the development of prestressing strand in Section 12.9 of the code. As part of this section, the development length equation is given as:

$$l_d = \left(\frac{f_{pe}}{3000}\right) d_b + \left(\frac{f_{ps} - f_{pe}}{1000}\right) d_b \qquad \textbf{\textit{Equation 2-1}}$$

Where: $f_{pe}$ is the effective stress in the prestressing steel in psi,

$f_{ps}$ is the stress in the steel which produces the nominal moment in psi, and

$d_b$ is the strand diameter in inches.

The commentary for Section 12.9.1 goes on to explain that Equation 2-1 is made up of two components: the first term, which represents the transfer length, and the second term, which may be used to determine the flexural bond length. Consequently, the equation for the transfer length of any prestressed member is described as:

$$l_t = \left(\frac{f_{pe}}{3000}\right) d_b \qquad \textbf{\textit{Equation 2-2}}$$

Commentary in Section R12.9 indicates that there are external factors which could produce shorter transfer lengths. These include strands with slightly rusted surfaces (as opposed to bright strands) and using a gentle release method. The effects of these and other factors are further addressed in Section 2.5.

However, Section 12.9 is not the only provision in ACI 318-08 which addresses the calculation of transfer length. In Section 11.3.4 and 11.3.5, provisions dealing with the shear strength of a member based upon the concrete in prestressed members, it states that the transfer length shall be assumed as the following for prestressing strand:

$$l_t = 50 d_b \qquad \textbf{\textit{Equation 2-3}}$$

where again, $d_b$ is the diameter of the strand in inches. Thus, the simplified form of Equation 2-3 gives a value equal to the transfer length of Equation 2-2 when $f_{pe}$ is equivalent to 150 ksi. As this may be a low assumption of the effective stress given today's practices, it could result in a shorter transfer length estimation, which, when used to calculate the nominal shear capacity in the end region of a member, could result in an overestimation of the available strength.

### 2.3.2 AASHTO

There are currently two different AASHTO code provisions in use for bridge design, the first being the *Standard Specifications for Highway Bridges* (2002) and the other the *LRFD Bridge Design Specifications* (2010). While many institutions are moving towards the newer LRFD Specifications, there are still entities, including ALDOT, which still use the Standard Specifications whenever allowed. As both codes are in use and address the topics of interest, the provisions of each are examined in the following section.

Section 5.11.4 of the AASHTO *LRFD Bridge Design Specifications* (2010), entitled "Development of Prestressing Strand," details the requirements for transfer length. It is stated that the stress in the strand at the end of the section can be considered zero, which then varies linearly to the end of the transfer length at which point the stress is equivalent to $f_{pe}$. This is the same definition as that given in Section 2.2.1 of this thesis. It is then stated in Section 5.11.4. that

$$l_t = 60d_b \qquad \textbf{\textit{Equation 2-4}}$$

This simple expression agrees with Equation 2-2 when a value of $f_{pe}$ equal to 180 ksi is assumed. As $f_{ps}$ is usually less than 180 ksi, this is a more conservative estimate of transfer length than the ACI 318-08 expressions.

AASHTO's *Standard Specifications for Highway Bridges* (2002) does not directly address transfer length as the *LRFD Bridge Design Specifications* does. Instead, a similar clause to that of ACI 318-08 can be found amongst the shear provisions for prestressed concrete. In Section 9.20.2.4, the Standard Specifications allow the following assumption to be made for the transfer length for sections using prestressing strand:

$$l_t = 50d_b \qquad \textbf{\textit{Equation 2-5}}$$

As with Section 11.3.4-11.3.5 of ACI 318-08, this expression is given for use in calculating the nominal shear capacity of a girder.  Thus, the shorter predicted transfer length of Equation 2-5, is less conservative for strength design than that of Equation 2-2 or Equation 2-4.

### 2.3.3 Background of Anchorage Equations for Fully Bonded Strands

As explained by Barnes, Burns, and Kreger (1999), the history of the development length equations presented above for both the ACI and AASHTO codes are based on the research and reports of Hanson and Kaar (1959) and Kaar, LaFraugh, and Mass (1963), which were done through the Portland Cement Association (PCA). Between the two studies, 83 pretensioned beams were examined and tested for performance in relation to strand size, embedment length, strand slip, strand surface condition, reinforcement percentage, time, transfer length, and concrete strength (Barnes, Burns, and Kreger 1999).

Tabatabai and Dickson (1995) explored the history of development length equation and how it evolved from the research done by Hanson and Kaar (1959) and Kaar, LaFraugh, and Mass (1963), along with a study performed by the American Association of Railroads.  As part of the efforts of the ACI Prestressed Concrete Committee, Alan H. Mattock derived equations that represented the relationships for the transfer length, flexural bond length, and the development length based on the results of the PCA research.  The committee then took the relationship and modified it to create the expression published in the 1963 ACI 318 (Barnes, Burns, and Kreger 1999).  The expression for the development length (in ksi units) was:

$$l_d = \left(f_{ps} - \frac{2}{3} f_{pe}\right) d_b \qquad \textit{\textbf{Equation 2-6}}$$

This is functionally equivalent to Equation 2-1 (in psi units), which has been altered only in terms of units and arrangement of terms since 1963. The transfer length portion of Equation 2-6 was functional equivalent to Equation 2-2. However, in the corresponding ACI 318R-63 commentary, it was stated that this value for transfer length was to be an average value rather than a conservative design value (Tabatabai and Dickson 1995). Furthermore, Barnes, Burns, and Kreger (1999) compared the original PCA transfer length data to the resulting code expression (Equation 2-2), and found that Equation 2-2 underestimated the transfer length for approximately 90 percent of the specimens. This shift from a mean value to an average underprediction was the result of using the effective long-term prestress after *all* losses ($f_{pe}$) in the code equation rather than the effective prestress at transfer (used in the original analysis by Mattock).

Tabatabai and Dickson (1995) go on to describe a clause found in a draft for the 1963 ACI 318 which is thought to be the origin of Equation 2-3. The clause allowed the transfer length to be taken as $50d_b$ for members containing "steel with a clean surface, released gently to a stress of 150 ksi" (Bennett 1963), resulting in the shear design provisions found in ACI 318 Section 11.3.4 and 11.3.5.

### 2.3.4 Recent Research on Transfer Bond in Prestress SCC

While code provisions provide recommendations by which to predict the transfer length, a long train of research has been performed in this area. The following sections examine previous studies that investigated the accuracy of these provisions with respect to prestressed members cast with SCC. Additional studies were summarized by Swords (2005), Levy (2007), and Boehm (2008).

#### 2.3.4.1 Girgis and Tuan (2005)

From the University of Nebraska-Lincoln, Girgis and Tuan (2005) examined three projects which incorporated similar bulb-tee girder shapes: NU1100, NU900, NU1350, as well as similar concrete mixtures that included two SCC mixtures and a conventional mixture. The conventional mixture was the same as the second SCC mixture, but did not contain any viscosity-modifying admixture (VMA) and contained a reduced amount of high-range water reducing (HRWR) admixture, both of which are used to achieve desired concrete properties for SCC. The concrete strength at transfer for the three projects was 6490 psi and 5980 psi for the two SCC mixtures and 6970 psi for the CVC mixture.

The girders contained 0.6-inch, low-relaxation prestressing strand which was tested according to Moustafa pullout test specifications. The average results were approximately the same as the Moustafa pullout test benchmark for SCC Mix 1 (43.4 k compared to the benchmark of 43.2 which was scaled up for 0.6 in. stand). SCC Mix 2 had a significantly higher pullout strength of 54.2 k, which was higher than the Moustafa benchmark. The CVC had strand pullout strength of 48.0 k.

Surface concrete strain measurements were taken with a demountable, mechanical (DEMEC) gauge for this study, with measurements being taken at 3, 7, 14, and 28 days after casting. The 95% maximum strain method was employed to determine the transfer length, which is fully described in Chapter 5.

After analysis, it was found that the recommendations made by ACI 318 and the AASHTO specifications in Equations 2-3, 2-4, and 2-5 were underestimations of the transfer lengths found in the SCC. The transfer length of Mix 1 was 36 inches, which barely adhered to the provisions of AASHTO LRFD of $60d_b$, but $50d_b$ was an underestimation. Mix 2 of SCC had a transfer length of 43 inches, a value that exceeded all recommendations. It was found however, that the CVC mixture did adhere to the limitations. Consequently, it was hypothesized that the presence of the VMA in the SCC adversely affected the transfer bond, creating longer transfer lengths.

#### 2.3.4.2 Ozyildirim (2008)

At the request of the Virginia Department of Transportation (VDOT), the Virginia Transportation Research Council performed a study led by Ozyildirim (2008) on using SCC in bulb-tee bridge girders. The bulb-tee shape was selected as it provided a more efficient design for use in long span bridges, as well as allowing for wider girder spacing than the AASHTO I-beam shapes. As a result, the constructed bridge on Route 33 over the Pamunkey River in West Point, Virginia contained eight 45-inch bulb-tee girders cast with SCC in one span and an adjacent span constructed with CVC.

Two identical girders were cast with SCC for use in testing, with 28-day concrete strengths of 8340 psi and 8800 psi. No information was given about the concrete strength of the test girders at release or any results of pullout tests that may have been performed. The girders were 60 ft in length and contained draped strands. The transfer lengths were determined and found to be less than the predicted values established from the code provisions.

#### 2.3.4.3 Staton et al. (2009)

A study was performed at the University of Arkansas considering the effects of SCC in prestressed beams. Consequently, twenty prestressed concrete beams with fully bonded strands were cast using two different SCC mixtures and one conventional mixture, all of which were high-strength concretes. The SCC mixtures had 28-day concrete strengths of 10,260 psi to 14,420 psi, and the conventional-slump concrete had strengths of 10,700 psi to 13,100 psi. All of the strands were released when the concrete strength was 5900 psi to 9850 psi. The beams had a 6.5 in. x 12 in. cross-section, were 18 feet in length, and contained two 0.6 in. diameter, 270 ksi, low-relaxation, seven-wire strands. The strand was tested in accordance to the North American Strand Producers (NASP) strand-bond test and found an average of 21 k was required to achieve an end slip of 0.100 inch.

The beams were cast on site at the university using a laboratory mixer. Given the size of the mixer, a single batch of concrete was made and placed before a second batch was made and the forms were filled, with no more than forty-five minutes passing between the final placement of the first batch and the initial placement of the second batch. Consequently, it was necessary to use limited internal vibration in the SCC beams in order to remove the thin crust that formed during the time between the placement of the two batches.

DEMEC strain gauges were used to determine the surface compressive strains, with measurements taken at 3, 5, 7, 14, and 28 days after release, which was done by gradual release to prevent damage to the ends of the beams that could result from a sudden method of release.

At the conclusion of the study, it was found that the transfer lengths of the beams tested were approximately 60 percent of those predicted based upon the code provisions provided by ACI 318-05 and AASHTO LRFD, indicating that the recommended expressions for transfer length were conservative for strength design.

#### 2.3.4.4 Ziehl et al. (2009)

The University of South Carolina conducted a study with three AASHTO Type III prestressed girders which were 59 ft 2 in. in length and cast with lightweight SCC which had 28-day compressive strengths which ranged from 6826 psi to 9167 psi, and an average release strength of approximately 6000 psi. Results of any pullout tests, if performed, were not given. Among other aspects of the study, the three girders were tested with respect to their transfer lengths. DEMEC points were used with both digital and analog strain gauges to collect the compressive surface strains of the concrete before and after transfer. No long-term measurements were taken. As with the other studies, the 95% average maximum strain (AMS) was utilized to determine the transfer lengths.

After interpreting the analysis results, it was found that even lightweight SCC adhered to the AASHTO LRFD provisions. It was found that the transfer lengths were equivalent to $44d_b$, which was less than $60d_b$. While the team realized that the three girders were a limited research sample, it still appeared that the code provisions were conservative for strength design.

#### 2.3.4.5 Pozolo and Andrawes (2011)

At the University of Illinois at Urbana-Champaign, a different approach was taken to examining transfer lengths. Pozolo and Andrawes (2011) performed an extensive overview of previous research projects performed with SCC and used finite element modeling to estimate the transfer lengths. However, a single SCC box girder was cast to collect experimental results. Bond quality was explored by performing fifty-six Moustafa pullout tests in both SCC and CVC. It was found that the difference between the first slip of the strands in the two types of concrete varied by 10 percent for all of the tests with first slip occurring at approximately 27 k. In order to discount any affects due to the variation of concrete strength, all the results were normalized and specific concrete strengths were not reported. As with the other studies, once the girder had been cast and the prestressing force was released, at which point the concrete strength was approximately 4000 psi, DEMEC gauges and points were used in conjunction with the 95% AMS method to determine the transfer length. As with the other studies, the ACI and AASHTO limitations on the extent of the transfer length encompassed the results, again proving the provisions to be conservative for strength design.

## 2.4 Code Provisions for Anchorage of Partially Debonded Strands

In many larger prestressed members, it is necessary to include partially debonded strands in the design. The following sections summarize the code provisions of ACI's *Building Code Requirements for Structural Concrete (ACI 318-08),* AASHTO's *Standard Specifications for Highway Bridges* (2002), and AASHTO's *LRFD Bridge Design Specifications* (2010) concerning anchorage of debonded strands.

### 2.4.1 Code Provisions

ACI's *Building Code Requirements for Structural Concrete (ACI 318-08)* (2008) does mention adjustments in the equations pertaining to the lengths associated with the development of the prestressing strand that are to be made in the presence of debonded strands. In section 12.9.3, it is stated that the development length, as shown in Equation 2-1, shall be doubled in situations when the precompressed tensile zone goes into tension under service load conditions. It thus follows that for members in which tension is never developed under service loads, the development length need not be doubled. In addition to this, the Commentary indicates that "for the analysis of sections with debonded strands at locations where strand is not fully developed, it is usually assumed that both the transfer length and the development length are doubled" (ACI 318 2008).

AASHTO *LRFD Bridge Design Specifications* (2010) also addresses the presence of partially debonded strands in Section 5.11.4.3 in situations in which the precompressed tensile zone is in tension when being subjected to service loads. While this section addresses the allowable amount and location of the debonded strands, as well as the adjustments to be made to the development length equation in the presence of debonding, the transfer length once a debonded strand becomes bonded to the surrounding concrete is not addressed. The only modification made to Equation 5.11.4.2-1 of Section 5.11.4.2 of the code, which calculates the development length, is a modification factor of two. Since nothing is mentioned regarding the associated transfer length for debonded strands, it is unclear whether the transfer length should also be multiplied by the modification factor of two along with development length or if it remains the same as for fully bonded strands.

Section 9.28.3 of the AASHTO *Standard Specifications for Highway Bridges* (2002) requires that the development length, based upon the development length equation specified in Section 9.28.1, be doubled. As with the AASHTO LRFD and ACI codes, this is only for situations in which the member is subjected to tensile forces in the precompressed tensile zone under service loads and only in areas with debonded strands. However, as with the LRFD provisions, the AASHTO Standard Specifications gives no guidance for how to address the transfer lengths within these zones. The resulting implications for the transfer lengths of partially debonded strands are unclear.

### 2.4.2 Background of Transfer Length Equations for Partially-Debonded Strands

ACI 318-08 cites two investigations in the commentary as part of the background for the provisions of debonded strands. In the first study, Kaar and Magura (1965) studied the behavior of five girders, three of which were subjected to static and dynamic loads until failure, and two girders which were subjected to shear loading until failure. It was found that a girder with twice the development length for debonded strands performed the same as one with draped strands when subjected to flexural loads tested to failure. At the same time, a girder with only a single development length developed fared significantly worse. When tested for shear capacity, a girder with twice the development length again performed similarly to a girder with draped strands, resulting in the code provisions suggesting a development length of twice that of fully bonded strands.

Rabbat et al. (1979) looked at the effect of fatigue loads on debonded strands in two different service load scenarios: the first, when the bottom fiber was stressed to zero tension, and the other when it was stressed to $6\sqrt{f'_c}$ under service-load cycles. It was found that in the cases in which tension was not developed at service loads, a single development length was adequate, but when tension was developed in the bottom fiber of the test specimens, twice the development length was needed. It was after these finding were published that the 1983 code began to stipulate that the development length for debonded strands need only be doubled in cases in which the precompressed tensile zone would be in tension under the service loads.

### 2.4.3 Previous Studies Associated with Partially Debonded Strands

While there has not been much research performed for debonded strands in SCC, previous researchers have investigated debonded strands and their performance in convention-slump concrete when compared with fully bonded strands. This section looks at two of these studies.

#### 2.4.3.1 Russell and Burns (1993)

Guidelines for debonding strands were available in the early 1990s but were based on engineering judgment rather than on experimentation. Consequently, a study at the University of Texas was performed by Russell and Burns (1993) examining the benefits and feasibility of debonding strands as an alternative to draping strands. Draped strands are created by stressing and then lowering the hold-down points, a process which has caused flawed strands to fail. It has been seen that these are areas of weakness as hold-down point apparatuses can fail and strands have snapped as a result of the higher forces. Not only do debonded strands not experience the additional stresses, but all that is required to construct them is to jacket the strands in a plastic sheathing, which is an easier and simpler procedure than draping the strands. The goal of the study was to be able to provide guidelines for the use of partially debonded strands based upon experimental evidence.

From the experimental results of the static tests performed on I-shaped beams, it was found that members with debonded strands were sensitive to cracking. Slip occurred as cracks formed, due to flexure or web shear, within the transfer zones of the debonded strands. It was found that beams containing staggered debonding, at which the debonded length varied from strand to strand, performed better than those with debonded lengths that were uniform.  This was attributed to the fact that beams with concurrent debonding were more susceptible to cracking which then led to bond failures.

The experimental measurements taken by Russell and Burns (1993) indicated problems within the code provisions.  It was determined that the provisions were too restrictive for debonded strands and in many cases were misleading in such a way that would permit unsafe designs.  As a result, recommendations were made to isolate the debonded strands to ensure they never experienced any cracking.  An expression was given which would limit the debonded length in simply supported members, and thus prevent cracking in these regions, and can be seen in Equation 2-7.

$$\frac{L_b+L_t}{Span} < \frac{1}{2}\left(1-\sqrt{1-\frac{M_{cr}}{M_u}}\right) \qquad \textbf{\textit{Equation 2-7}}$$

Where: $L_b$ is the debonded length in inches

$L_t$ is the transfer length in inches

$M_{cr}$ is the cracking moment in kip-inches

$M_u$ is the ultimate moment in kip-inches

Which can then be simplified for highway bridge girders and become:

$$L_b + L_t < 0.16 * Span \qquad \textbf{\textit{Equation 2-8}}$$

#### 2.4.3.2 Barnes, Burns, and Kreger (1999)

Barnes, Burns, and Kreger (1999) also performed a study at the University of Texas at Austin which examined the anchorage behavior of 0.6 in. strands arranged in a 2 in. grid pattern within AASHTO Type I I-Beams.  As part of the investigation, several different debonded configurations were examined.  While it was determined that debonding was an effective method of reducing extreme concrete stresses after the release of the prestressing strand, it was found that the transfer lengths of the debonded strands were no longer than comparable fully bonded strands.

Special care has to be taken to ensure that the debonded strands are fully developed in flexural members. As transfer zones with debonded strands are nearer the center of the beam than transfer zones of fully bonded strands, there is an increased risk that critical cross-sections are located within the transfer zone for the partially debonded strands. If cracking occurs in these areas, the anchorage of the debonded strands can be compromised and the structural integrity of the beam can be impaired.

In addition, Barnes, Burns, and Kreger (1999) recommended that in relation to development length of partially debonded strands, the prestressing strand behaved similarly to cutoff bars in reinforced concrete. Consequently, debonded strands should also be subjected to the code provisions dictating the limitations on terminating nonprestressed reinforcement. Meanwhile, another way to properly guarantee adequate anchorage of the debonded strands is to ensure cracking does not occur across the bonded length of strand within $20d_b$ of the transfer length. Consideration of these ideas offers an alternative explanation for the anchorage failures of debonded strands experienced in the previous studies. Further study was recommended to simplify the design provisions for anchorage of partially debonded strands.

### 2.5 Previous Research Associated with this Study

The research presented in this study is an extension of an earlier study examining the potential implementation of self-consolidating concrete in precast, prestressed bridge girders. Consequently, the three earlier studies of transfer bond in Alabama SCC are outlined in the following sections.

### 2.5.1 Concentrically Prestressed Prisms (Swords 2005)

Swords began the extended study by examining thirty-six concentrically prestressed concrete specimens. The 10-foot long prisms were cast from five different concrete mixtures, which included one CVC mixture and four SCC mixtures. The SCC mixtures included high-strength and low-strength mixtures using Class C fly ash as an admixture, and high-strength and low-strength mixtures utilizing ground-granulated blast-furnace (GGBF) slag. Six prisms were cast using each mixture, three of which were prestressed with a single strand, and three which were prestressed with two strands spaced at 2 in. on center (Swords 2005). The configurations of these prisms can be seen in Figure 2-4. An additional set of six prisms were cast from high-strength SCC with fly ash admixture because the first set had an air content that exceeded ALDOT's specifications for maximum allowable air content.

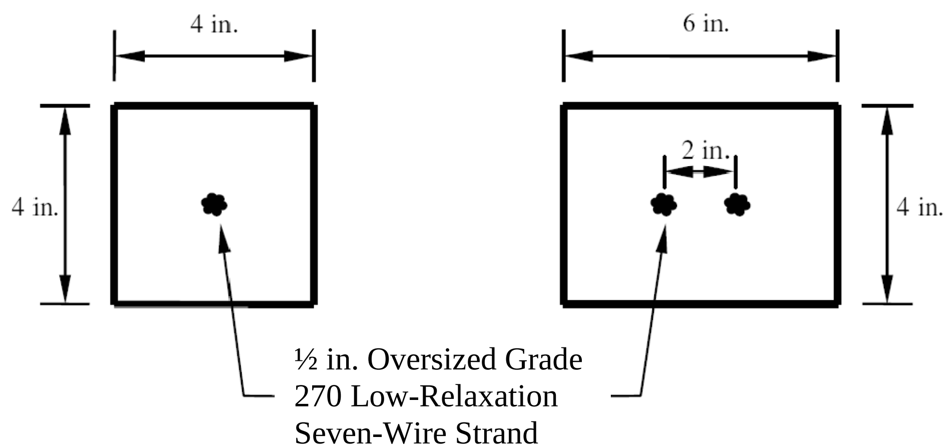



**Figure 2-4: Concentrically Prestressed Singe- and Double-Strand Cross Sections** (Swords 2005)

As a result of having thirty-six prisms, seventy-two transfer zones were considered as a transfer length was measured at each end of a prism. The strands were flame cut, and mechanical strain gauges were used to measure the surface strains in the concrete.  The strains were then analyzed to determine the transfer lengths.  Transfer lengths were determined at four different ages: immediately after transfer, 2–4 days after transfer, 7 days after transfer, and 28–46 days after transfer.

### 2.5.2 Eccentrically Prestressed T-Beams (Levy 2007)

Levy continued the study by testing sixteen T-beams with two ½-inch ‘special’ strands.  Four different concrete mixtures were utilized to cast the beams, one of which was CVC, and three were SCC mixtures.  The mixtures were the same as those used by Swords in 2005, with the exception that the high-strength SCC mixture with Class C fly ash was not used.  Four beams with varying lengths of 9 ft 8 in., 13 ft 0 in., 16 ft 4 in., and 23 ft 0 in. were cast from each concrete mixture.  The cross section utilized for the project was constant throughout and can be seen in Figure 2-5.

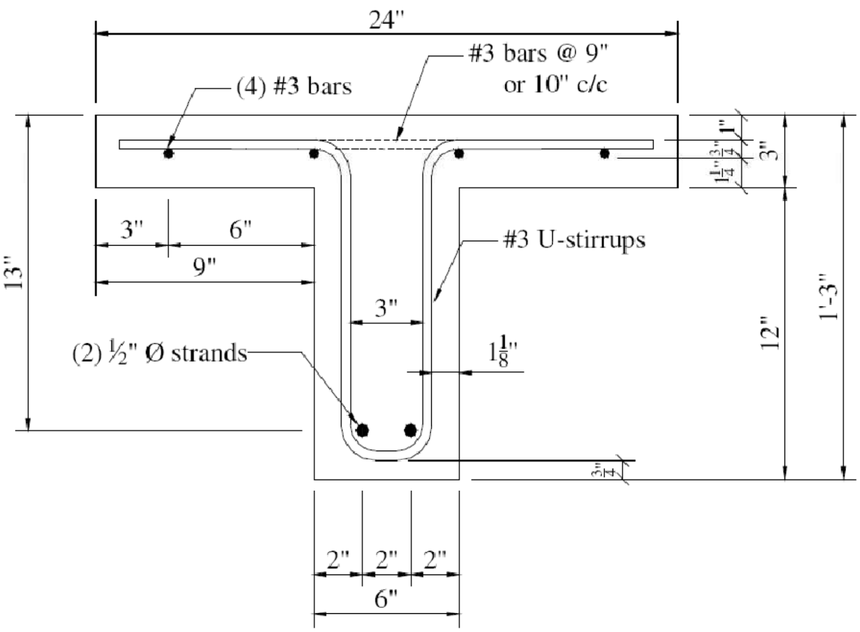



**Figure 2-5: Eccentrically Prestressed Concrete T-beam Cross Section**
(Levy 2007)

The prestressing strand was released by flame-cutting and was done in such a way that a live end and a dead end were created. As a result, thirty-two transfer zones were created, each of which were determined from measured concrete surface strains. Measurements were taken immediately after transfer of the prestressing to the concrete and 4 days after transfer.

### 2.5.3 Prestressed AASHTO Type I Girders (Boehm 2008)

To ensure the results from the previous small-scale tests could be applied to full-scale bridge girders, Boehm studied the behavior of six prestressed AASHTO Type I girders. Three concrete mixtures were used to cast the 40-foot long girders, including one CVC mixture and two SCC mixtures. Two girders were cast of each mixture: CVC, high-strength SCC utilizing GGBF slag, and moderate-strength SCC utilizing GGBF slag. These were the same mixtures used by both Swords (2005) and Levy (2007). SCC containing Class C fly ash was not used as part of the investigation. A typical cross section of the girders observed can be seen in Figure 2-6.

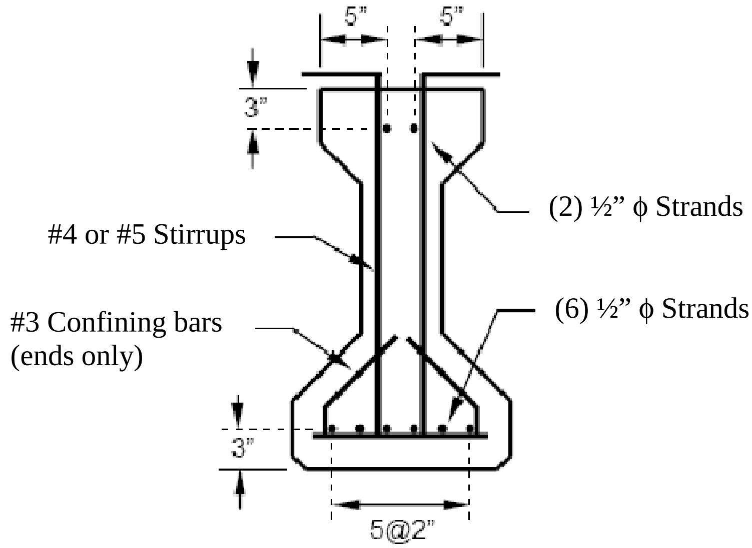



**Figure 2-6: Prestressed AASHTO Type I Girder Cross Section**
(Boehm 2008)

Twelve transfer zones were created after flame-cutting the strands. The surface strains were once again measured using a DEMEC gauge with measurements taken immediately after the transfer of the prestressing force to the concrete, as well as at intervals ranging from 3 days to 3 months.

## 2.6 Transfer Bond Theory

There are several issues that could be considered when examining the transfer length. First, there are the mechanisms by which the bond is actually created between the strand and the concrete. There are also many factors that can have an effect on the quality of the mechanisms. The mechanisms and factors are discussed in this section.

### 2.6.1 Bond Mechanisms

In order for transfer lengths to be developed, stress from the prestressing strand has to be transfer into the concrete. This is accomplished through transfer bond stress.

Hanson and Kaar (1959) identified three mechanisms by which the prestressing strand becomes bonded to the concrete: adhesion, friction, and mechanical resistance.

#### 2.6.1.1 Adhesion

Adhesion is the mechanism in which the concrete literally ‘sticks’ to the prestressing strand. Not only is it a weak bond that is easily broken, but the strand almost always experiences some slip relative to the concrete near the end of the member, breaking the bond that had been developed. Consequently, adhesion plays a minor role in the development of the transfer bond stress (Barnes, Grove, and Burns 2003).

#### 2.6.1.2 Friction

As some slip does occur between the strand and the concrete, friction plays a much larger role than adhesion and is considered the primary mechanism in creating the transfer bond stress (Hanson and Kaar 1959). This is achieved through the Hoyer Effect, which is illustrated in Figure 2-7. This form of friction is a result of the Poisson Effect. As the prestressing strand is stretched, the strand diameter decreases creating a smaller strand. With the steel held in this position, the concrete is cast around the strands and allowed to harden. When the strand is released, it tries to expand back to its original shape, but it cannot expand without compressing the hardened concrete. Consequently, radial compressive stresses result, which allow for stress transfer from the prestressing steel to the concrete through friction.

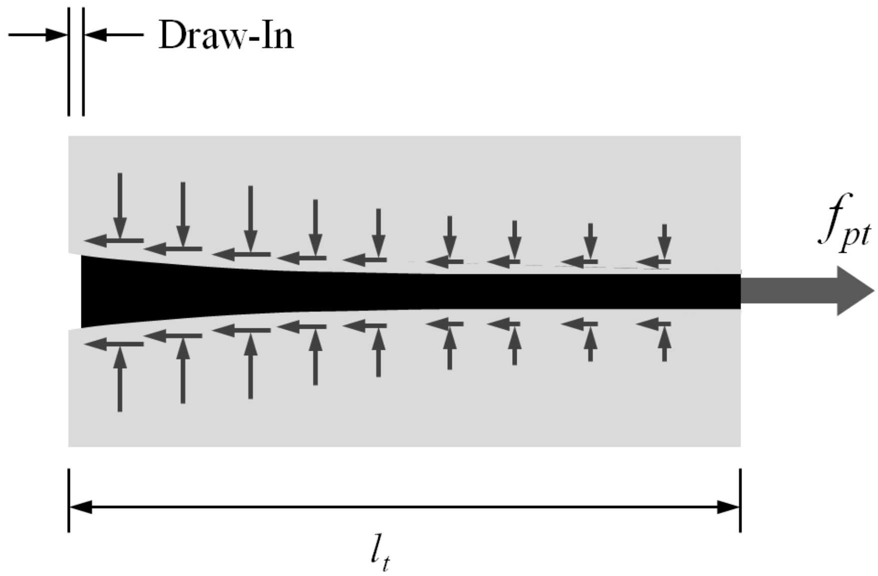



**Figure 2-7: Hoyer Effect**

#### 2.6.1.3 Mechanical Resistance

A smooth wire is incapable of providing any mechanical resistance. However, given the helical pattern of a seven-wire strand of prestressing, the strand can provide mechanical resistance via a bearing stress. Once the concrete is hardened and strands released, the ridges of the strand can push against the grooves cast around the strand, allowing for the stress in the prestressing strand to be transferred to the concrete and prevent sliding (Levy 2007). In the experiments done by Hanson and Kaar (1959), it was seen that members with the seven-wire strand were able to develop larger moments, even after general bond slip had occurred, than members that contained individual smooth strands.

### 2.6.2 Bond Factors

In addition to the mechanisms that create the bond, Hanson and Kaar (1959) identify several factors also affecting the quality of bond, and thus the transfer length. The main factors that contribute to this are the amount of prestress, surface condition of the strand, strength of the concrete, and the method of stress transfer. Yet it was also seen that the concrete strength, percentage of steel, and the embedment length are interrelated terms which must all be considered and taken into account during the design process. These mechanisms and factors are described in the following sections.

#### 2.6.2.1 Time-Dependent Effects

Barnes, Grove, and Burns (2003) report that studies dating back to 1951 have experienced effects relating to time. Typically, the transfer lengths continue to grow for a few days after release, after which it continues to grow at a much slower rate, with measurements taken as many as thirty months after release. Most of these affects can be attributed to the long-term properties of creep and shrinkage in the concrete and relaxation in the prestressing steel (Barnes, Grove, and Burns 2003). As these factors continue to alter the state of the concrete, the interaction between the concrete and steel varies as well.

Results presented by Staton et al. (2009) concluded that the time-dependent effects were more prominent in CVC rather than SCC, which was "attributed to less time-dependent softening of the concrete grip on the strands compared with the high-strength concrete (HSC) transfer-length growth" (Staton et al. 2009).

#### 2.6.2.2 Member Cross-Section

Research performed by Levy (2007) and Boehm (2008), when compared to that done by Swords (2005), documented the effect of cross-sectional size on the transfer length. It was found that as the cross-section increased, the transfer length decreased. Levy fabricated T-beams which were 24 in. in width and 15 in. in height which were then compared to Swords' 6 in. x 4 in. prisms. When the transfer lengths were compared, Levy's transfer lengths were 36 percent smaller than those recorded by Swords (2005). Consequently, Boehm (2008) compared the results collected from the transfer lengths of AASHTO Type I girders to Levy's results, finding that the transfer lengths were 31 percent shorter than the T-beams and 56 percent shorter than the prisms.

#### 2.6.2.3 Concrete Strength

As the strand initially bears on the concrete, an inelastic response is created and cracking occurs in the concrete in the local region directly around the strand. The amount and extent of this effect is dictated by the tensile capacity and the stiffness of the concrete. Both the stiffness and tensile capacity of the concrete can be approximated as being proportional to the square root of the concrete compressive strength. Based upon this theory and experimental results, Mitchell et al. (1993) suggested that the transfer length should be inversely proportional to the square root of the compressive strength (Barnes, Grove, and Burns 2003).

Many experimental transfer length testing programs performed over the last fifty years, including Hanson and Kaar (1959) and Barnes, Burns, and Kreger (1999), found correlations between concrete strength and the transfer length. However, a newer study conducted by Staton et al. (2009) using SCC indicated that there was not a correlation between the concrete strength and the transfer length. Unlike the other studies examined, the study performed by Staton et al. (2009) did not vary the concrete strengths based on variations in the concrete mixtures, but rather releasing the prestressing strand at varying concrete strengths given how long it had been since casting. This method of varying the concrete strength has much less effect on the transfer length than variation of 28-day concrete strength (i.e. variation of the mixture proportions). Consequently, the study does not provide enough evidence to conclude that the concrete strength does not affect transfer lengths, especially as studies performed on SCC and CVC by Swords (2005), Levy (2007), and Boehm (2008) indicate that there is a direct correlation over a wide range of concrete strengths.

#### 2.6.2.4 Prestressing Strand

As can be seen strictly from Equations 2-3, 2-4, and 2-5, an important factor is the strand diameter, as this is the only variable included in all of these equations. This is a result of the fact that as the diameter changes, so does the amount of surface area that can come into contact with the concrete. On the other hand, because the cross-sectional area of the strand increases with the *square* of the diameter, the amount of force that must be transferred over this surface area outpaces the increase in surface area. As a result—for the same level of prestress—the transfer length increases with the diameter.

Evidence has also presented itself that the quality of the strand surface condition can also have an effect on the transfer length. The theory accredits weathered strands as having more frictional resistance than strand that is pristine, thus reducing the transfer length. However, as described by Martin and Scott (1976), it is considered impractical to incorporate the added advantage of weathered strand as it is not a feature that can be manufactured, nor is there a way to dictate the length of exposure required to achieve the proper amount of weathering (Barnes, Grove, and Burns 2003). Furthermore, although Barnes, Burns, and Kreger (1999) observed that weathered strands had a shorter *average* transfer length than nonweathered strands, several specimens with weathered strands exhibited longer transfer lengths than the control specimens. Thus, the dispersion of the results with weathered strands was too large to result in a reliable design advantage.

#### 2.6.2.5 Method of Stress Transfer

There are generally two categories of prestress transfer methods: sudden release and gradual release. As the name indicates, sudden release typically involves the cutting of the strand. A gradual release can be accomplished by moving the block in which the strands are anchored allowing the strand to relax slowly, or a sudden release may be performed at one location prior to releasing any of the other strands. By doing this, the remaining ends would then be considered a gradual release as the majority of the stress has already been released. Numerous studies have shown that transfer lengths associated with sudden release methods result in longer transfer lengths, which is credited to the "dynamic effect associated with the transfer of energy from the strand to the concrete member" (Barnes, Grove, and Burns 2003).

Staton et al. (2009) reported a lack of correlation between the live-end and dead-end transfer lengths recorded. However, this is due to the fact that a gradual release method was employed.

#### 2.6.2.6 CVC versus SCC

Limited information is currently available for the bond performance of SCC, as this type of concrete is still relatively new and has yet to gain widespread acceptance. However, there has been a push to move towards this type of concrete and, as a result, more studies have been performed examining the properties.

Swords (2005), Levy (2007), and Boehm (2008) each found the transfer lengths in girders cast with SCC to be longer that those found in girders cast with CVC. When looking at values obtained from girders cast with concrete with less than 30 percent GGBF slag, it was found that the transfer lengths in the SCC were 28 percent, 4 percent, and 7 percent longer than the transfer lengths in CVC for each of the studies, respectively.

Staton et al. (2009) reported minimal differences in transfer lengths between the first type of SCC cast and the HSC. While it appeared that the transfer lengths in SCC girders were slightly shorter, a statistical analysis showed that the transfer lengths for the two concrete types were the same. For the second type of SCC, transfer lengths were found to be statistically shorter than HSC.

A study at Lehigh University by Naito, Parent, and Brunn (2006) examined the use of SCC in bulb-tee girders. Two concrete mixtures were utilized, one being SCC and the other CVC, with each mixture being cast into two 45 in. tall bulb-tee girders, which resulted in four girders 35 ft in length. Analysis of the girders found that the difference between the transfer length in the SCC was 0.1 inches shorter than that of CVC, again indicating that SCC and CVC are interchangeable.

Results found by Ozyildirim (2008) also indicated that the same methods used for the design of girders with CVC can also be used for the design of girders cast with SCC.

Finally, Ziehl et al. (2009) found that in lightweight SCC, the transfer lengths associated with the SCC were 15 percent greater than those found for the corresponding high-strength lightweight concrete.

### 2.7 Summary

There are many aspects of transfer lengths that must be considered when performing any type of analysis or study. Of the two major organizations that recommend methods of predicting transfer lengths for design, three different approaches are supplied, two of which rely strictly on the diameter of the strand being used, and one which also takes into account the stress in the strand. However, it is known that there are many other factors that can affect the transfer lengths in SCC. Even so, all results except for one situation indicated that the current code provisions provided a safe estimate of the transfer length, even though in many cases it was a significant overestimation.

ACI and AASHTO also address the presence of debonded strands, but only briefly, and not with respect to transfer length. Due to the lack of any additional information, it may be assumed that the transfer length of a debonded strand—measured from the end of the debonded length—is the same as that of a fully bonded strand at the end of a member when not under service loads that create tension in the precompressed tensile zone.

It was also seen that there were three different mechanisms through which the stress in the strand can be transferred to the concrete. The most effective mechanism was friction, which manifested its self through the Hoyer Effect. Through these mechanisms, there were also several factors that played a role in the effectiveness of the mechanisms which included time-dependent issues, member cross section, concrete strength, properties and characteristics of the prestressing strand, method of prestress transfer, and type of concrete (CVC of SCC). Of these, it was determined that the concrete strength, prestressing strand diameter, and method of prestress transfer had the largest impact on the transfer lengths.

From recent research it was also seen that even though the transfer lengths in SCC were sometimes found to be longer than those in CVC, they still generally adhered to the guidelines set forth by the design codes. However, it was still of interest to further explore the effects of SCC in large, full-scale girders in comparison to the test specimens previously examined, as well as the behavior of the transfer lengths of debonded strands.

# Chapter 3 Design and Construction of Experimental Specimens

## 3.1 Introduction

While casting the girders in the fall of 2010 for the four-span bridge on State Route 22 over Hillabee Creek in Tallapoosa County, Alabama, twelve of the twenty-eight girders were instrumented for use in this study of transfer bond behavior. All of the girders were PCI Bulb-Tee girders and were plant-cast with either a conventionally vibrated concrete (CVC) mixture or with a self-consolidating concrete (SCC) mixture. Two different girder sizes were cast: BT-54 with a length of 97 ft 10 in. and BT-72 with a length of 134 ft 2 in. The twelve girders selected to undergo analysis included three BT-54 girders of CVC, three BT-54 girders of SCC, three BT-72 girders of CVC, and three BT-72 girders of SCC. Further details about the design and fabrication of the girder specimens are presented in this chapter.

## 3.2 Specimen Identification

In order to distinguish the twelve girders and the specific transfer zone location being referred to, a specimen identification system was developed and can be seen in Figure 3-1.

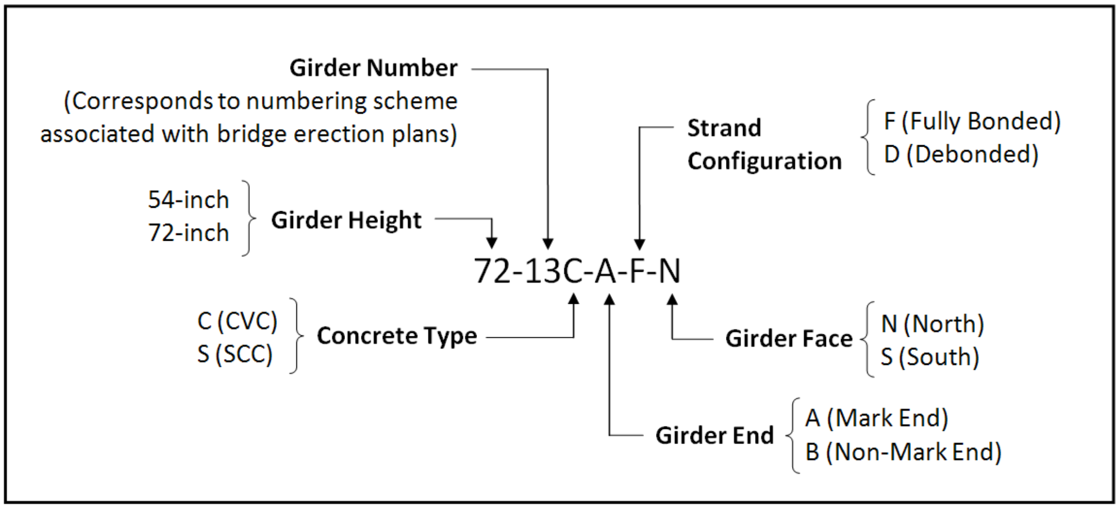



**Figure 3-1: Specimen Identification**

The first distinguishing factor for the girders is the height of the specimen in question. Half of the girders were 54 in. in height and the other half were 72 in. in height, corresponding to the designation BT-54 and BT-72 respectively. A numbering scheme was established by the precast concrete producer to number the girders from 1 to 14 for the BT-54 girders and 1 to 14 for the BT-72 girders to aid during the erection process. These identification numbers were also applied to the girder specimens so that associations between each girder and their final location within the finished bridge were possible if ever needed.

Girders 1 through 7 of both the 54 in. and 72 in. girders were cast using SCC, designated with an ‘S’, and girders 8 through 14 were cast with CVC, designated with a ‘C’. It was then necessary to distinguish between the two ends of each girder as both ends were utilized to collect transfer length data. The mark end, which was located on the east end of the girder when on the casting line in the plant and should be the southwest end of the girder in the actual bridge, is denoted by an ‘A’. The non-marked end is denoted ‘B’.

Finally, two different strand-bonding zones were under investigation in several of the members: the transfer zone for the fully bonded strands and the transfer zone for the debonded strands. The fully bonded transfer strands at the end of each girder consisted of twenty-two strands at the bottom of the member. This region is designated ‘F’. Debonded strands in the bottom of each girder were covered with a plastic sleeve for a length of 10 feet measured from the girder end. The resulting transfer zone for these partially debonded strands is denoted ‘D’. For strain measurement purposes, it was also necessary to distinguish between the north and south faces of the girder while the girder was sitting in the prestressing bed at the plant.

### 3.3 Specimen Design

Figure 3-2 and Figure 3-3 show the standard dimensions of a BT-54 girder and a BT-72 girder respectively. The BT-54 girders were 97 ft 10 in. in length and the BT-72 girders were 134 ft 2 in. In addition to these dimensions, each girder was skewed in order for the bridge to have the proper orientation once constructed. As a result, every girder, regardless of it being a BT-54 girder or a BT-72 girder, had a 15 degree skew.

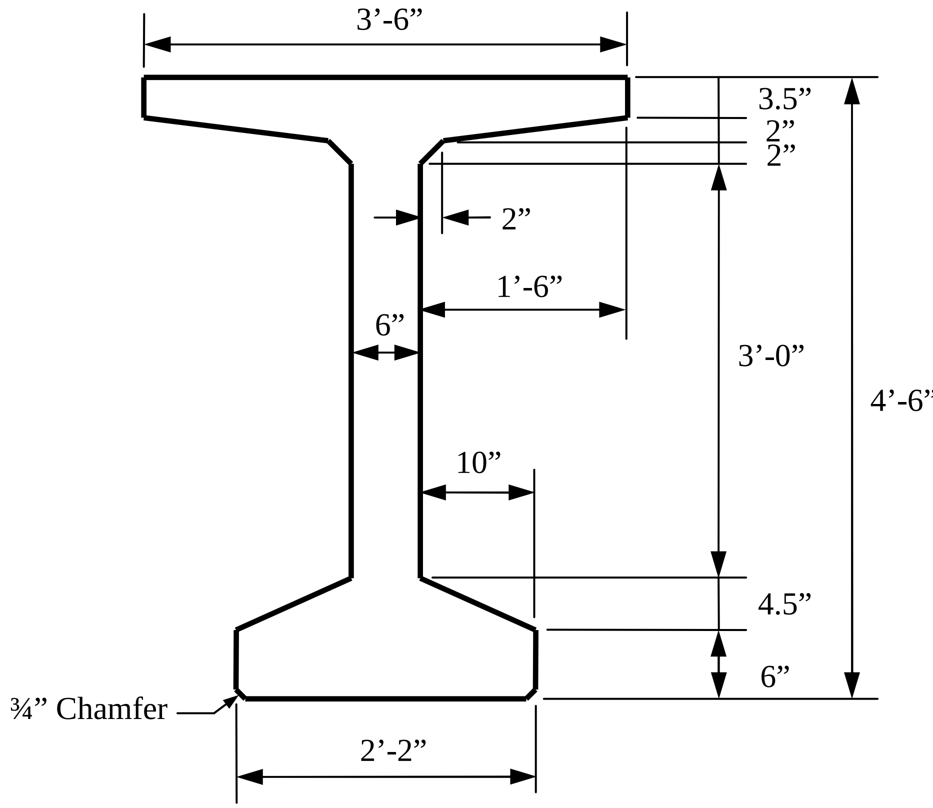



**Figure 3-2: Typical BT-54 Girder Cross Section**

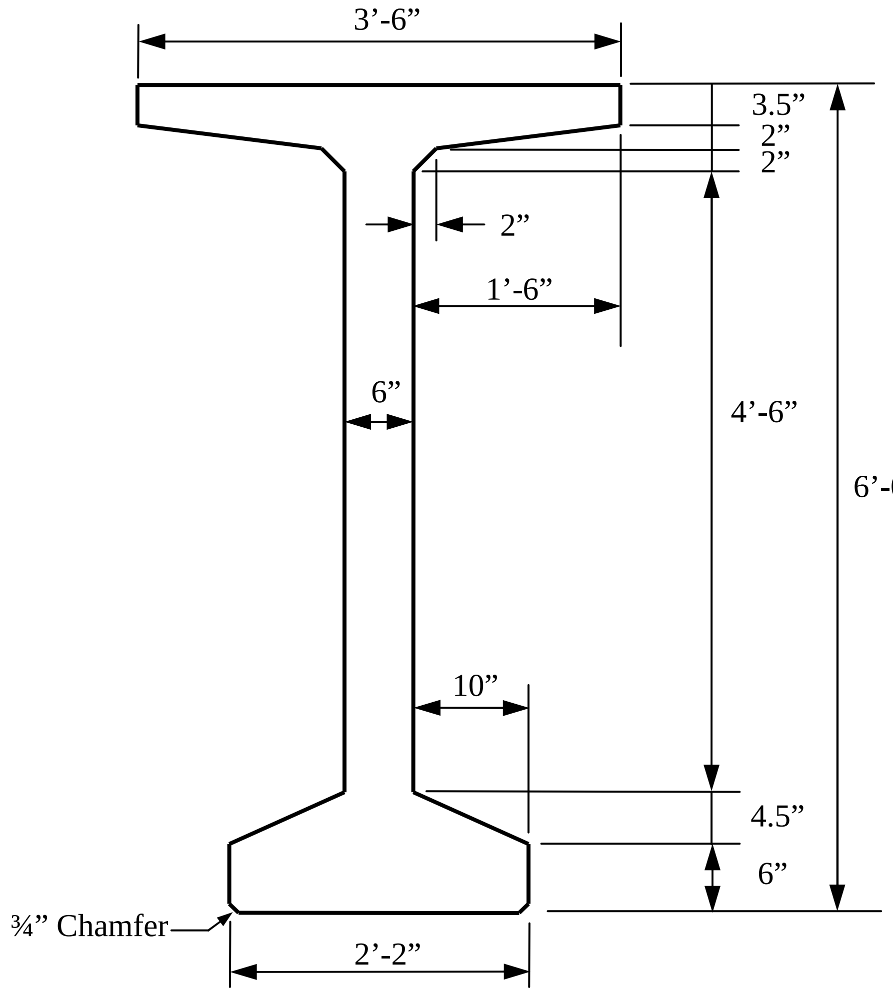



**Figure 3-3: Typical BT-72 Girder Cross Section**

### 3.3.1 Strand Arrangement

Given the difference in size and length between the BT-54 girders and the BT-72 girders, it was necessary to have two different strand arrangements for each of the girders. All strands were seven-wire, Grade 270, low-relaxation strands and were either ½-inch strands or ½-inch 'special' strands. The BT-54 girders contained a total of forty strands: twenty-eight ½-inch diameter strands in the bottom of the section tensioned to 30,980 pounds each, eight ½-inch diameter strands draped along the length of the member tensioned to 30,980 pounds each, and four ½-inch diameter top strands lightly tensioned to 5,000 pounds each.

The BT-72 girders had the same strand arrangement, but had ten additional draped strands, and incorporated ½-inch ‘special’ strand. Specifically, it contained twenty-eight ½-inch ‘special’ diameter strands in the bottom of the section tensioned to 33,800 pounds each, eighteen ½-inch ‘special’ diameter strands draped along the length of the member tensioned to 33,800 pounds each, and four ½-inch diameter top strands lightly tensioned to 5,000 pounds each.

This corresponded to a specified jacking stress ($f_{pj}$) of 202.5 ksi for the draped and the bottom strands and 32.7 ksi for the top strands in both cross sections. The specific location of each strand at both midspan and at the end of the section can be seen in Figure 3-4 and Figure 3-5 for the BT-54 girders, and Figure 3-6 and Figure 3-7 for the BT-72 girders. In addition, the draping profiles can be seen in Figure 3-8 and Figure 3-9.

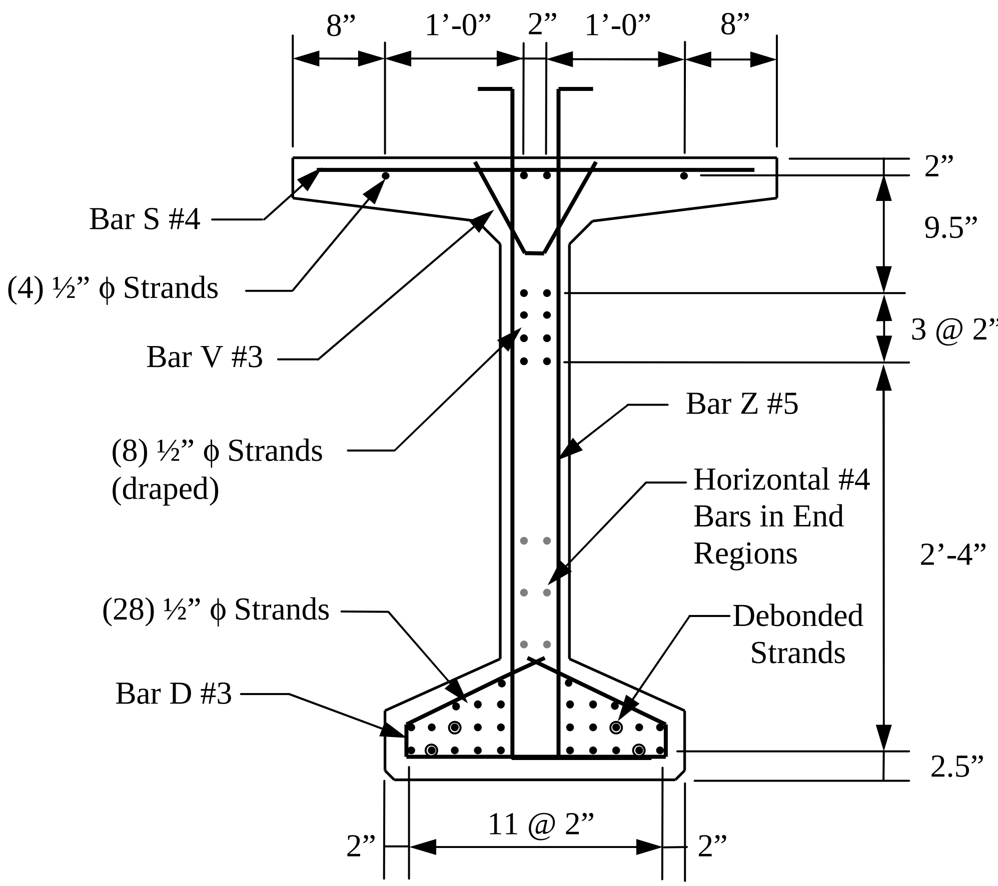



**Figure 3-4: Mild Steel and Strand Arrangement for BT-54 Girder at End of Span**

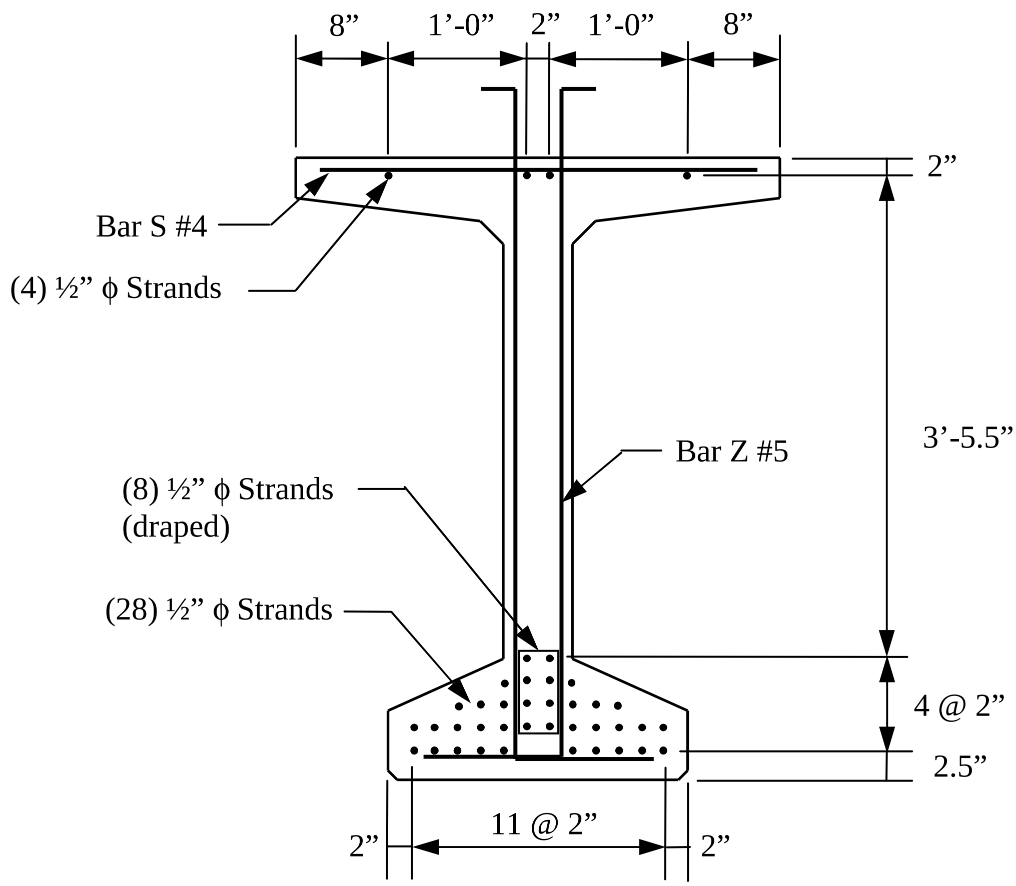



**Figure 3-5: Mild Steel and Strand Arrangement for BT-54 Girder at Midspan**

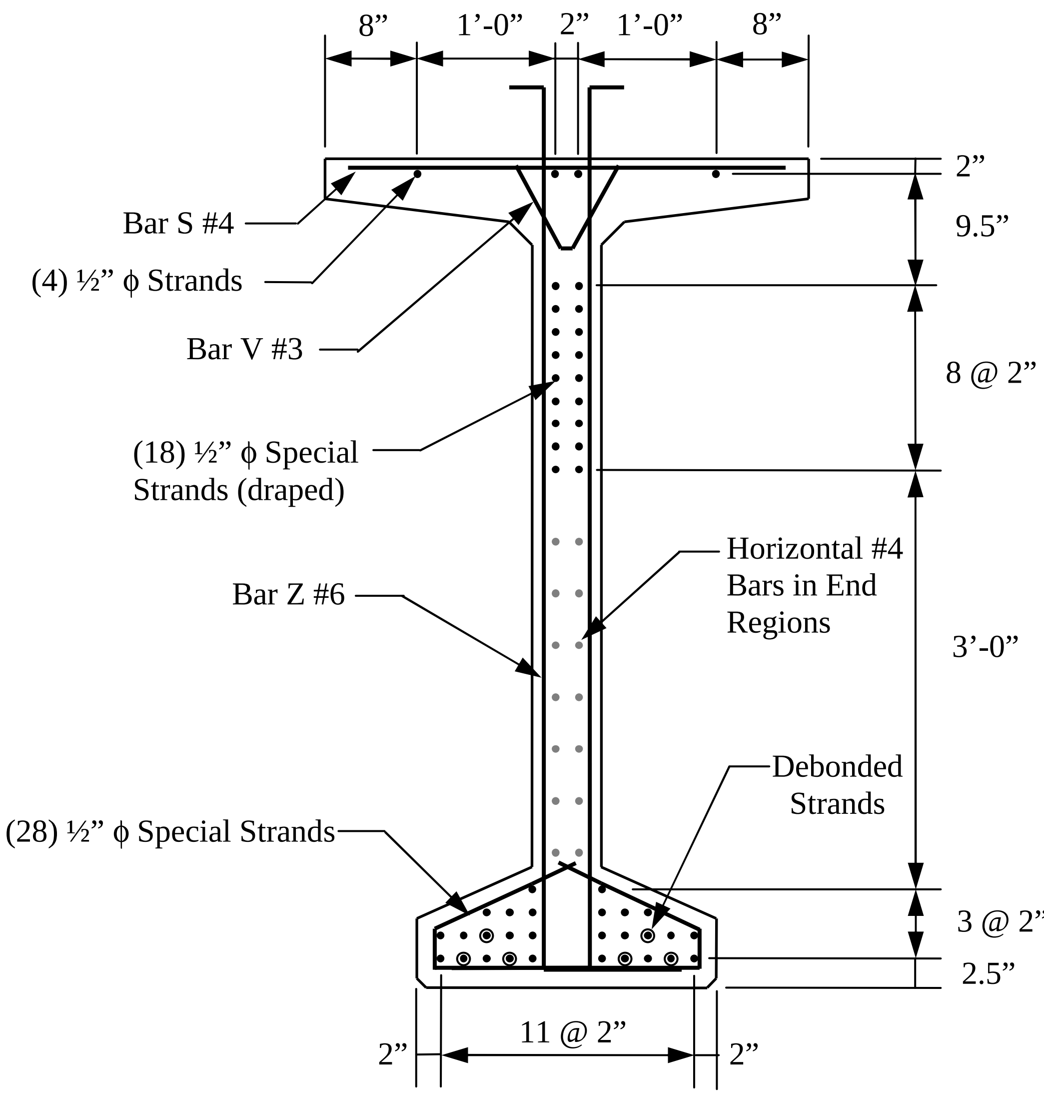



**Figure 3-6: Mild Steel and Strand Arrangement for BT-72 Girder at End of Span**

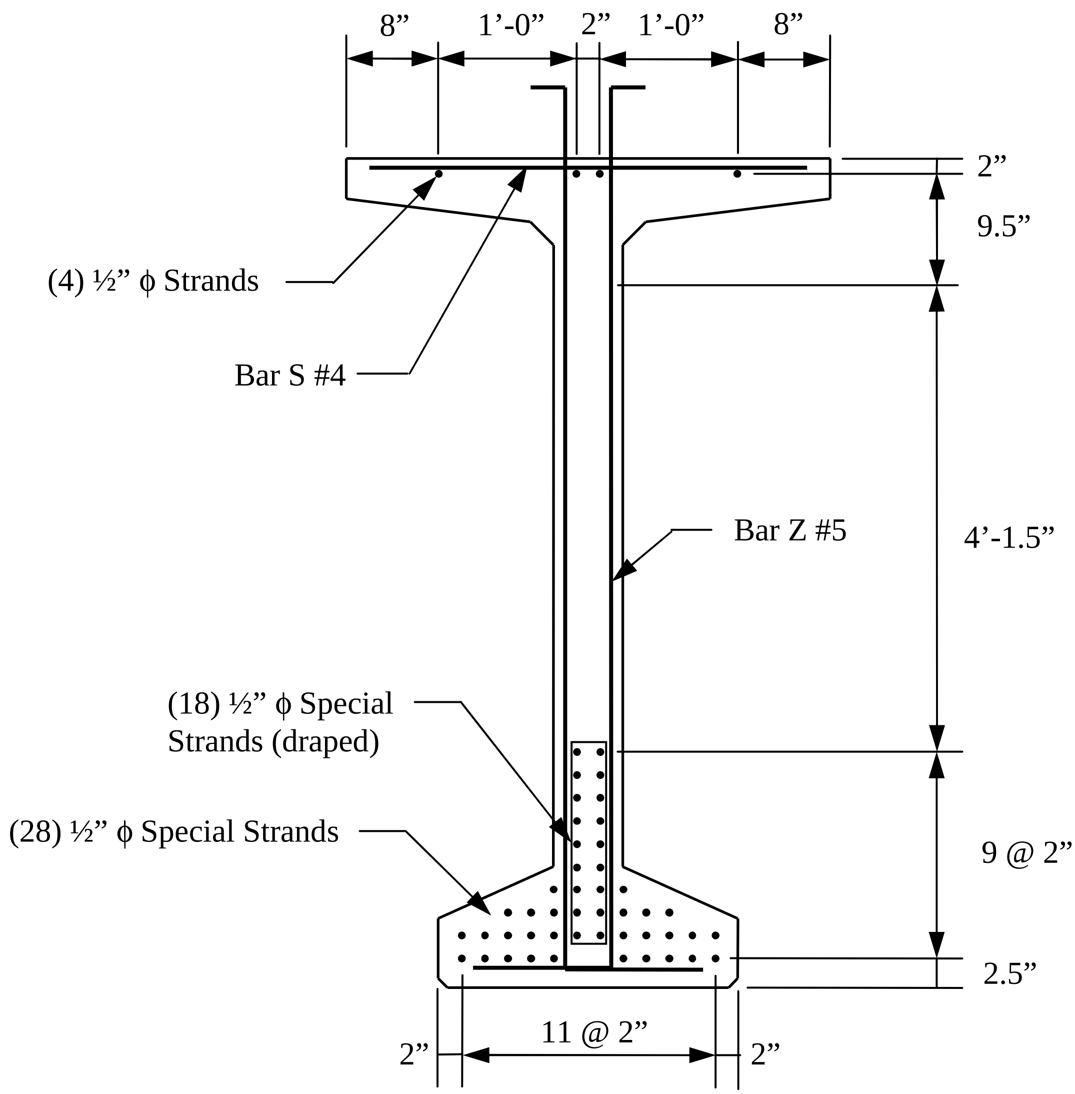



**Figure 3-7: Mild Steel and Strand Arrangement for BT-72 Girder at Midspan**

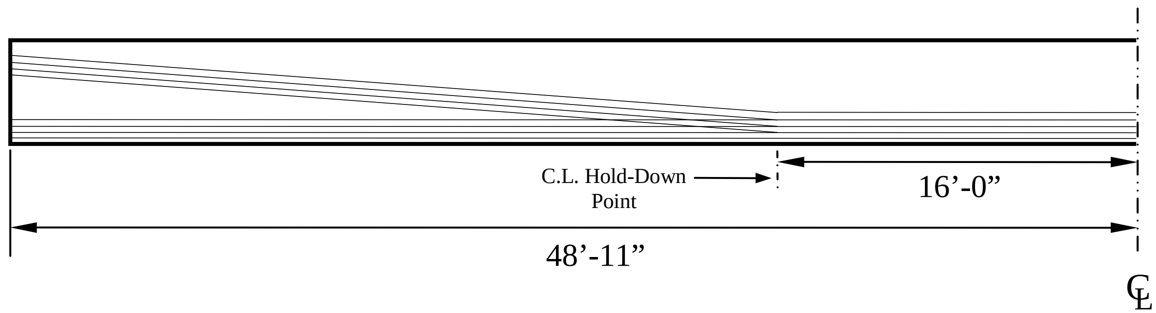



**Figure 3-8: Profile and Hold-Down of Draped Strands for BT-54 Girder**

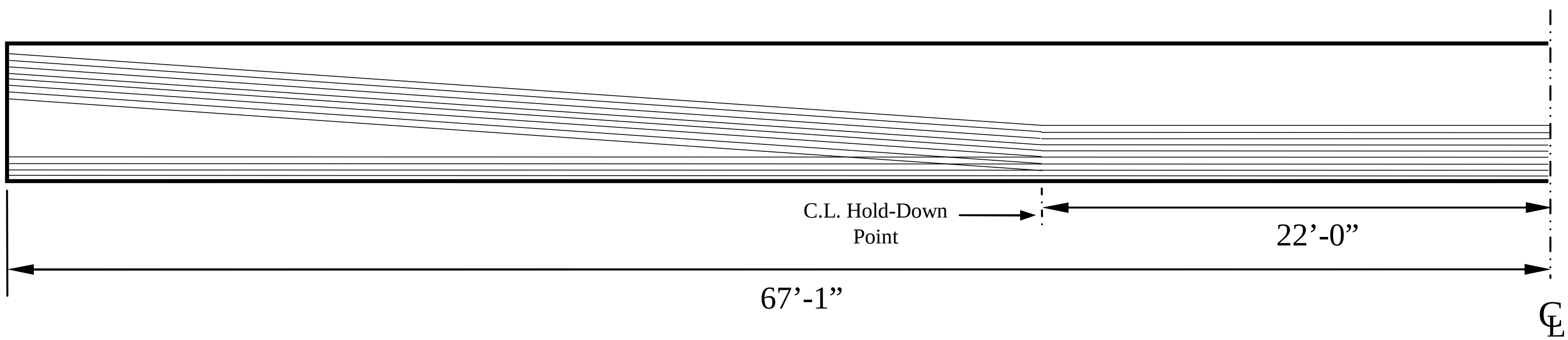



**Figure 3-9: Profile and Hold-Down of Draped Strands for BT-72 Girder**

It was also necessary to debond some strands to satisfy allowable stress limits. Consequently, four strands were debonded for a distance of 10 ft from the end of the girder in the BT-54 girders, and six strands were debonded for the same length in the BT-72 girders. This was accomplished by sheathing the strands with a plastic casing and sealing with tape. The debonded strands are denoted with a circle around the strand in the figures above.

### 3.3.2 Nonprestressed Reinforcement Arrangement

The configuration of nonprestressed steel, used to resist shear forces and anchorage zone forces, was the same for both bulb-tee girder shapes, the only difference being the spacing and distances over which the mild steel reinforcing bars were placed. There were four different shapes used in the steel cages which included Z-bars, bottom steel confinement (D-bars), straight bars (S-bars), and V-bars. These shapes and their location within the cross section can be seen in Figure 3-4 through Figure 3-7. The spacing at which the mild steel is placed throughout the girder is visible in Figure 3-10 and Figure 3-11.

In addition to the mild steel spaced throughout the length of the girder, there were six additional S-bars in the BT-54 girders and fourteen S-bars in the BT-72 girders which were placed in each end of the girder. The additional bars were placed horizontally, parallel to the prestressing strands, and ran from the end of the girder into the web of the section for a distance of 52 in. in the BT-54 girders and 72 in. in the BT-72 girders. The vertical location of the bars can be seen in the cross-section diagrams: Figure 3-4 and Figure 3-6, as well as the longitudinal placement in Figure 3-10 and Figure 3-11.

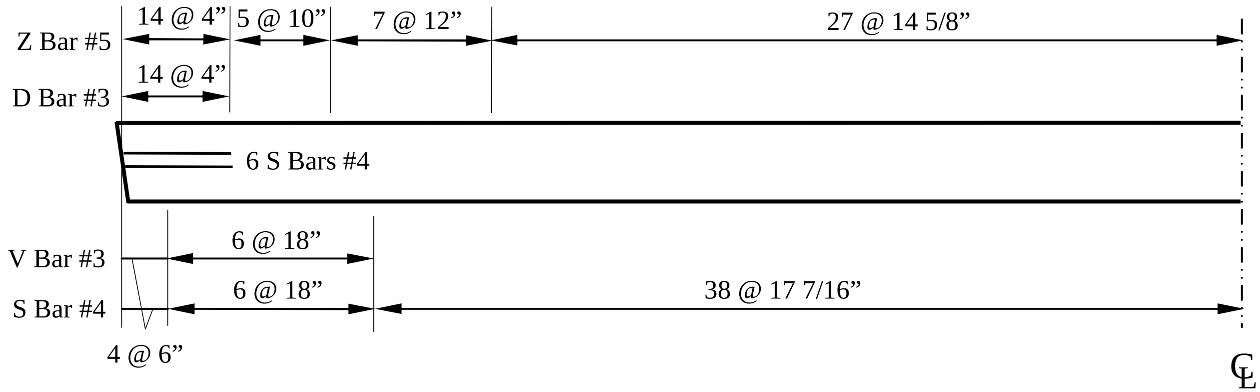



**Figure 3-10: Mild Steel Spacing in BT-54 Girders**

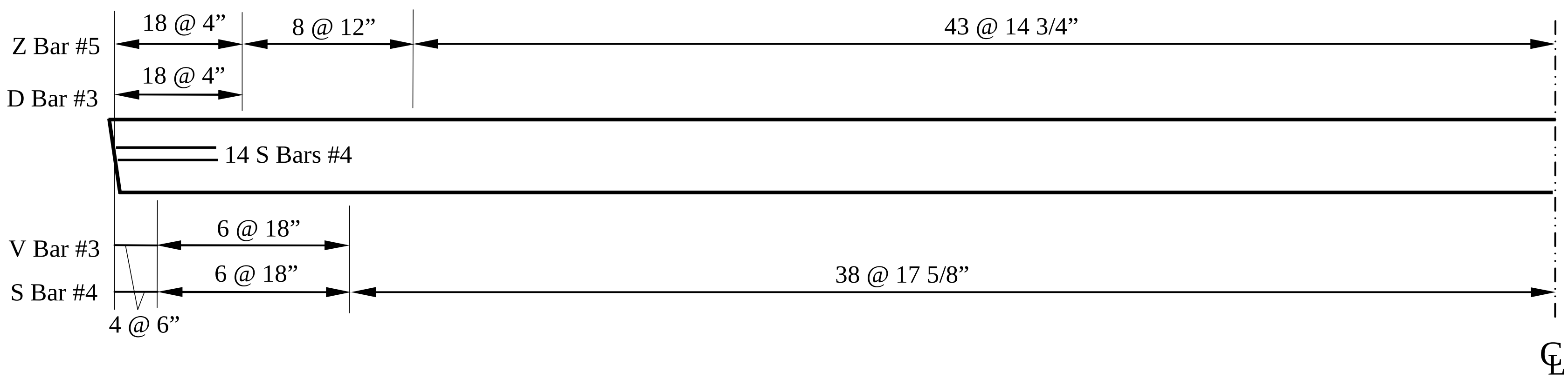



**Figure 3-11: Mild Steel Spacing in BT-72 Girders**

Given the large spalling and bursting stresses that result from anchorage of the prestressing forces, the mild steel was heavily concentrated in the ends of the girders in order to counteract the effects any cracking in the region may induce. This can be seen in the fact that the spacing of the steel in the girder ends is much denser than at midspan, as well as the fact that the bottom confining steel and the V-bars are only located in these regions, as can be seen in Figure 3-12.

**Figure 3-12: End of Span Steel Configuration**

### 3.4 Material Properties

The material properties of all the components used in the construction of the bulb tee girders are discussed in this section. The materials include CVC, self-consolidating concrete, prestressing strand, and nonprestressed reinforcement.

### 3.4.1 Concrete

Two different concrete mixtures were used in the fabrication of the bulb-tee girders, all of which was mixed on site at Hanson Pipe & Precast prestressing plant in Pelham, Alabama. The concrete mixtures were designed by the contractor to satisfy the special provision for the prestressed bridge girders, which can be seen in Appendix B. The two concrete mixtures included CVC and SCC, both of which included Type III portland cement. All of the admixtures were supplied by W.R. Grace and included an air-entraining admixture (Darex AEA EH), a hydration-stabilizing admixture (Recover), a viscosity-modifying admixture (V-Mar 3), and a high-range water reducing admixture (ADVA Cast 575). The admixtures are presented in terms of ounces per cubic yard. Besides differences in the amounts of admixtures added, the SCC contained #78 limestone, whereas the CVC contained #67 limestone. Summaries of the components used in each mixture can be seen in Table 3-1. After casting the BT-54 girders, the plant personnel felt that the SCC mixture could be improved upon, which was reflected by the slight variation in the BT-72 SCC mixture, which included double the amount of VMA.

**Table 3-1: Summary of Concrete Mixtures**

| Item | BT-54 SCC | BT-54 CVC | BT-72 SCC | BT-72 CVC |
|---|---|---|---|---|
| **Water Content (pcy)** | 266 | 238 | 265 | 234 |
| **Cement Content (pcy)** | 758 | 696 | 760 | 708 |
| **GGBF Slag Content (pcy)** | 134 | 124 | 135 | 125 |
| ***w/cm*** | 0.30 | 0.29 | 0.30 | 0.28 |
| **SSD Coarse Agg. #78 (pcy)** | 1528 | 0 | 1550 | 0 |
| **SSD Coarse Agg. #67 (pcy)** | 0 | 1923 | 0 | 1950 |
| **SSD Fine Agg. (pcy)** | 1384 | 1163 | 1370 | 1179 |
| ***s/agg* (by weight)** | 0.48 | 0.38 | 0.47 | 0.38 |
| **Air-Entraining Admixture (oz/cy)** | 0.3 | 0.3 | 0.2 | 0.2 |
| **HRWR Admixture (oz/cy)** | 11 | 8 | 11 | 7 |
| **Viscosity-Modifying Admixture (oz/cy)** | 2 | 0 | 4 | 0 |
| **Hydration-Stabilizing Admixture (oz/cy)** | 2 | 1 | 2 | 1 |
| **Total Air Content (%)*** | 4.1 | 4.2 | 4.0 | 3.2 |

*Average of air content determined from fresh test results.

It was necessary to batch large quantities of each mixture during any given girder placement, thus, a representative sample were taken at the beginning, middle, and end of the casting process for each mixture each day. Fresh concrete properties were tested, the results of which can be seen in Table 3-2. Traditional data were collected for the CVC mixtures, including the slump, air content, and unit weight of the concrete. Additional properties collected for the SCC included the slump flow, visual stability index (VSI) rating, and T-50 test result. The slump and slump flow values reported were from tests performed by Auburn University researchers for research purposes only, while the slump and slump flow values taken for concrete acceptance purposes were not included. Occasionally, the first batch of concrete was rejected by ALDOT inspectors because of test results that were outside of the acceptable ranges which are defined in Appendix B.

**Table 3-2: Fresh Concrete Properties**

| Beam | Sample No | Unit Weight (lbs) | Slump (in.) | Slump Flow (in.) | Air (%) | T50 (sec.) | VSI |
|---|---|---|---|---|---|---|---|
| 54-2S | 1 | 149.1 | - | 28.00 | 3.3 | - | 1.5 |
| | 2 | - | - | 27.50 | 4.4 | - | 1.0 |
| | 3 | - | - | 26.00 | 4.5 | - | 1.0 |
| 54-4S | 1 | - | - | 27.00 | 2.6 | 7 | 1.0 |
| | 2 | - | - | 26.00 | 3.0 | 6 | 1.0 |
| | 3 | - | - | 27.00 | 4.6 | 8 | 1.0 |
| 54-7S | 1 | - | - | 26.00 | 5.5 | 7 | 1.5 |
| | 2 | - | - | 26.00 | 4.2 | 8 | 1.5 |
| 54-8C | 1 | - | 9.00 | - | 4.5 | - | - |
| | 2 | - | 8.75 | - | 3.9 | - | - |
| 54-11C | 1 | - | 8.50 | - | 4.2 | - | - |
| | 2 | - | 9.00 | - | 4.5 | - | - |
| | 3 | - | 8.75 | - | 4.4 | - | - |
| 54-13C | 1 | 152.3 | 9.00 | - | 3.9 | - | - |
| | 2 | 153.2 | 10.00 | - | 4.0 | - | - |
| | 3 | - | 8.75 | - | 4.0 | - | - |
| 72-2S | 1 | 149.8 | - | 22.50 | 4.2 | 9 | 1.0 |
| | 2 | - | - | 24.00 | 3.7 | 10 | 1.0 |
| | 3 | - | - | 22.00 | 3.8 | 15 | 1.0 |
| 72-4S | 1 | - | - | 26.00 | 3.3 | 8 | 1.0 |
| | 2 | - | - | 26.00 | 4.3 | 9 | 0.0 |
| | 3 | - | - | 23.00 | 4.8 | 14 | 0.0 |
| 72-7S | 1 | 150.1 | - | 25.00 | 3.7 | 10 | 0.0 |
| | 2 | - | - | 23.00 | 4.5 | 10 | 0.0 |
| | 3 | - | - | 24.00 | 3.8 | 11 | 0.0 |
| 72-8C | 1 | - | 8.50 | - | 4.0 | - | - |
| | 2 | - | 9.00 | - | 4.3 | - | - |
| | 3 | - | 8.75 | - | 3.5 | - | - |
| 72-11C | 1 | 153.4 | 8.50 | - | 3.6 | - | - |
| | 2 | - | 9.00 | - | 3.1 | - | - |
| | 3 | - | 9.00 | - | 3.5 | - | - |
| 72-13C | 1 | - | 9.00 | - | 3.1 | - | - |
| | 2 | - | 9.00 | - | 2.5 | - | - |
| | 3 | - | 9.25 | - | 3.1 | - | - |

During casting, representative 6-inch by 12-inch concrete sample cylinders were made. As the age of the concrete reached different benchmarks, the cylinders were tested to determine the concrete properties at the time in question. The averaged results of the hardened concrete properties for each girder can be seen in Table 3-3, as well as the amount of time that had elapsed between the placement of the concrete and the prestressed strand release the following day. In addition, the required strengths that had to be obtained for the girders of each size are also shown.

**Table 3-3: Hardened Concrete Properties**

| | **Release** | | | **28-Days** | |
|---|---|---|---|---|---|
| **Beam** | **Age (hrs)** | **$f'_{ci}$ (psi)** | **$E_{ci}$ (ksi)** | **$f'_c$ (psi)** | **$E_c$ (ksi)** |
| 54-2S | 24 | 9010 | 6200 | 10240 | 6400 |
| 54-4S | 24 | 8680 | 6300 | 10800 | 6600 |
| 54-7S | 24 | 7940 | 6100 | 10180 | 6200 |
| 54-8C | 25 | 8760 | 6400 | 10360 | 6800 |
| 54-11C | 24 | 7860 | 6700 | 9670 | 6900 |
| 54-13C | 23 | 8790 | 7100 | 10590 | 7400 |
| Required BT-54 | - | 5200 | 6000 | - | - |
| 72-2S | 22 | 8220 | 5800 | 10550 | 6400 |
| 72-4S | 19 | 7860 | 5900 | 10770 | 6400 |
| 72-7S | 24 | 8120 | 5800 | 10490 | 6300 |
| 72-8C | 23 | 8290 | 6700 | 10770 | 7000 |
| 72-11C | 20 | 8320 | 6800 | 11050 | 7700 |
| 72-13C | 22 | 8770 | 7100 | 10850 | 7300 |
| Required BT-72 | - | 5800 | 8000 | - | - |

The concrete strength at prestress release was significantly higher than the required strength specified for both the BT-54 girders and the BT-72 girders. This could be a function of the strands not being released until approximately 23 hours after concrete placement as opposed to release occurring approximately 18 hours after placement which is closer to an industry standard. A summary of both the fresh and hardened concrete properties can be seen in Table 3-4.

**Table 3-4: Summary of Concrete Properties**

| | | SCC | | CVC | |
|---|---|---|---|---|---|
| | | **Range** | **Average** | **Range** | **Average** |
| **Fresh Properties** | **Unit Weight (lbs)** | 149.1-150.1 | 149.7 | 152.3-153.4 | 153.0 |
| | **Slump (Flow) (in.)** | 22.0-28.0 | 25.2 | 8.5-10.0 | 8.9 |
| | **Air (%)** | 2.6-5.5 | 4.0 | 2.5-4.5 | 3.8 |
| | **T50 (sec)** | 6-15 | 9.5 | - | - |
| | **VSI** | 0.0-1.5 | 1.0 | - | - |
| **Hardened Properties** | **Age at Release (hrs)** | 19-24 | 23 | 20-25 | 23 |
| | **$f'_{ci}$ (psi)** | 7860-9010 | 8310 | 7860-8790 | 8460 |
| | **$f'_c$ (psi)** | 10180-10800 | 10510 | 9670-11050 | 10550 |
| | **$E_{ci}$ (ksi)** | 5800-6300 | 6020 | 6400-7100 | 6800 |
| | **$E_c$ (ksi)** | 6200-6600 | 6380 | 6800-7700 | 7180 |

### 3.4.2 Prestressing Strand

The strand used in this investigation was low-relaxation, Grade 270, seven-wire prestressing strand. Two different sizes were used: the BT-54 girders contained ½-inch strand, where as the BT-72 girders incorporated ½-inch ‘special’ strand. All of the strand was stored outside using normal practices for ALDOT girders and exhibited some slight weathering effects. The typical surface condition of the strand can be seen in Figure 3-13. The steel was supplied by two different strand providers. The ½-inch strand was provided by Strand-Tech Martin, Inc. out of Summerville, South Carolina. The ½-inch ‘special’ strand was provided by American Spring Wire out of Houston, Texas: the same type and manufacturer as the strand used for the previous phases of prestressed SCC research at Auburn University.

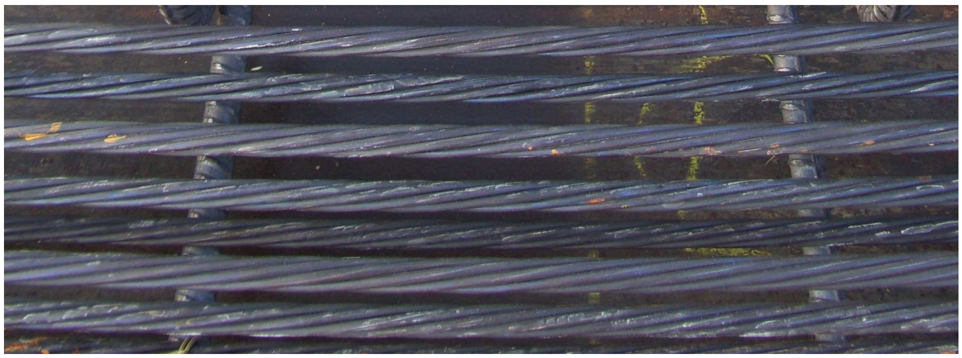


**Figure 3-13: Typical Surface Condition of Prestressing Strand**

Strand pullout testing was employed to determine a quantifiable estimate of the relative bond quality of the strands prior to construction of the girders. After testing both the ½-inch and the ½-inch ‘special’ strands, it was determined that the strands met the acceptability criteria specified by Logan (1997) without exhibiting excessively strong bond capacity. This was a desirable result as it allows for the transfer length behavior measured in this study to be representative of what could be expected from strands that have acceptable, but not superior, bond capacity. A description and the results of the pullout tests are detailed in Appendix C.

### 3.4.3 Nonprestressed Steel Reinforcement

Nonprestressed, mild steel reinforcing bars were used to strengthen the section in relation to shearing and anchorage forces and to limit the size of any associated cracks. The steel used was ASTM A615 Grade 60 reinforcing bars. As mentioned earlier, four different shapes were used, each of which can be seen in Figure 3-4 and Figure 3-6.

Additionally, the spacing of these bars can be seen in Figure 3-10 and Figure 3-11. The bars ranged in size from #3 bars to #6 bars.

## 3.5 Specimen Fabrication

The twenty-eight girders were fabricated for State Route 22 over Hillabee Creek in Tallapoosa County, Alabama at Hanson Pipe and Precast prestressing plant in Pelham, Alabama in September and October 2010. However, transfer behavior was studied on only twelve of the twenty-eight girders. While two different sized girders were created, the process followed to cast a girder of either size is identical and is described in the following section.

### 3.5.1 Casting Configuration of Precast, Prestressed Bridge Girders

Each of the girders was cast on one of two lines utilized for the project in the prestressing plant. Due to the length of the casting beds, it was possible to cast three BT-54 girders on one line, with a single line being cast in one day. The bed layout for a typical single casting day of BT-54 girders can be seen in Figure 3-14. Two days of casting with CVC were completed as well as two days of casting using SCC, creating twelve of the fourteen BT-54 girders needed for the bridge. To complete the castings required for the BT-54 girders, a single casting of only two girders on the line was done to create the thirteenth and fourteenth girders, one girder being cast with CVC and the other with SCC. In this case, the CVC girder was cast first so that the vibration used on the girder would not affect the SCC girder. The girder layout for the last day of casting for the BT-54 girders can be seen in Figure 3-15.


**Figure 3-14: Casting Configuration of Three BT-54 Girders**


**Figure 3-15: Casting Configuration of Two BT-54 Girders**

Given the extra length associated with the BT-72 girders, it was only possible to cast two girders on a line, with this configuration being seen in Figure 3-16. This resulted in three days of casting with CVC and three days of casting with SCC. To fulfill the number of girders, one last day of casting was done with one beam each of CVC and SCC.

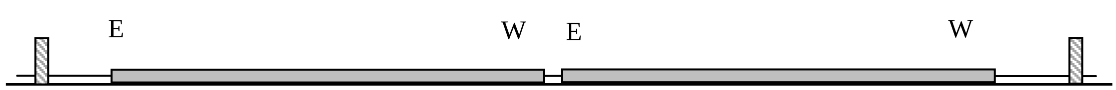



**Figure 3-16: Casting Configuration of BT-72 Girders**

Throughout this process, only a single beam per casting was designated as a specimen for use within the study. The one exception was both beams from the final day of BT-54 casting, which consisted of one CVC girder and one SCC girder, were utilized.

Casting dates for the girders spanned from September 21, 2010 to October 28, 2010. The BT-54 girders were cast first, followed by the BT-72 girders.

### 3.5.2 Fabrication of Precast, Prestressed Bridge Girders

Normal plant casting procedures and protocol were followed since the girders cast during the duration of the study were for use in a bridge, not strictly for research. Given the size and type of girders needed, the prestressing bed was laid out with the proper components required to cast the appropriately sized girder. Once this was done and the bed was cleaned, strand was pulled in the correct configuration through the headers and the numerous hold-down points (for draped strands). The hold-up points were then raised to the proper elevation so that the draped strands would acquire the correct draping configuration once the strands were tensioned.

The strands were pulled to the specified jacking stress using a hydraulic jack. Each strand was partially stressed before pulling it to its final tension, allowing an opportunity to discover a major flaw in the strand prior to a sudden failure. Strand tensions and elongations were checked according to standard ALDOT-mandated procedures.

Once the strands were in the correct location and tensioned to the appropriate force, plastic sheaths were added to designated strands for debonding and tied into place. The strands designated as debonded strands can be seen in Figure 3-4 and Figure 3-6. A picture of the installed sheaths can be seen in Figure 3-17. These strands were debonded for a designated length of 10 ft from the end of the girder in both girder sizes. Finally, the mild steel reinforcing cage was built around the strands. This was done by tying the pre-bent deformed rebar into the correct locations and tying the bars and strands together.

The bed of the forms was then oiled by squirting the oil underneath the strands. This was done very carefully to prevent the oil from coming into contact with the strands and compromising the integrity of the bond.

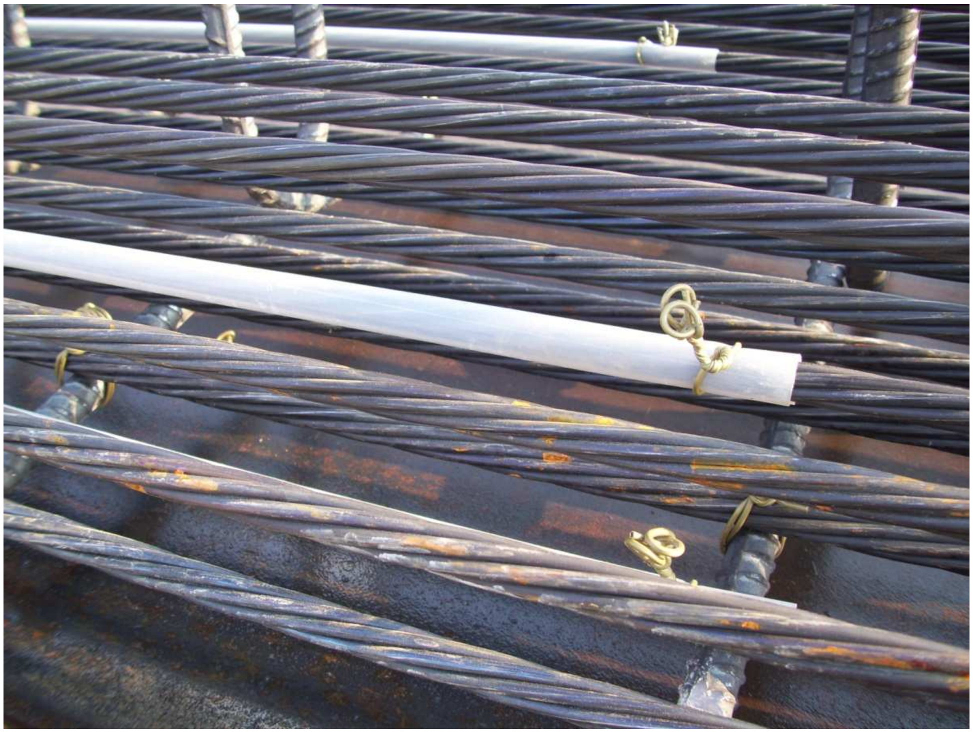


**Figure 3-17: Plastic Sheathing on Debonded Strands**

As fully described in Chapter 4, a new system of mounting the demountable, mechanical (DEMEC) points, used in measuring the surface compressive strains of the concrete in order to determine the transfer lengths, for the study was created. The new system required a portion of the system to be cast into the girder. Consequently, the pre-fabricated portion of the DEMEC bars was installed into the steel cage at six designated locations prior to the formwork being moved into place. Section 4.2.2 gives a complete description of this process. A bar tied to the prestressing strands can be seen in Figure 3-18.

**Figure 3-18: DEMEC System Tied into Steel Cage**

Before the forms were put into place, they were sprayed with a form release agent that allowed for easy removal of the forms once the concrete was hardened. It was imperative to ensure the release agent did not come into contact with the mild steel or the prestressing strands. If this were to happen, the form oil would prevent the steel from becoming properly bonded to the concrete, the prestressing would be rendered ineffective, and the girder would be substandard and unsafe. The side forms were then put into place and secured. The beginning of this process can be seen in Figure 3-19.

**Figure 3-19: Formwork Being Put into Place**

All of the concrete was batched on site and transported from the batching area to the prestressed beds by a concrete delivery vehicle which could carry 4 cubic yards of concrete. Figure 3-20 shows one of the vehicles placing concrete into the formwork. Numerous trips were required from the batching area to the prestressing line in order to fill the formwork. While the trucks were transporting the concrete, a truck was occasionally pulled aside in order to test the fresh concrete to ensure its properties met the project specifications, shown in Appendix B, as well as to sample the concrete.


**Figure 3-20: Concrete Delivery Vehicle Placing Concrete**

The process used to place the CVC and the SCC was different given the different physical characteristics of each concrete. To place the CVC, the concrete delivery vehicle would begin placement at one end of the girder and release discrete amounts of concrete before inching forward and releasing more concrete. The concrete was very viscous and deliberate placement was required as the concrete did not easily move once placed and care had to be taken to not overfill the form at any one location. Figure 3-21 shows this as the concrete can be seen building up behind the vertical bars preventing the concrete from moving into the next area. Another concrete delivery would follow behind the first, placing concrete to fill the formwork.

**Figure 3-21: CVC Placement**

It was also necessary to use vibration for the CVC placement  in order to ensure that all the air pockets were eliminated that may have formed in the girder.  This was done in two ways.  First, an external vibrator was mounted in a track on the side of the formwork that ran along the length of the girder, which is visible in Figure 3-22.  As the concrete was placed, the vibrator was moved along the track, show in Figure 3-23, appropriately.  Second, the workers also used internal vibration, as can be seen in Figure 3-24.

**Figure 3-22: External Vibrator**

**Figure 3-23: External Vibration Track**

**Figure 3-24: Internal Vibration of Concrete**

The SCC was much easier to place. As with the CVC, the concrete delivery vehicle began placement at one end of the girder. As the SCC was released, it would flow into the formwork and through the steel cages under its own weight and without the need for internal vibration. Care still had to be taken as the rate at which the SCC flowed was slower than the rate at which the concrete was dispensed from the delivery truck via an auger-driven chute. As a result, the concrete delivery vehicle was still required to move along the length of the girder. However, the SCC was still easier to place as the trucks did not need to move near as often, nor be as precise in where along the girder the concrete was placed. It was also safer for the workers as individuals were not on top of the form work applying internal vibration. Figure 3-25 shows the concrete that has almost filled the web from a previous placement, as well as concrete from a more recent placement flowing from the discharge point to fill the web and starting to fill the flange.

**Figure 3-25: Placement of Self-Consolidating Concrete**

In the two situations in which a conventional girder and a SCC girder were cast on the same line on the same day, which occurred once for the set of BT-54 girder castings and once for the BT-72 girders, the conventional girder was always cast first.  As the formwork was continuous along the entire length of the bed, any vibrations applied to the conventional girder would have some effect on the entire bed.  Placing the CVC and vibrating it before the placement of the SCC guaranteed that the SCC would not receive any vibrations that could affect the resulting girder.

After all the concrete had been placed, the top surface was roughened and any accessories required for the particular girder were added. The surface was roughened to approximately ¼ in. by running a type of metal rake with several fingers across the wet concrete as seen in Figure 3-26. However, given that the SCC would not keep a roughened surface since the concrete would just reconsolidate after being raked, the SCC girders were not raked until the top concrete surface had started to set slightly to allow it to retain the roughened surface. Finally, a curing blanket and a weatherproof tarp were draped over the bed and forms, pictured in Figure 3-27, to help the concrete to cure and was left overnight. In addition, all of the girders were steam cured.

**Figure 3-26: Concrete Surface Roughening**

**Figure 3-27: Curing Blankets and Placement of Weatherproof Tarp**

The next day, approximately 23 hours after the concrete had been placed, the curing blankets were rolled up, and the side forms removed. Figure 3-28 shows the removal process. The final steps required to prepare the DEMEC strain-measurement system, which is described in detail in Chapter 4, were completed and the actual distance from the end of the girder to the center of the first DEMEC point was measured to the nearest quarter inch. Finally, an initial set of measurements were taken, the process of which is also included in Chapter 4.

**Figure 3-28: Removal of Side Forms**

After representative concrete cylinders were tested to ensure the concrete had reached the required strength for release, the prestressing force was transferred to the girders by flame cutting the strands, which can be seen in Figure 3-29. This required several workers, one located at the far east and west ends of the bed, as well as a man at every space in between girders. The men then proceeded to cut the strands. The order the strands were cut was predefined and, after each cut, the workers checked to make sure each cut for a single strand was made before continuing to the next strand. Diagrams of this sequence for both the BT-54 girders and the BT-72 girders can be seen in Figure 3-30 and Figure 3-31. The strands were flame cut in the following order: the very bottom outside strand on either side, the four top strands, the top two draped strands, the hold-down points, each remaining draped strand working from top to the bottom of the draped portion, and the bottom strands working from the top to the bottom and from the outsides in.

**Figure 3-29: Flame Cutting**

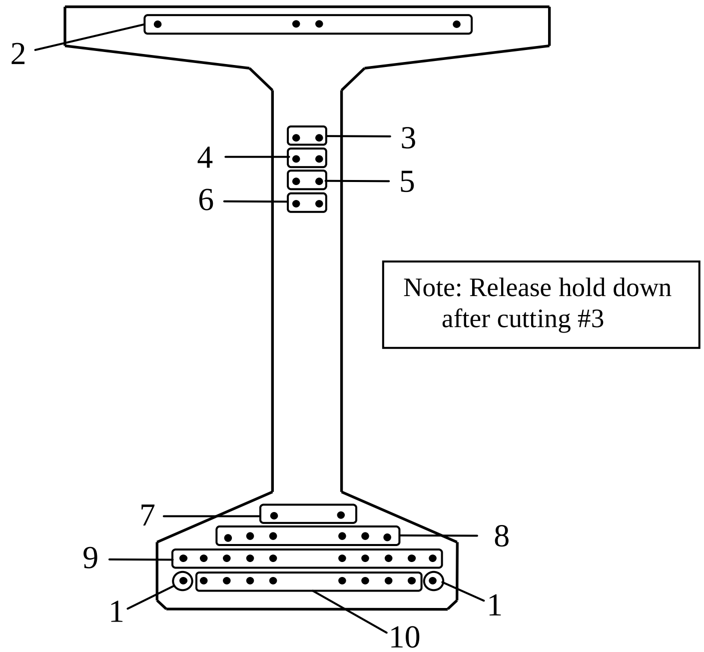



**Figure 3-30: Flame Cutting Sequence for BT-54 Girders**

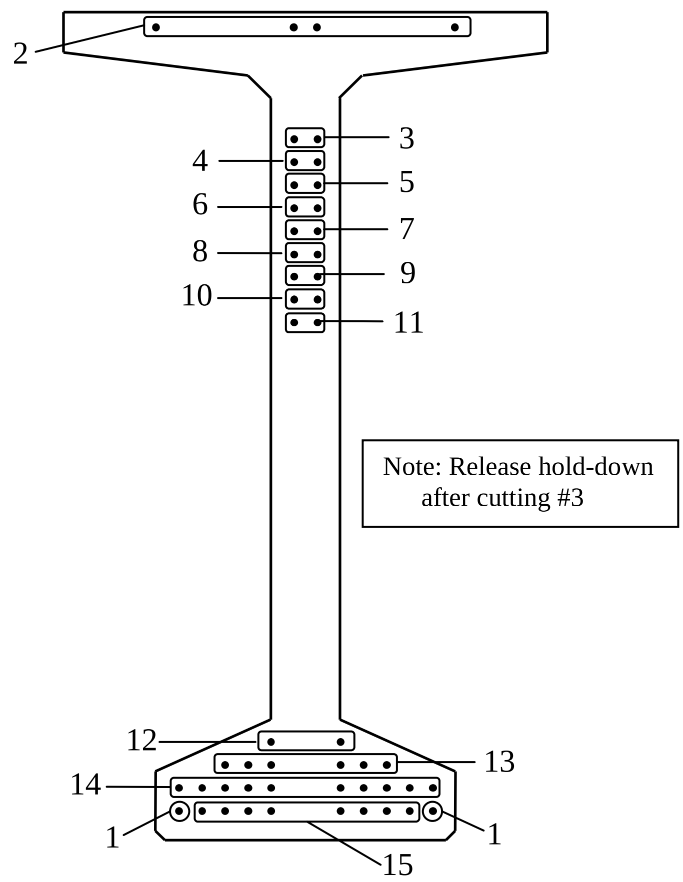



**Figure 3-31: Flame Cutting Sequence for BT-72 Girders**

Once the strands had been released, a second set of DEMEC measurements were taken before the girders were moved into the storage yard by large mobile cranes.  A crane carrying a BT-54 girder into the storage area is shown in Figure 3-32.  When moving the BT-72 girders, two cranes were required due to the longer length of the BT-72 girders.  The girders were then stored in the yard, having been placed on supports that were configured to resemble the final support conditions within the bridge.  The support conditions and end of three BT-54 girders in the storage yard can be seen Figure 3-33.  The girders were kept in the yard until they were transported to the bridge site for construction.


**Figure 3-32: Transporting BT-54 Girder to the Storage Yard**


Figure 3-33: Girder Support Conditions in Storage Yard

# Chapter 4 Transfer Length Test Program

## 4.1 Introduction

Twelve PCI bulb-tee girders were studied and analyzed with respect to their transfer lengths. Each girder had four transfer zones, one at each end of the girder and one at each end associated with the transfer zone of the debonded strands. Each end transfer zone was examined for all twelve girders, along with one of the two debonded-strand transfer zones for ten of the twelve girders. Consequently, there were thirty-four resulting transfer zones: twenty-four end transfer zones and ten debonded-strand transfer zones. Transfer lengths were determined by analyzing the concrete surface strains measured in each transfer zone. A demountable, mechanical (DEMEC) strain gauge was used to measure the concrete surface strains. Strains were measured immediately after prestress transfer, as well as at 7 days and 28 days after transfer in order to capture any time-dependant changes in the transfer length. The measurement techniques are described in this chapter.

## 4.2 Test Procedure

Given the large scale of the specimens being tested and the fact that testing would occur on the premises of the precast plant facility, a new method and system for mounting the demountable, mechanical (DEMEC) points was created that allowed for the DEMEC points to be inserted into the concrete rather than having to be glued to the surface. Inspiration for the instrumentation adapted for use within this study was inspired by similar methods utilized by Ben Graybeal of the Federal Highway Administration and Canfield (2005).  This new method required significant preparations before casting, but allowed for a reduction in both the time needed and the number of procedural steps required between form removal and prestress transfer—minimizing disruption of normal plant production procedures.

### 4.2.1 Cast-In-Place DEMEC Mounting System

The DEMEC mounting system was fabricated and constructed to allow the DEMEC targets to be easily and quickly mounted onto the concrete girder specimen.  It was also important to create a system that could be easily installed at the plant.  The following section details the DEMEC mounting system that was utilized in the plant.

The DEMEC bars each consisted of a 72 in. x 1 in. metal strip with thirty-four holes spaced at 50 mm on center.  Each of the holes was countersunk at 82 degrees to accommodate a machine screw.  Five larger holes were then countersunk at each end and at equal intervals along the bar in between the holes previously made.

The bars were assembled by placing a screw into each of the thirty-four smaller holes.  A drop of lubricant was then placed on the screw before a washer and a threaded insert were attached to it and tightened until it was finger-tight.  Larger screws were then inserted into the five larger holes, lubricant added, a washer, coupling nut, and threaded rod attached until these too were finger-tight.  The assembled hardware can be seen in Figure 4-1 with the entire bar shown in Figure 4-2.

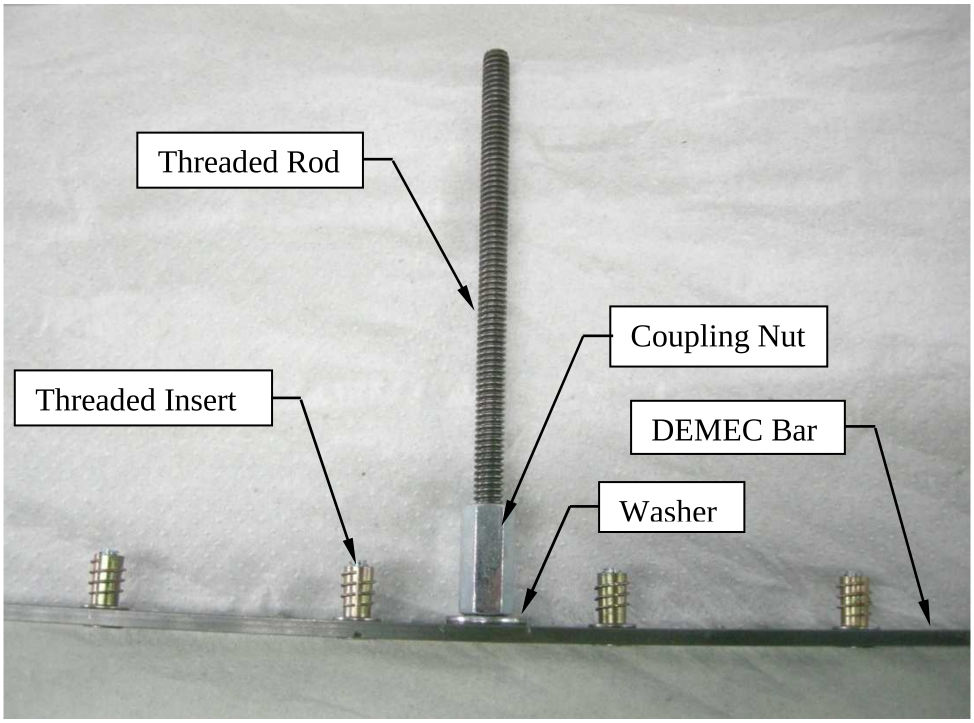



**Figure 4-1: Assembled DEMEC Mounting System**

**Figure 4-2: Fully Assembled DEMEC Mounting System**

The intent behind the design of the mounting system was to devise a way to have the threaded inserts cast into the concrete at the locations where DEMEC measurements were desired.  The threaded rods were added to the system to provide a way to tie the bar into the steel cage prior to casting.  Once all of the hardware had been assembled, a strip of quick-recovery super-resilient polyurethane foam with an adhesive backing was placed over the heads of the screws and covered the entire length of the bar.  The foam was added to prevent concrete from coming into contact with the screw heads, as well as to provide a barrier between the formwork and the bar, preventing the bar from being embedded completely within the concrete and becoming inaccessible.

It was also necessary to fabricate specialized DEMEC target screws.  This was done by drilling a small hole in the center of hex head screw to ensure the proper positioning of the conical points of the DEMEC gauge.  A fabricated DEMEC screw is shown in Figure 4-3.

**Figure 4-3: DEMEC Screw**

### 4.2.2 Specimen Preparation

Each girder had six segments along which measurements were taken with each segment being 5.5 feet in length to ensure the entire transfer length was captured within the segment.  These segments were located on the bottom flange of each girder and included each end of the girder, as well as a segment starting 9 feet from the mark end of the girder to capture the transfer length of the debonded strands.  Thus, there were three transfer zones measured, each with a segment on each girder face.  These segment configurations can be seen in Figure 4-4.

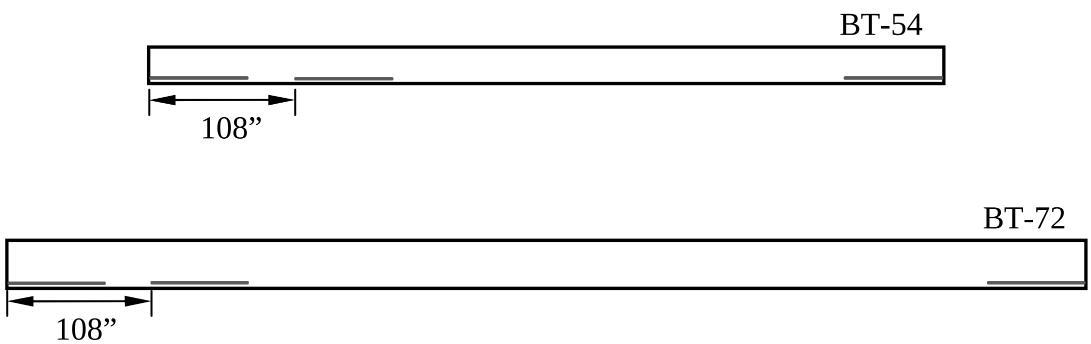



**Figure 4-4: Location of Areas Analyzed for Transfer Length, BT-54 and BT-72 (Mirrored on Opposite Face)**

The pre-fabricated portion of the DEMEC bars was installed into the steel cage and can be seen in Figure 4-5.  Each bar was anchored so that the DEMEC strip stood slightly outside the edge of the bed.  Consequently, when the forms were moved into place prior to the concrete placement, the final location of the DEMEC bar would be snug against the form, also helping to prevent the bar from becoming embedded in the concrete.

**Figure 4-5: Final Location of DEMEC Bar Prior to Casting**

Once the concrete had cured overnight, the tarps and formwork were removed. At this point, the DEMEC strips could be seen on the outside face of the bottom flange of the bulb tee girder, as seen in Figure 4-6. Despite all efforts, on occasion it was possible for some paste to seep in between the foam and the forms. When this occurred, the thin paste was easily knocked away with a hammer. It was then possible to remove the foam, exposing the heads of the screws that had been used to hold the threaded inserts prior to hardening of the concrete. This can be seen in Figure 4-7.

**Figure 4-6: Exposed DEMEC Strip Immediately After Form Removal**

**Figure 4-7: Removal of Foam Adhesive**

Each screw was removed with a power drill, including the screws associated with the threaded anchors, and discarded. Doing this allowed the metal strip to be pried away from the concrete, as seen in Figure 4-8, leaving a smooth surface with a threaded insert at every location a DEMEC target was needed. At this point, the prefabricated DEMEC target screws could be screwed into place. The beginning of this process is shown in Figure 4-9. Finally, each DEMEC target was labeled with odd numbers from one to sixty-seven beginning with the point closest to the end of the girder. Since each point was located 50 mm, or approximately 2 inches, from the next point, these labels represented the approximate distance the DEMEC point was from the end of the girder, in 25 mm increments. A finalized DEMEC segment can be seen in Figure 4-10. As a result, the DEMEC targets could be measured at any time of interest.

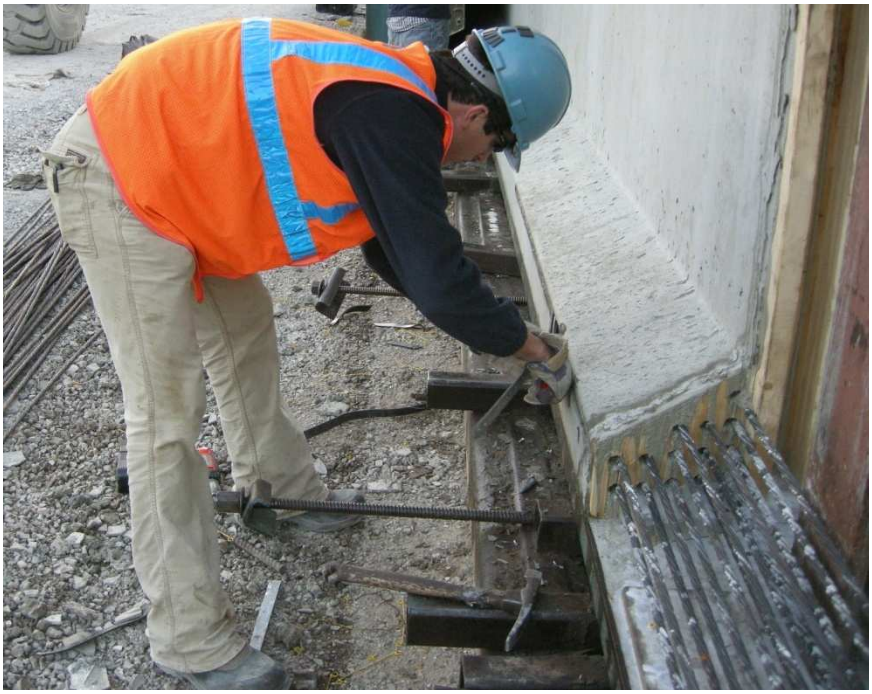


**Figure 4-8: Removal of Metal DEMEC Bar**

**Figure 4-9: DEMEC Strip after Metal Bar Removed and DEMEC Points Started**

**Figure 4-10: Finalized DEMEC Target Installation**

### 4.2.3 Concrete Surface Strain Measurement

The process for reading the DEMEC targets with the gauges was consistent throughout the entire project. Before any of the points were measured, an initial reference reading was taken of an Invar reference bar corresponding with the gauge in use and was recorded. The process of measuring the DEMEC points was then started. A 200 mm DEMEC strain gauge was used to read the points, and can be seen in Figure 4-11, along with the reference bar.


**Figure 4-11: DEMEC Strain Gauge and Reference Bar**

To measure the distance between the points, the moveable conical point of the gauge was inserted in the target on the right-hand end of the 200 mm span. The fixed conical point was then inserted into the left-hand target, and a reading was taken. The left-hand, fixed point was then removed, reinserted into the same target, and the gauge read again. If the second reading was within one gauge divisions (0.0016 mm of surface displacement) of the first, both measurements were recorded. Consequently, each 200 mm interval was read twice to confirm repeatability in the measurement. The gauge was then moved to the next successive target and the process was repeated. An image of a reading being taken can be seen in Figure 4-12. Once the measurement had been completed, due to the overlapping nature of the measurements, a single 50 mm interval was included within four overlapping 200 mm gauge lengths.  All six segments were read to complete a set of measurements for a girder.


**Figure 4-12: Performing DEMEC Strain Measurement**

Several different sets of measurements were taken at different ages throughout the early life of the girders, as previous studies indicated that the transfer length grew for several days after prestress transfer.  The first set of measurements was taken immediately after the forms were removed to serve as a benchmark for the undeformed state of the girder.  Measurements were also taken immediately after prestress transfer, 7 days after transfer, and 28 days after transfer, at which point it was determined that the transfer length had stopped changing significantly.  The 7- and 28-day strains were measured while the girders were in their storage positions in the precast plant.  During storage, the girders were supported so as to have the same span length that would be experienced in the actual bridge. The recorded strains were analyzed to determine the transfer length associated with each zone.

### 4.3 Determination of Transfer Length

Once the data had been collected for the thirty-four transfer zones contained within the twelve PCI bulb-tee girders, it was analyzed to acquire the appropriate information needed to determine the transfer length for each zone. The raw measurements were converted into concrete surface strains that could be plotted and the 95% average maximum strain (AMS) plateau was found for each concrete age. The transfer length was measured by determining the distance to the intersection of the 95% AMS plateau and the strain profile, as can be seen in Figure 4-13. The detailed process through which this was done is described in the following sections.

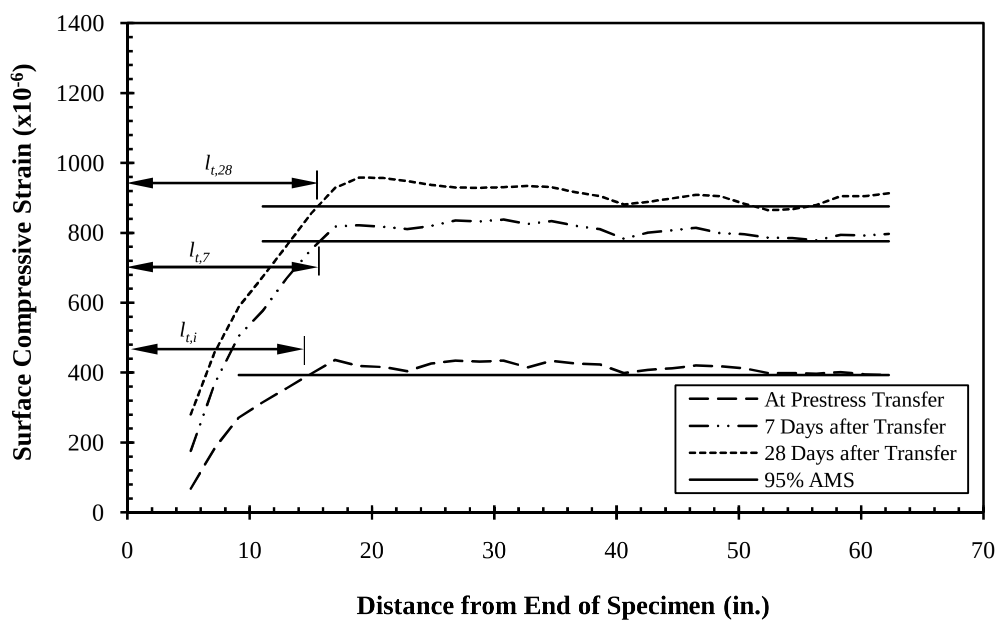



**Figure 4-13: Compressive Strain Profile**

### 4.3.1 Construction of Surface Compressive Strain Profiles

The first step required to create a surface compressive strain profile for any of the girders was to transform the measurements collected at the prestressing facility into strain values. The following steps were followed to perform this conversion.

1. As some deformation measurements varied by as much as 0.0016 mm between the first and second reading, the average reading was determined at each location at each age.
2. A reference reading was taken prior to taking the surface DEMEC target measurement for every age throughout the study. The reference reading for each age was subtracted from the surface DEMEC target reading at each location for the same age. The resulting difference is described as the "relative reading" in the next step.
3. The change in compressive strain at each location corresponding to the age in question was determined by subtracting the relative reading prior to transfer from the relative reading at the age of interest. The resulting difference was then multiplied by the appropriate gauge factor to determine the strain over the 200 mm gauge length. At this stage, each measured strain was assigned to the absolute position of the DEMEC point at the middle of the 200 mm gauge length measured.
4. As mentioned previously, the girders were skewed at 15 degrees. Consequently, the resulting strains were determined for the center line of the girder by averaging the strains from opposite faces of the girder at corresponding points (point 1 on the north face was averaged with point 1 of

the south face). The distance from the end of the girder of each point was also averaged with the corresponding distance of the point on the other face to determine the midpoint between the two points, which can then be used to determine the distance from the end of the girder along the centerline for the resulting averaged strain.

5. The strains were then smoothed: a single strain value was assigned to each distance along the centerline by averaging the strain assigned to a particular location with the strains assigned to the immediately adjacent locations. A visual aid depicting the smoothing portion of this process can be seen in Figure 4-14.

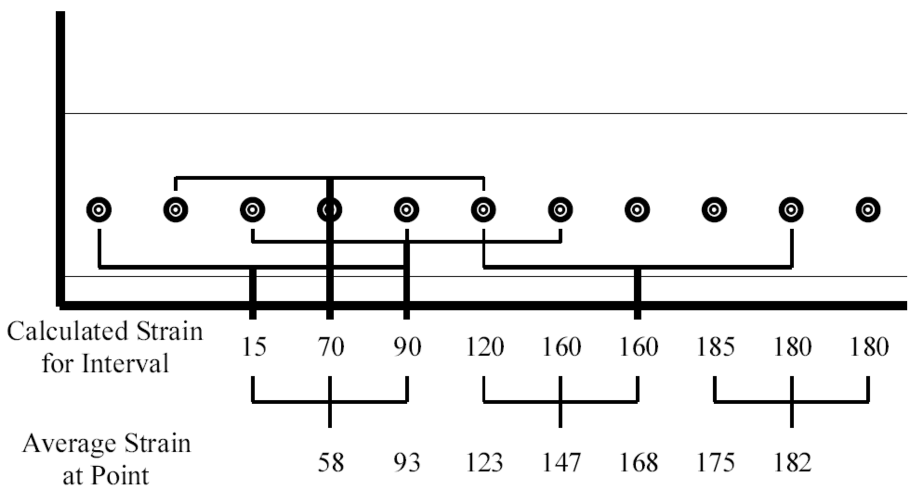



**Figure 4-14: Assignment of Surface Compressive Strain Values**
(Barnes, Burns, and Kreger 1999)

6. The smoothed strain values were then plotted in relation to their absolute values. The resulting graph depicts the concrete strain along the centerline of the girder.

In previous cases in which large specimens were tested for transfer length, as with the specimens being tested by Barnes, Burns, and Kreger (1999), adjustments needed to be made to account for the self-weight of the girder. When eccentrically placed prestressing strands are released, it causes the member to bend upwards. As a result, the self-weight of the beam induces strains that are superimposed on the strains that result from the prestressing alone. The self-weight strains vary along the length of the member and make it difficult to precisely identify the plateau strain that indicates the end of each transfer length. In order to ensure that the transfer lengths were accurately determined, it was necessary to consider this effect and account for it.

In this situation, given the very large size of the girders with respect to both their height and length, the self-weight was expected to have a large influence on the compressive strain profile results. Since draped strands were present in this set of girders, it was hypothesized that the variable eccentricity of the draped strands could also have an effect of on the concrete strains. However, inspection of the measured surface strains indicated that the compressive strains that resulted from the eccentricity of the draped strands balanced the self-weight strains so effectively that it was unnecessary to make any corrections to the strain profiles that were generated. This observation agreed with elastic strains computed for each of the various sources of deformation. In addition, it was determined that the need for a correction to account for *self-weight* creep was not needed as the effect of adding this correction was smaller than the effective precision of the DEMEC gauge. Also, steps were taken later in the process to account for the larger effect of creep due to the prestress force. A graph of the compressive strain values for an end transfer zone at transfer, at 7 days, and at 28 days is shown in Figure 4-15.

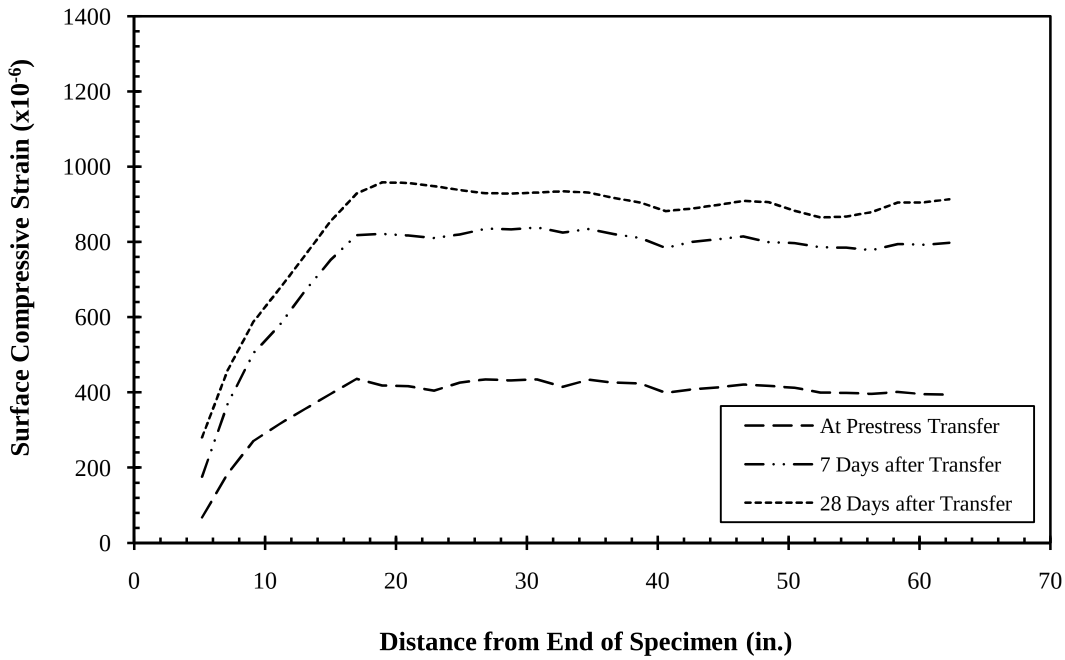



**Figure 4-15: Compressive Strain Profile for End Transfer Zone (Girder End 52-2S-A-F)**

### 4.3.2 Determination of Average Maximum Strain: End Transfer Zones

Once the surface strain profiles were plotted, the 95% AMS method was applied to determine the transfer length for each zone. This was first done by determining the 100% AMS value. As can be seen in Figure 4-15, the strains increase at a fairly constant slope before a plateau is reached. As in the definition of transfer length in Section 2.2.1, the stress in the strand will continue to increase until the effective prestress is reached within the section. This can be seen visually in the graph as the strain increases along the transfer length and then plateaus once the prestress is fully effective. The 100% AMS is the averaged strain value of the plateau portion of the graph, with the onset of the plateau being determined visually. The results for the same girder shown in Figure 4-15 can also be seen in Figure 4-16 along with the 100% AMS values at each time interval in which measurements were taken.

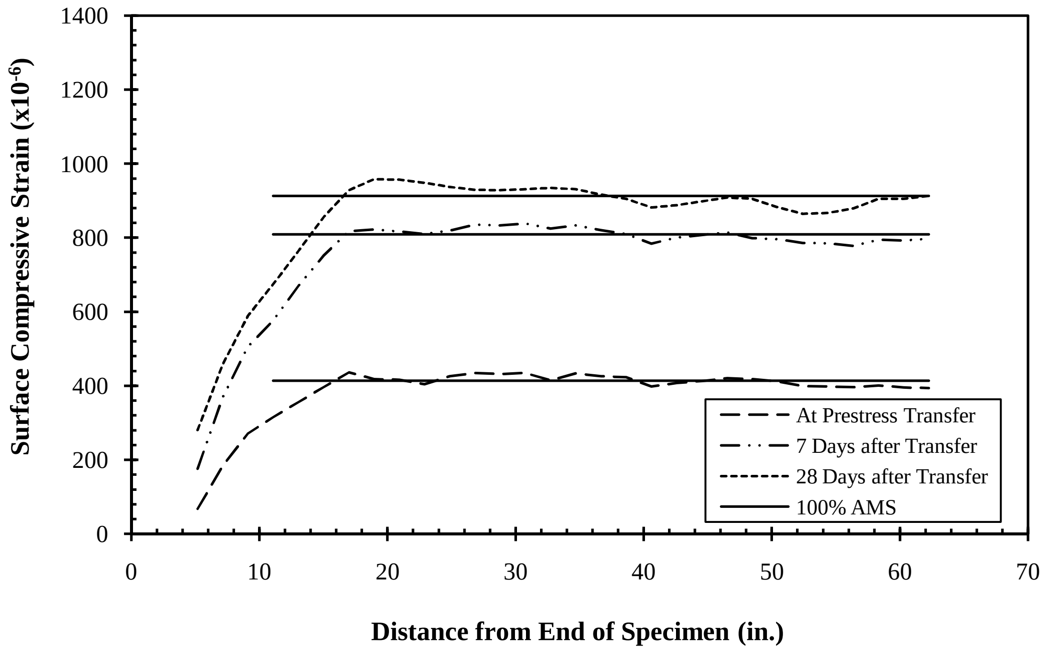



**Figure 4-16: 100% AMS of Concrete Surface Strains (Girder End 52-2S-A-F)**

The 95% AMS method was used to determine the transfer lengths within this study. This method uses the intersection of a horizontal line representing 95% of the average maximum strain and the surface compressive strain profile to establish the extent of the transfer length. This method was chosen as it requires little or no subjective judgment. While it can be argued whether an additional point should be added or removed from the averaged data to determine the 100% AMS, whether it be from the interpretation of the beginning of the plateau region or an error in a another point, the addition or subtraction of an additional point will have a minor effect on the resulting 100% AMS value (Russell and Burns 1993).

A 95% AMS value was used instead of 100% for two reasons.  First, it provided a clearly identifiable intersection location between the compressive strain profile and the bounding horizontal line (Russell and Burns 1993).  Second, the reduction in AMS, which may appear to artificially shorten the transfer length reading, actually compensates for the slight artificial elongation of the transfer length that comes from the rounding of the strain profile as a result of the smoothing process employed (Boehm 2008).

The 95% AMS calculation was different for the initial transfer lengths than it was for the later-age transfer lengths.  For the initial measurements, as can be expected, the 95% AMS value was computed by multiplying the 100% AMS value by 0.95.  The reduced AMS value was plotted against the strain profile of the measurements taken immediately after the release of the prestressing strands.  The location of the intersection of the strain profile and the 95% AMS line was taken as the end of the transfer length, which can be seen in Figure 4-17.

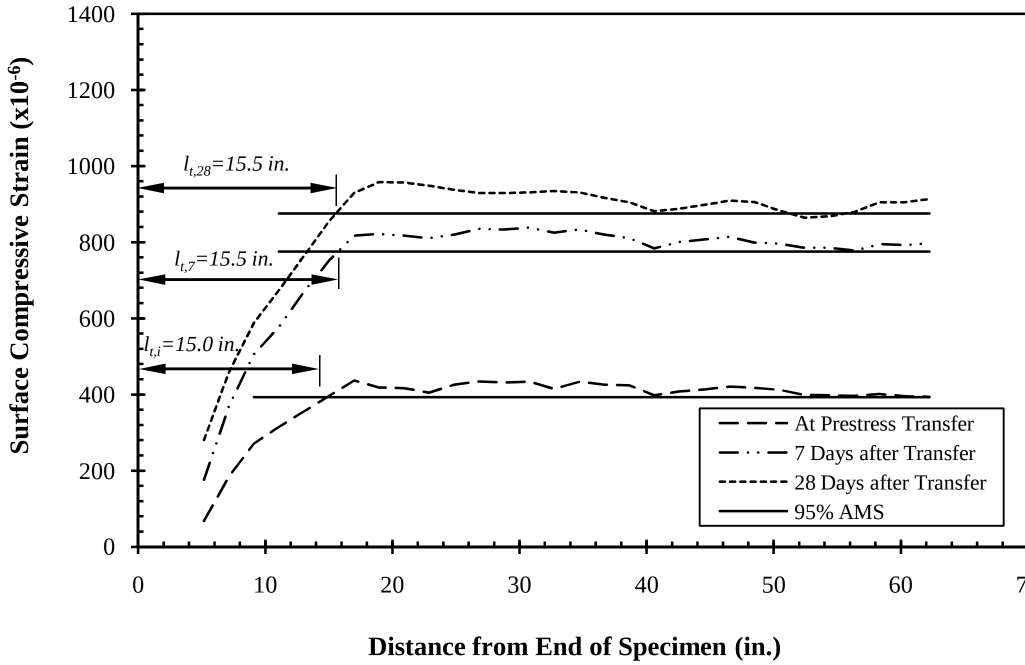



**Figure 4-17: Transfer Lengths with 95% AMS for End Transfer Zone (Girder End 54-2S-A-F)**

However, determining the 95% AMS for any long-term values was more complicated. Two time-dependent factors must be taken into account when analyzing long-term data. First, as the concrete continues to cure, the concrete shrinks. Second, once the strands were cut, the self-weight of the beam began to play a factor. The girder supported its own weight while it continued to cure causing creep. While both of these effects must be considered in long-term 95% AMS calculations, the means in which to account for each phenomena were different.

When looking at the effect of creep, the strains resulting from creep were assumed to be proportional to the applied load (Barnes, Burns, and Kreger 1999). Consequently, the strain profile is *amplified* over time due to creep alone, but the direct use of the 95% strain value will not artificially decrease the apparent transfer length. Thus, this technique would remain accurate if all time-dependent strain growth was only due to creep.

Conversely, shrinkage is not proportional to the applied load: it results in a *translation* of the strain profile. In this case, use of a factor of 0.95 results in an artificial decrease in the apparent transfer length (Barnes, Burns, and Kreger 1999). If all of the time-dependent deformation was due to shrinkage, it would be appropriate to reduce the 100% long-term strain profile by the same strain *value* (not percentage) by which initial 100% AMS was reduced.

The long-term effects are a result of both creep and shrinkage. After inspecting creep and shrinkage data, collected according to ASTM C512 (ASTM 2002) and shown in Figure 4-18, for the concrete that had been used to cast the girders, it was estimated that $^{1}/_{3}$ of the changes were due to shrinkage and $^{2}/_{3}$ were due to creep. Consequently, the long-term 95% AMS values were determined by appropriately weighting the creep-only and shrinkage-only approaches discussed above. This can also be seen in Equation 5-1.

$$\varepsilon_{c,95\%} = \varepsilon_{c,100\%} - \left[\frac{1}{3}\left(0.05 * \varepsilon_{c,100\%\ immediate}\right) + \frac{2}{3}\left(0.05 * \varepsilon_{c,100\%}\right)\right]$$

***Equation 5-1***

Where: $\varepsilon_{c,95\%}$ is the long-term 95% AMS value desired,

$\varepsilon_{c,100\%}$ is the 100% AMS value at the same long-term time as $\varepsilon_{c,95\%}$, and

$\varepsilon_{c,100\%\ immediate}$ is the 100% AMS value immediately after transfer.

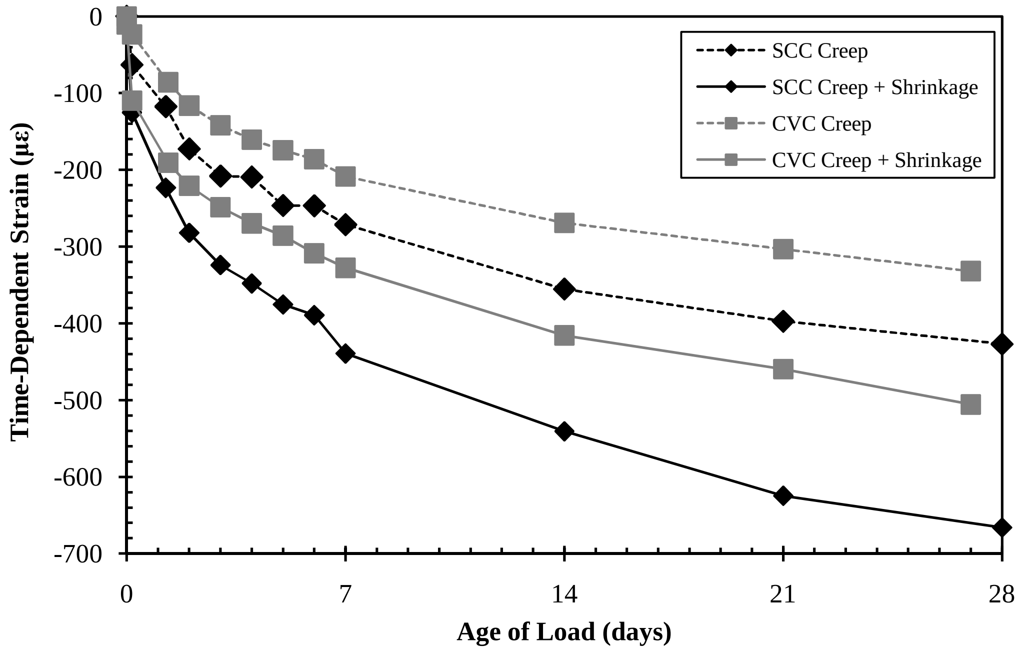



**Figure 4-18: Creep and Shrinkage Data**

### 4.3.3 Determination of Average Maximum Strain: Debonded-Strand Transfer Zones

Determining the 95% AMS of the debonded strands was more complicated than that of the fully bonded strands since it was appropriate to only consider the compressive strain resulting from the debonded strands. To accomplish this, the 100% AMS was determined the same way as that of the fully bonded strands: by determining the average of the plateau of the compressive strain profile. This can be seen in Figure 4-19.

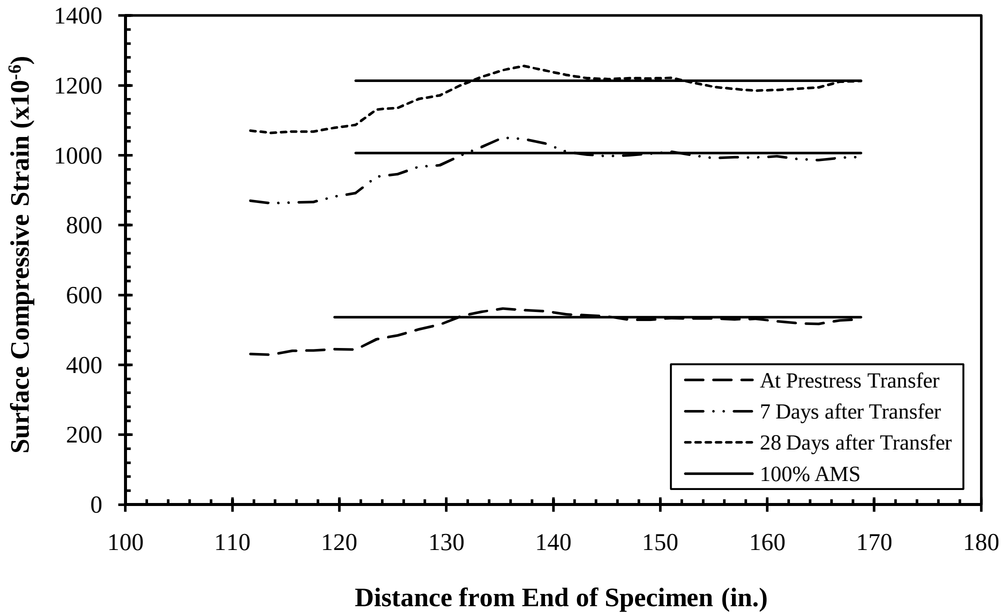



**Figure 4-19: 100% AMS of Concrete Surface Strains for Debonded Strand (Girder End 72-4S-A-D)**

Once the 100% AMS was determined for the debonded strands, the 100% AMS for the same end of the same girder was subtracted from the debonded AMS to determine the AMS due to only the debonded strands. At that point, the same procedure as followed previously was applied to account for the 5 percent of the difference between the 100% AMS of the fully bonded strands and 100 percent of the debonded strands accounting for both creep and shrinkage as was done previously. Equation 5-2 exhibits this process.

$$\varepsilon_{c,95\%\ debonded} = \varepsilon_{c,100\%\ debonded} - 0.05 * (\varepsilon_{c,100\%\ debonded} - \varepsilon_{c,100\%\ bonded})$$

***Equation 5-2***

Where: $\varepsilon_{c,95\%\ debonded}$ is the 95% AMS value for the debonded strain profile,

$\varepsilon_{c,100\%\ debonded}$ is the 100% AMS value for the debonded strain profile, and

$\varepsilon_{c,100\%\ bonded}$ is the 100% AMS value for the corresponding fully bonded strain profile at the same time.

The same methodology was used to determine the long-term 95% AMS debonded values as that which was used for the fully bonded strain profiles, as described in Equation 5-1. Finally, the transfer lengths for the debonded strands were calculated and can be seen in Figure 4-20. Since the sheathing on the strands did not end until 10 feet from the end of the girder, the transfer length was taken as the distance from the beginning of bonding (120 in. from the end) to the point where the strain profile crossed the 95% AMS threshold.

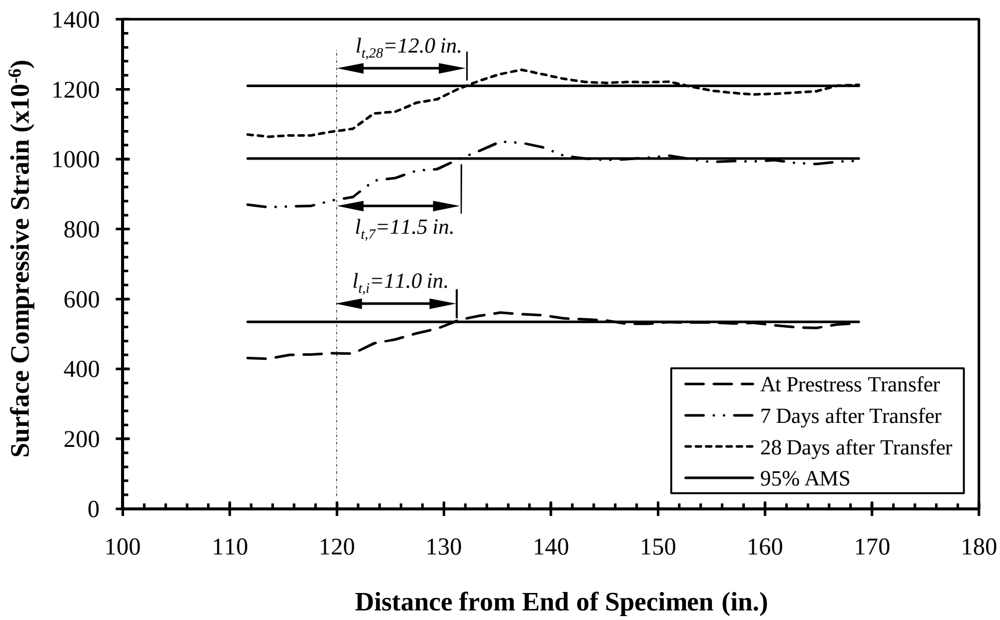



**Figure 4-20: Transfer Lengths with 95% AMS for Debonded-Strand Transfer Zone (Girder End 72-4S-A-D)**

### 4.3.4 Precision of Results

After looking at the factors which were a part of the study, it was determined that a transfer length precision of 0.5 inches was suitable. Given that each DEMEC point was spaced at 1.97 inches (50 mm), and the results of the strain data were then smoothed and interpolated, it would be inappropriate to report results more precisely than to the nearest tenth of an inch. However, given the local conditions in which measurements were taken, many of which required awkward movements to read the gauge, and the assumptions made with the regard to the involvement of creep and shrinkage in the long-term analyses, more uncertainty was introduced into system. Consequently, it was judged that 0.5 inches is the most appropriate precision for these transfer lengths.

# Chapter 5 Results and Discussion

## 5.1 Results and Discussions

Thirty-four transfer zones were analyzed and incorporated as a part of this study. Of these, twenty-four zones anchored fully bonded strands, and ten anchored debonded strands. The results of the testing and analysis are presented in the chapter, along with comparisons made with respect to the effects of each parameter considered, the previous phases of the study, and the current design convention.

### 5.1.1 Transfer Length Results of Fully Bonded Strands

For the twelve PCI bulb-tee girders tested, twenty-four transfer lengths of fully bonded strands were measured. The strain profiles and the resulting transfer length of each transfer zone can be seen in Appendix D. Each of the zones underwent the analysis procedure described previously, resulting in transfer length determination at transfer of the prestressing force, seven days after transfer, and twenty-eight days after transfer. A summary of the results, as well as some of the relevant properties, can be seen in Table 5-1. The table includes information on the concrete cylinder compressive strength at transfer ($f'_{ci}$), the strand jacking stress ($f_{pj}$), and the strand stress immediately after transfer ($f_{pt}$). The concrete strength at transfer, the jacking stress, and the transfer lengths were all values that were measured and collected in the field. The stress in the prestressing strand after transfer was calculated based upon the jacking stress, a reasonable estimate for the strand relaxation prior to transfer, and the average concrete strain change that was measured at transfer in the girder being examined.

**Table 5-1: Summary of Transfer Length Results and Properties for Fully Bonded Strands**

| | | | | | | End A | | | End B | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Concrete | Depth (in.) | Girder ID | $f'_{ci}$ (psi) | $f_{pj}$ (ksi) | $f_{pt}$ (ksi) | Initial (in.) | 7-Day (in.) | 28-Day (in.) | Initial (in.) | 7-Day (in.) | 28-Day (in.) |
| SCC | 54 | 2S | 9010 | 202.5 | 190 | 15.0 | 15.5 | 15.5 | 15.5 | 17.5 | 17.0 |
| | | 4S | 8680 | 202.5 | 190 | 20.0 | 19.0 | 20.5 | 14.0 | 15.0 | 15.5 |
| | | 7S | 8760 | 202.5 | 190 | 14.0 | 16.0 | 16.5 | 9.5 | 10.5 | 10.0 |
| | 72 | 2S | 8220 | 202.4 | 188 | 16.0 | 17.5 | 17.5 | 14.5 | 19.0 | 17.0 |
| | | 4S | 7860 | 202.4 | 188 | 9.5 | 12.0 | 13.5 | 18.0 | 19.0 | 20.0 |
| | | 7S | 8120 | 202.4 | 188 | 9.0 | 9.5 | 9.0 | 20.0 | 19.5 | 18.5 |
| CVC | 54 | 8C | 7940 | 202.5 | 190 | 9.0 | 12.0 | 11.5 | 15.0 | 19.0 | 17.0 |
| | | 11C | 7860 | 202.5 | 190 | 10.0 | 10.5 | 12.0 | 12.5 | 13.5 | 14.5 |
| | | 13C | 8790 | 202.5 | 190 | 9.0 | 12.0 | 11.0 | 12.0 | 14.5 | 14.0 |
| | 72 | 8C | 8290 | 202.4 | 190 | 11.0 | 13.0 | 15.0 | 11.0 | 13.5 | 15.0 |
| | | 11C | 8320 | 202.4 | 190 | 10.5 | 13.0 | 14.0 | 9.0 | 11.5 | 11.5 |
| | | 13C | 8770 | 202.4 | 190 | 7.5 | 9.0 | 10.0 | 12.5 | 16.0 | 16.0 |

### 5.1.2 Transfer Length Results Partially Debonded Strands

Ten partially debonded tension zones were considered in four of the BT-54 girders and six of the BT-72 girders. The full compressive strain profiles created for each debonded-strand transfer zone can be seen in Appendix E. Table 5-2 shows the transfer lengths for the debonded strands at transfer, 7 days, and 28 days.

**Table 5-2: Summary of Transfer Length Results and Properties for Debonded Strands**

| Concrete | Depth (in.) | Girder ID | $f'_{ci}$ (psi) | $f_{pj}$ (ksi) | $f_{pt}$ (ksi) | Initial (in.) | 7-Day (in.) | 28-Day (in.) |
|---|---|---|---|---|---|---|---|---|
| SCC | 54 | 2S | 9010 | 202.5 | 190 | 14.5 | 11.5 | 12.0 |
| | | 4S | 8680 | 202.5 | 190 | 14.0 | 13.5 | 13.5 |
| | 72 | 2S | 8220 | 202.4 | 187 | 11.5 | 13.5 | 14.0 |
| | | 4S | 7860 | 202.4 | 187 | 11.0 | 11.5 | 12.0 |
| | | 7S | 8120 | 202.4 | 187 | 14.5 | 13.5 | 13.5 |
| CVC | 54 | 11C | 7860 | 202.5 | 191 | 13.0 | 12.0 | 12.0 |
| | | 13C | 8790 | 202.5 | 191 | 12.5 | 11.5 | 10.5 |
| | 72 | 8C | 8290 | 202.4 | 189 | 11.5 | 12.0 | 13.0 |
| | | 11C | 8320 | 202.4 | 189 | 7.0 | 9.5 | 9.5 |
| | | 13C | 8770 | 202.4 | 189 | 12.0 | 13.5 | 11.0 |

### 5.1.3 Effect of Time

As previously discussed in Chapter 2, time does play a role in the transfer length results. In previous phases of the study, it was discovered that while the lengths continued to grow for several weeks after transfer, the majority of the growth occurred within the first few days. Consequently, measurements were taken for this study at both 7 days and 28 days in order to capture larger changes that would occur within the first few days, as well as any additional growth that would be seen afterward. The growth from the initial measurements taken immediately after transfer to the long-term measurements at 28 days for the fully bonded strands are visible in Figure 5-1, with the average transfer length for each concrete type, girder size, and time period given.

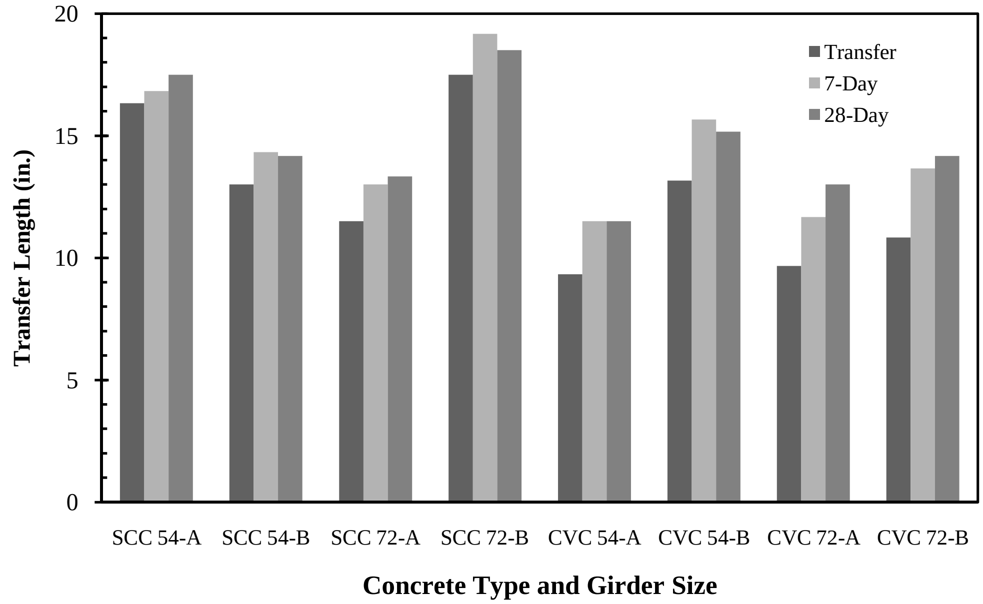



**Figure 5-1: Effect of Time on Transfer Lengths**

While inconsistent in the amount of growth, it can be seen that the 7-day and 28-day transfer lengths are almost always longer than the initial lengths. The long-term measurements were then compared to the measurements taken immediately after transfer in order to depict the percentage of growth in the transfer length from the lengths taken after release. A breakdown of the growth each type of concrete and girder size experienced is detailed in Table 5-3. The values shown are the results of the ratio of the long-term transfer length compared to the initial transfer length.

**Table 5-3: Time Effects on Transfer Length of Bulb-Tee Girders**

| Concrete | Size | Fully Bonded $l_{t,7\text{-}days}/l_{t,initial}$ | Fully Bonded $l_{t,28\text{-}days}/l_{t,initial}$ | Debonded $l_{t,7\text{-}days}/l_{t,initial}$ | Debonded $l_{t,28\text{-}days}/l_{t,initial}$ |
|---|---|---|---|---|---|
| SCC | 54 | 1.07 | 1.08 | 0.88 | 0.90 |
| | 72 | 1.13 | 1.12 | 1.05 | 1.08 |
| CVC | 54 | 1.21 | 1.19 | 0.92 | 0.88 |
| | 72 | 1.23 | 1.33 | 1.18 | 1.13 |

As mentioned before, the trend for transfer length growth over time has shown that a large amount of growth occurs in the few days immediately after transfer, with some, but significantly less growth occurring beyond that point. This can be seen in the results as significant growth occurred between the initial and seven-day measurements, anywhere from 7 percent to 23 percent, with minimal growth occurring after that. The growth seen from 7 days to 28 days after transfer was typically just a few percent or less. Overall, the transfer lengths grew 18 percent from the values collected after transfer. It would also appear that the transfer lengths grew more when cast in CVC than when cast in SCC. This trend agrees with that found by Staton et al. (2009), who reported that the additional growth of the CVC compared to the SCC was attributed to time-dependent softening of the CVC around the strands.

The debonded-strand transfer lengths saw significantly less growth, and in some cases appeared to shorten, over the twenty-eight day time period. Much of the growth of transfer lengths has been previously associated with creep and shrinkage within the concrete. After transfer release, very large stresses are found in the concrete and the ends of the girder, with less towards the center as the self-weight of the girder counteracts the prestressing forces. With this in mind, the concrete more towards the end of the girders would experience more creep effects than that located more towards the middle of the girder. Consequently, the transfer lengths of the fully bonded strands, which were associated with the more highly stressed concrete, saw more growth due to the creep effects than the transfer lengths associated with the debonded strands located ten feet towards the center of the beam.

The results were also compared to previous results found by Swords (2005), Levy (2007), and Boehm (2008). Each study utilized a different time period in which to observe the variation of the transfer length. Swords (2005) and Boehm (2008) took strain measurements at multiple time intervals in order to capture the time frame in which most of the transfer length growth occurred. Levy (2008), on the other hand, only took measurements after prestress transfer and four days after transfer, as it was assumed that most of the growth in the transfer lengths would occur within the shorter period. Consequently, there was not a uniform time after release in which each of the different studies took measurements.

While comparisons can be made for the three previous phases within one day of the next, the first longer-term set of measurements for this study was at an age of 7 days. While twice as long as the time frames looked at by the other researchers, the comparison of the 7-day data to the 3- and 4-day data may still be enlightening. The fact that a majority of the growth occurs over the first few days and then slows down dramatically might indicate that the seven-day results found as part of this study were only a little bit longer than if they had been taken at three days. As the previous studies also examined several types of SCC with varying strengths, it was necessary to only consider the high-strength SCC mixtures which corresponded to the SCC used in the bulb-tee girders tested as part of this study. Consequently, Table 5-4 shows the amount of growth seen in each study. Overall, the PCI bulb-tee transfer lengths found in this study grew approximately 10 percent from the time of release, which is within the range established by the previous studies. Results from this and previous studies show that it is inappropriate to use the initial transfer lengths as an estimate of long-term transfer lengths given the significant growth that occurs with time.

**Table 5-4: Effect of Time on Transfer Lengths in SCC**

| Study | Days | $\frac{l_{t,long-term}}{l_{t,immediate}}$ |
|---|---|---|
| Prisms (Swords 2005) | 3 | 1.04 |
| T-Beams (Levy 2007) | 4 | 1.15 |
| AASHTO Type I (Boehm 2008) | 3 | 1.16 |
| PCI BT (current study) | 7 | 1.10 |

### 5.1.4 Effect of Debonding Strands

As there seems to be no differentiation between transfer lengths of fully bonded and debonded strands in the code provisions, it was logical to look at the debonded strands compared to the fully bonded strands. Transfer lengths for partially debonded strands were not measured in girders BT-54-7S and BT-54-8C. Table 5-5 shows the ratio obtained when comparing the debonded-strand transfer length ($l_{t,debonded}$) to the transfer length of the fully bonded strands ($l_{t,end}$) in the same end region of the same girder. Averages were then made for the four combinations of concrete type and girder size, as well as averages reflecting the SCC and the CVC.

**Table 5-5: Comparison of Transfer Lengths of Fully Bonded and Debonded Strands**

| Concrete | Girder ID | Depth (in.) | $l_{t,debond}/l_{t,end}$ | Average | |
|---|---|---|---|---|---|
| SCC | 2S | 54 | 0.77 | 0.72 | 0.89 |
| | 4S | | 0.66 | | |
| | 2S | 72 | 0.80 | 1.06 | |
| | 4S | | 0.89 | | |
| | 7S | | 1.50 | | |
| CVC | 11C | 54 | 1.00 | 0.98 | 0.93 |
| | 13C | | 0.95 | | |
| | 8C | 72 | 0.87 | 0.88 | |
| | 11C | | 0.68 | | |
| | 13C | | 1.10 | | |

On average, the transfer lengths for the debonded strands were about 10 percent shorter than for the corresponding fully bonded strands; however, this relationship is not consistent for all the girders. One particularly interesting result was that of girder BT-72-7S, which indicated that the debonded-strand transfer length was 50 percent longer than the transfer length of the fully bonded strands. Upon closer analysis, it would appear that this was due to the fact that the transfer length for the fully bonded strands in this girder end was abnormally short (9 in.). Based on the small set of debonded-strand transfer lengths measured in this study, there does not seem to be a consistent difference between the transfer bond behavior of fully bonded and partially debonded prestressing strands. There is certainly no evidence to suggest that the debonded-strand transfer lengths were longer than the fully bonded strands, and confirms the results found by Russell and Burns (1993) and Barnes, Burns, and Kreger (1999). Consequently, there is no reason to suggest that the code provisions be modified to contain special provisions pertaining to the transfer length of debonded strands for situations in which cracking is not present.

### 5.1.5 Effect of Placement Sequence and Beam Orientation

After looking at the results of the research, it appeared that the transfer length of one end of a girder was often significantly shorter than the corresponding transfer length at the opposite end of the girder, which can be seen in Table 5-1. As an end-to-end comparison was the most direct comparison that could be made, since the same concrete mixture proportions and the same prestressing force were present throughout the girder, an outside factor had to be considered.

One theory was based upon the size of the girders in question. The smaller of the girders (BT-54) contained over sixteen cubic yards of concrete. Consequently, by the time the second end of the girder was cast, a significant amount of time had transpired from the time that the first end was cast, despite the fact that concrete was almost continuously being added to the girder. Another concern was that as the SCC was cast, the concrete was placed in the first end of the girder and began to flow to the bottom of the other end. By the time it did reach the other end, the possibility of segregation had the potential of being higher. As a result, a comparison was made between the transfer length at the second end of the girder in relation to the transfer length at the end in which concrete placement began. A summary of these findings can be seen in Table 5-6, where ‘End 1’ refers to the first end of the girder placed and ‘End 2’ refers to the final end placed.

**Table 5-6: Comparison of Transfer Lengths Based Upon Placement Sequence**

| Concrete | Girder ID | Depth (in.) | $l_{t,end\ 2}/l_{t,end\ 1}$ | Average | |
|---|---|---|---|---|---|
| SCC | 2S | 54 | 1.10 | 0.82 | 0.77 |
| | 4S | | 0.76 | | |
| | 7S | | 0.61 | | |
| | 2S | 72 | 1.03 | 0.73 | |
| | 4S | | 0.68 | | |
| | 7S | | 0.49 | | |
| CVC | 8C | 54 | 1.48 | 1.03 | 1.09 |
| | 11C | | 0.83 | | |
| | 13C | | 0.79 | | |
| | 8C | 72 | 1.00 | 1.14 | |
| | 11C | | 0.82 | | |
| | 13C | | 1.60 | | |

Given the wide range of ratios seen, from 0.49 to 1.10 for the SCC and 0.79 to 1.60 for the CVC, it is difficult to conclude that concrete placement was the cause of the differences between the two different ends of the girders. However, looking at averages, it does appear that the placement sequence of SCC an effect on the transfer lengths. As the average transfer lengths were shorter in the end that was placed second during concrete placement, it was hypothesized that the effect of the concrete being placed laterally as the concrete flowed from the first end to the second end allowed for a more effective form of bond anchorage, probably in the form of mechanical resistance, than what was achieved in the first end of concrete placement where the SCC was dropped into place vertically.

It was also considered that the effect of cutting the strand could have some impact on the transfer lengths. Previous research done by Swords (2005), Levy (2007) and Boehm (2008) did show that a significant difference was seen between the live and dead ends of the girder that was caused by flame cutting all the strands at one girder end, called the *live* end, before cutting the detensioned strands in the remaining locations, the *dead* ends. This was not the case for the bulb-tee girders as the strand was cut at each end simultaneously, resulting in all live ends. It was noticed, however, that on nearly all the ends of the girders in which a longer length of strand was exposed, at the beginning and end of the prestressing bed, longer transfer lengths were measured. When the exterior ends of the girders (those adjacent to a longer length of unrestrained prestressing strand) were compared to the interior ends (those adjacent to another beam end, and thus a shorter length of unrestrained strand), an effect was evident. This can be seen in the interior-to-exterior transfer length ratios for each beam reported in Table 5-7.

**Table 5-7: Comparison of Transfer Lengths Based Upon Beam Orientation**

<table>
<tr><th>Concrete</th><th>Girder ID</th><th>Depth (in.)</th><th>$l_{t,interior}$/ $l_{t,exterior}$</th><th colspan="2">Average</th></tr>
<tr><td rowspan="6">SCC</td><td>2S</td><td rowspan="3">54</td><td>1.10</td><td rowspan="3">0.82</td><td rowspan="6">0.77</td></tr>
<tr><td>4S</td><td>0.76</td></tr>
<tr><td>7S</td><td>0.61</td></tr>
<tr><td>2S</td><td rowspan="3">72</td><td>0.97</td><td rowspan="3">0.71</td></tr>
<tr><td>4S</td><td>0.68</td></tr>
<tr><td>7S</td><td>0.49</td></tr>
<tr><td rowspan="6">CVC</td><td>8C</td><td rowspan="3">54</td><td>0.68</td><td rowspan="3">0.76</td><td rowspan="6">0.79</td></tr>
<tr><td>11C</td><td>0.83</td></tr>
<tr><td>13C</td><td>0.79</td></tr>
<tr><td>8C</td><td rowspan="3">72</td><td>1.00</td><td rowspan="3">0.82</td></tr>
<tr><td>11C</td><td>0.82</td></tr>
<tr><td>13C</td><td>0.63</td></tr>
</table>

Table 5-7 shows that there does seem to be a correlation between the girder ends that have longer lengths of exposed strand and longer transfer lengths. When the strands were cut at locations with longer lengths of exposed strand, the release was more violent as more potential energy was converted into kinetic energy. As seen with the live ends from the previous studies performed by Swords (2005), Levy (2007), and Boehm (2008), the more dynamic the release was, the longer the transfer lengths tended to be. The transfer lengths found for the interior ends of the girders studied, while still longer than what would have been found at dead ends, were shorter (22 percent) than those seen on the exterior ends of the girders, independent of the type of concrete in use. However, it needs to be noted that in many instances, the exterior end of the girder was also the first girder end to be cast. Without the ability to separate these two variables into significantly large sample sizes, it is difficult to comment on the extent that these variables had upon the transfer lengths.

### 5.1.6 Normalization of Results

When looking at the results presented in this chapter, as well as those collected previously in association with other phases of the project, there were many variables present. In order to properly compare the values independent of strand size, concrete strength, and prestress intensity, steps were taken to ensure like quantities were available for the comparisons.

From Mattock's original equation, Equation 2-7, it can be seen that the transfer length was found to be proportional to both the stress in the prestressing steel, $f_{pe}$, and the strand diameter, $d_b$. In addition, it is inversely proportional to the average transfer bond stress capacity, $\bar{u}_{tb}$, of the concrete (Barnes, Grove, and Burns 2003). This can be summarized by Equation 5-3:

$$l_t \propto \frac{f_{pt}}{\bar{u}_{tb}} d_b \qquad \textbf{\textit{Equation 5-3}}$$

Mitchell et al. (1993) recognized the need for a correction factor to account for the effects of high-strength concrete, which took the form of the inverse of the square root of the concrete strength at transfer. Barnes, Burns, and Kreger (1999) justified this by reasoning that the average transfer bond stress capacity of the concrete is heavily dependent on the stiffness and tensile strength of the concrete, which are also often approximated as being proportional to the square root of the concrete compressive strength. Consequently, by assigning a constant of proportionality, α, Equation 5-3 can be modified to the following equation:

$$l_t = \alpha \frac{f_{pt}}{\sqrt{f'_{ci}}} d_b \qquad \textbf{\textit{Equation 5-4}}$$

By solving for the proportionality constant, α, a measured transfer length can be normalized in terms of concrete strength, the effective prestress after release, and the strand diameter. This value can then be used to compare transfer length results by providing a measure which quantifies the "relative transfer bond performance after normalization for the effects of prestress magnitude and concrete strength" (Boehm 2008) as well as strand size. Research done by both Swords (2005) and Levy (2007) confirmed that the relationship shown in Equation 5-4 provided the best correlation between experimental results and predicted values. Consequently, the same relationship is used in the following sections, and many comparisons are based upon the proportionality constant, α, that corresponds to each measured transfer length. The average α values can be seen for both SCC and CVC for the transfer lengths of the fully bonded strands in Figure 5-2 and the debonded lengths in Figure 5-3, where the measured transfer lengths from the bulb-tee girders are plotted versus $\frac{f_{pt}}{\sqrt{f'_{ci}}} d_b$. The domain of the data on these plots is small because there is little variation in the $\frac{f_{pt}}{\sqrt{f'_{ci}}} d_b$ values for the transfer lengths in this study.

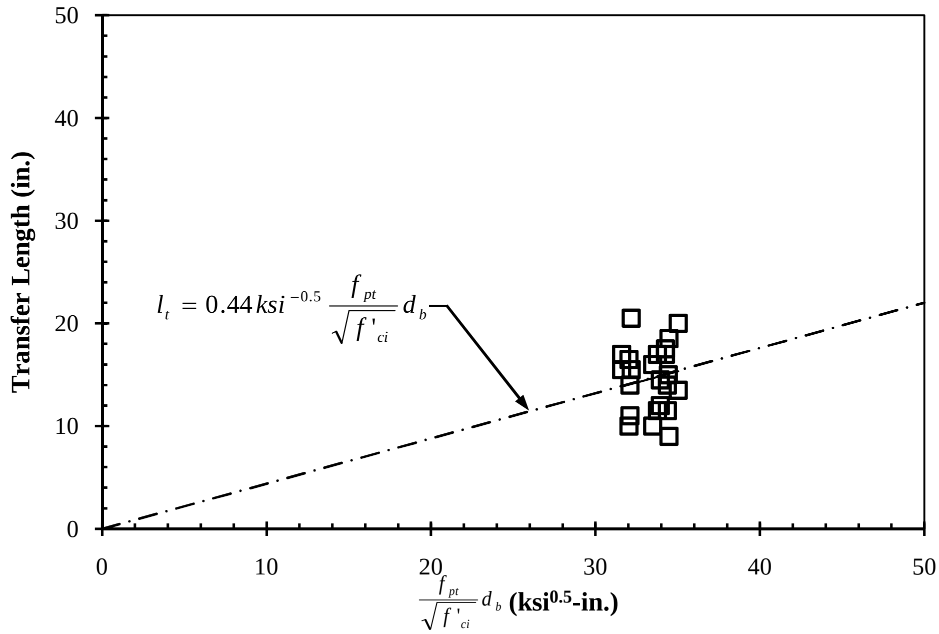



**Figure 5-2: Transfer Length as a Function of Tendon Prestress and Concrete Strength at Transfer of Fully Bonded Strands**

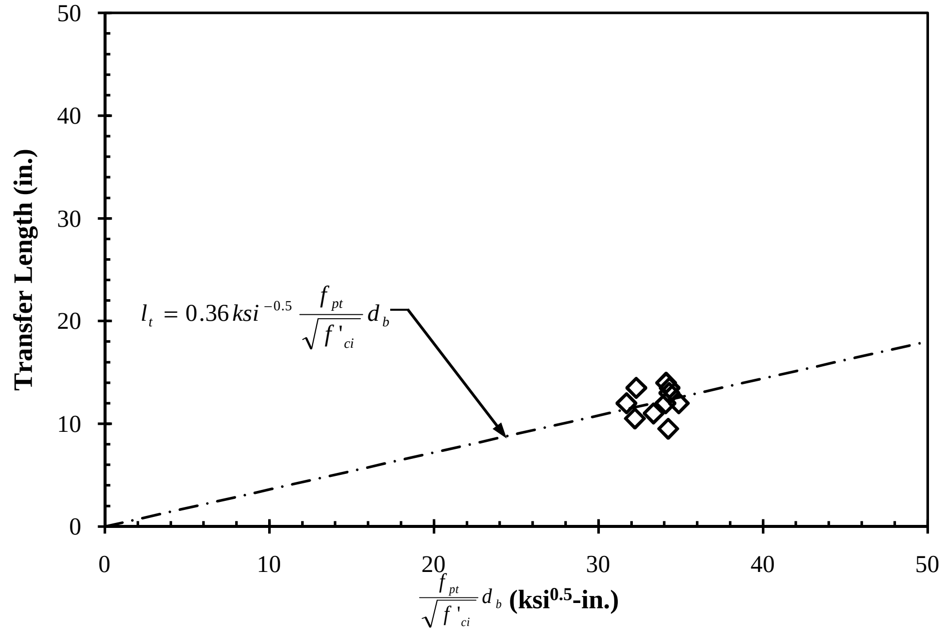



**Figure 5-3: Transfer Length as a Function of Tendon Prestress and Concrete Strength at Transfer of Debonded Strands**

### 5.1.7 Effect of Concrete Strength

While this particular study did not focus on the effects of the concrete strength, Swords (2005), Levy (2007), and Boehm (2008) determined in previous studies that the concrete strength does affect the transfer length of SCC with higher strength concretes producing shorter transfer lengths. Variable concrete strengths were seen amongst the results of this study; however, the variation of strengths fell within a range of 1150 psi (from 7860 psi to 9010 psi) at transfer, a range too small to illuminate a clear sensitivity in the transfer length results. However, while the range of concrete strengths in this study was not broad enough to justify conclusions about the effect of concrete strength on transfer length, it was still necessary to account for the differences in the concrete strength, strand size, and prestress when examining the effects of other variables. This was accomplished by applying the normalizing process described previously and comparing $\alpha$ values.

### 5.1.8 Effect of Concrete Type—CVC versus SCC

The comparison of transfer bond behavior in SCC to transfer lengths in CVC is the primary focus of this study. In order to properly compare the transfer lengths as a function of the type of concrete employed, the transfer lengths measured at an age of 28 days were compared. Table 5-8 shows the averaged transfer lengths of the fully bonded strands for each girder, thus representing each of the twelve girders, along with the corresponding debonded-strand transfer lengths where applicable. Averages based on the type of concrete and size of the girder are also shown.

**Table 5-8: Comparison of SCC and Conventional Mixture Transfer Length Results**

<table>
<tr><th rowspan="2">Concrete</th><th rowspan="2">Girder ID</th><th rowspan="2">Depth (in.)</th><th colspan="3">Fully Bonded</th><th colspan="3">Debonded</th></tr>
<tr><th>$l_t$ (in.)</th><th colspan="2">Average $l_t$<br>(in.) (in.)</th><th>$l_t$ (in.)</th><th colspan="2">Average $l_t$<br>(in.) (in.)</th></tr>
<tr><td rowspan="6">SCC</td><td>2S</td><td rowspan="3">54</td><td>16.25</td><td rowspan="3">15.8</td><td rowspan="6">15.9</td><td>12.00</td><td rowspan="3">12.8</td><td rowspan="6">13.0</td></tr>
<tr><td>4S</td><td>18.00</td><td>13.50</td></tr>
<tr><td>7S</td><td>13.25</td><td>N/A</td></tr>
<tr><td>2S</td><td rowspan="3">72</td><td>17.25</td><td rowspan="3">15.9</td><td>14.00</td><td rowspan="3">13.2</td></tr>
<tr><td>4S</td><td>16.75</td><td>12.00</td></tr>
<tr><td>7S</td><td>13.75</td><td>13.50</td></tr>
<tr><td rowspan="6">CVC</td><td>8C</td><td rowspan="3">54</td><td>14.25</td><td rowspan="3">13.3</td><td rowspan="6">13.5</td><td>N/A</td><td rowspan="3">11.3</td><td rowspan="6">11.2</td></tr>
<tr><td>11C</td><td>13.25</td><td>12.00</td></tr>
<tr><td>13C</td><td>12.50</td><td>10.50</td></tr>
<tr><td>8C</td><td rowspan="3">72</td><td>15.00</td><td rowspan="3">13.6</td><td>13.00</td><td rowspan="3">11.2</td></tr>
<tr><td>11C</td><td>12.75</td><td>9.50</td></tr>
<tr><td>13C</td><td>13.00</td><td>11.00</td></tr>
</table>

As previously mentioned, given the variability of the concrete strength, the strand size, and the stress being anchored in the prestressing strand, the α value for each transfer length was used to allow for more direct comparisons to be made between the various transfer zones. Table 5-9 shows the average α values for the transfer length of the fully bonded strands of each girder and Table 5-10 shows the corresponding values for the debonded-strand transfer zones. Consequent averages for girders of each size with respect to the type of concrete used, as well as the overall averages of the transfer length of both SCC and CVC, are given.

**Table 5-9: Comparison of SCC and Conventional Mixture Results of Fully Bonded Strands**

| Concrete | Girder ID | Depth (in.) | α ($ksi^{-0.5}$-in.) | Average α ($ksi^{-0.5}$-in.) | Average α ($ksi^{-0.5}$-in.) |
|---|---|---|---|---|---|
| SCC | 2S | 54 | 0.51 | 0.50 | 0.48 |
| | 4S | | 0.56 | | |
| | 7S | | 0.41 | | |
| | 2S | 72 | 0.50 | 0.46 | |
| | 4S | | 0.48 | | |
| | 7S | | 0.40 | | |
| CVC | 8C | 54 | 0.42 | 0.40 | 0.40 |
| | 11C | | 0.39 | | |
| | 13C | | 0.39 | | |
| | 8C | 72 | 0.44 | 0.40 | |
| | 11C | | 0.37 | | |
| | 13C | | 0.39 | | |

**Table 5-10: Comparison of SCC and Conventional Mixture Results of Debonded Strands**

| Concrete | Girder ID | Depth (in.) | α | Average α | |
|---|---|---|---|---|---|
| | | | (ksi$^{-0.5}$-in.) | (ksi$^{-0.5}$-in.) | (ksi$^{-0.5}$-in.) |
| SCC | 2S | 54 | 0.38 | 0.40 | 0.39 |
| | 4S | | 0.42 | | |
| | 7S | | N/A | | |
| | 2S | 72 | 0.41 | 0.38 | |
| | 4S | | 0.34 | | |
| | 7S | | 0.39 | | |
| CVC | 8C | 54 | N/A | 0.34 | 0.33 |
| | 11C | | 0.35 | | |
| | 13C | | 0.33 | | |
| | 8C | 72 | 0.38 | 0.33 | |
| | 11C | | 0.28 | | |
| | 13C | | 0.33 | | |

It is clearly apparent from the results that the transfer lengths in SCC are longer than those found in CVC.  For fully bonded strands, SCC was found to produce transfer lengths 20% longer than CVC, and 18% longer transfer lengths for debonded strands. The transfer lengths in the SCC were approximately 2 inches (4 strand diameters) longer.

A second, more in depth comparison was also made.  Each transfer zone for fully bonded strands within an SCC girder was paired with a corresponding transfer zone for fully bonded strands in a CVC girder.  The pairs were assigned in such a manner that every variable between the two zones was accounted for.  The paired ends matched with respect to the girder size, strand size, orientation on the prestressing bed, placement order, and final placement within the bridge.  The one exception to these rules were girders BT-54-7S and BT-54-8C, in which the girder end that was placed first for one girder was paired with the end final placement end of the other girder.  However, the exterior and interior girder end orientations were still paired because this was shown to be more important than the order of concrete placement. The 28-day α values were utilized for this comparison, which took into account the slight variation in concrete strength and prestress magnitude within each pair. Table 5-11 displays the SCC and CVC girder ends that were paired, whether it was the first or final girder end to be placed, and whether the orientation of the girder on the prestressing bed resulted in it being an exterior or interior girder end. Finally, the ratio of the SCC α value to the CVC α for each pair is reported, as well as an average for each girder size.

**Table 5-11: Paired SCC and CVC Girder Ends and Comparisons with Respect to Girder Depth**

| Depth (in.) | SCC End | CVC End | Start/Finish Placement | Exterior/ Interior | $\alpha_{SCC}/\alpha_{CVC}$ | Average |
|---|---|---|---|---|---|---|
| 54 | 2S-A | 13C-B | 1 | E | 1.13 | 1.25 |
| | 2S-B | 13C-A | 2 | I | 1.57 | |
| | 4S-A | 11C-B | 1 | E | 1.49 | |
| | 4S-B | 11C-A | 2 | I | 1.36 | |
| | 7S-A | 8C-B | Mixed | E | 1.02 | |
| | 7S-B | 8C-A | Mixed | I | 0.92 | |
| 72 | 2S-A | 13C-B | 1 | E | 1.07 | 1.19 |
| | 2S-B | 13C-A | 2 | I | 1.66 | |
| | 4S-A | 11C-B | 1 | E | 1.15 | |
| | 4S-B | 11C-A | 2 | I | 1.40 | |
| | 7S-A | 8C-B | 1 | E | 0.60 | |
| | 7S-B | 8C-A | 2 | I | 1.23 | |

The pairs were also grouped with respect to both the order in which the ends were placed and whether the end was an exterior or interior end. The results can be seen in Table 5-12 and Table 5-13.

**Table 5-12: Paired SCC and CVC Girder Ends and Comparisons with Respect to Placement Sequence**

| Start/Finish Placement | SCC End | CVC End | Depth (in.) | Exterior/ Interior | $\alpha_{SCC}/\alpha_{CVC}$ | Average |
|---|---|---|---|---|---|---|
| 1 | 2S-A | 13C-B | 54 | E | 1.13 | 1.09 |
| 1 | 4S-A | 11C-B | 54 | E | 1.49 | |
| 1 | 2S-A | 13C-B | 72 | E | 1.07 | |
| 1 | 4S-A | 11C-B | 72 | E | 1.15 | |
| 1 | 7S-A | 8C-B | 72 | E | 0.60 | |
| 2 | 2S-B | 13C-A | 54 | I | 1.57 | 1.45 |
| 2 | 4S-B | 11C-A | 54 | I | 1.36 | |
| 2 | 2S-B | 13C-A | 72 | I | 1.66 | |
| 2 | 4S-B | 11C-A | 72 | I | 1.40 | |
| 2 | 7S-B | 8C-A | 72 | I | 1.23 | |
| Mixed | 7S-A | 8C-B | 54 | E | 1.02 | 0.97 |
| Mixed | 7S-B | 8C-A | 54 | I | 0.92 | |

**Table 5-13: Paired SCC and CVC Girder Ends and Comparisons with Interior and Exterior Girder Ends**

| Exterior/ Interior | SCC End | CVC End | Start/Finish Placement | Depth (in.) | $\alpha_{SCC}/\alpha_{CVC}$ | Average |
|---|---|---|---|---|---|---|
| E | 2S-A | 13C-B | 1 | 54 | 1.13 | 1.08 |
| E | 4S-A | 11C-B | 1 | 54 | 1.49 | |
| E | 2S-A | 13C-B | 1 | 72 | 1.07 | |
| E | 4S-A | 11C-B | 1 | 72 | 1.15 | |
| E | 7S-A | 8C-B | 1 | 72 | 0.60 | |
| E | 7S-A | 8C-B | Mixed | 54 | 1.02 | |
| I | 2S-B | 13C-A | 2 | 54 | 1.57 | 1.36 |
| I | 4S-B | 11C-A | 2 | 54 | 1.36 | |
| I | 2S-B | 13C-A | 2 | 72 | 1.66 | |
| I | 4S-B | 11C-A | 2 | 72 | 1.40 | |
| I | 7S-B | 8C-A | 2 | 72 | 1.23 | |
| I | 7S-B | 8C-A | Mixed | 54 | 0.92 | |

All three tables verify that the SCC transfer lengths are usually longer than the corresponding CVC transfer lengths. However, it is possible to see that not only does SCC produce longer transfer lengths, but the effect is more pronounced when coupled with the effect of the concrete placement sequence or the girder orientation on the prestressing bed. As mentioned previously, it is difficult to separate the effects of the placement sequence from the girder orientation as the two are coupled in ten of the twelve girders.

Girgis and Tuan (2005) speculated that the additional length of SCC transfer lengths is due to the presence of VMA, which is one of the admixtures included in the SCC mixtures to produce its characteristic properties. While they cite the VMA as being the only varying factor between the tests performed on the SCC specimens and the CVC specimens, there were several other variable factors that could have contributed to the differences seen.

Comparisons of the results of this study were also made with the results from the previous studies. In each case, transfer lengths for SCC mixtures including less than 30 percent GGBF slag, the only type of SCC used in conjunction with the bulb-tee girders in this study, were normalized with respect to the conventional mixture associated with each particular study, with all studies being considered at seven days. The one exception was the results from Levy's (2007) study in which case the values were taken at four days. The averaged results can be seen in Table 5-14.

**Table 5-14: Comparison of Normalized α Values**

| Study | $\frac{\alpha_{SCC}}{\alpha_{CVC}}$ |
|---|---|
| Prisms (Swords 2005) | 1.28 |
| T-Beams (Levy 2007) | 1.04 |
| AASHTO Type I (Boehm 2008) | 1.07 |
| BT-54 (current study) | 1.25 |
| BT-72 (current study) | 1.19 |
| Bulb-Tee (current study) | 1.22 |

While there is a discrepancy in how much longer the transfer lengths are in SCC than CVC (4 percent –28 percent longer), it is apparent that SCC does have an effect on the length of the transfer lengths. It was previously hypothesized by Boehm (2008) that the varying factor for the degree of elongation was dependent on the size of the specimens. This could be the case as results of this study do see this trend, first with the bulb-tee girders as the transfer lengths were 22 percent longer in SCC than CVC compared to being 7 percent longer in the AASHTO Type I girders examined by Boehm (2008) and 4 percent longer in the T-beams studied by Levy (2007). However, the larger BT-72 girders saw a smaller difference in the transfer lengths (19 percent) than those found in the smaller BT-54 girders (25 percent). In addition, Swords (2005) found the largest difference between the SCC and the CVC, which were related to the smallest cross sections examined.

### 5.1.9 Effect of Strand Size

For the first time in Auburn University SCC studies, a variation in the prestressing strand was considered. Two different strand sizes were used: ½-inch strand, which has a diameter of 0.50 in., and ½-inch ‘special’ strand, which has a diameter of 0.52 in., with the ½-inch being used in the BT-54 girders and the ½-inch ‘special’ in the BT-72 girders. Previous projects only used the ½-inch ‘special’ prestressing strand. Given the way both ACI and AASHTO phrase their provisions, it is not uncommon for transfer lengths to be measured in terms of the diameter of the strand. Table 5-15 shows the fully bonded, 28-day transfer lengths of the different girders in terms of the number of diameters seen. However, it is also necessary to consider the effects of the concrete strength and the prestress intensity, the results of which can be seen in Table 5-16, which shows the α values corresponding to Table 5-15.

**Table 5-15: Transfer Lengths as a Function of Strand Diameter**

| Strand $\phi$ (in.) | Concrete | Girder ID | No. of $d_b$ | Average No. of $d_b$ | |
|---|---|---|---|---|---|
| 0.50 | SCC | 2S | 33 | 32 | 29 |
| | | 4S | 36 | | |
| | | 7S | 27 | | |
| | CVC | 8C | 29 | 27 | |
| | | 11C | 27 | | |
| | | 13C | 25 | | |
| 0.52 | SCC | 2S | 33 | 30 | 28 |
| | | 4S | 32 | | |
| | | 7S | 26 | | |
| | CVC | 8C | 29 | 26 | |
| | | 11C | 24 | | |
| | | 13C | 25 | | |

**Table 5-16: Transfer Length α Values as a Function of Strand Diameter**

| Strand ϕ (in.) | Concrete | Girder ID | α (ksi$^{-0.5}$-in.) | Average α (ksi$^{-0.5}$-in.) | Average α (ksi$^{-0.5}$-in.) |
|---|---|---|---|---|---|
| 0.50 | SCC | 2S<br>4S<br>7S | 0.51<br>0.56<br>0.41 | 0.50 | 0.45 |
| | CVC | 8C<br>11C<br>13C | 0.42<br>0.39<br>0.39 | 0.40 | |
| 0.52 | SCC | 2S<br>4S<br>7S | 0.50<br>0.48<br>0.40 | 0.46 | 0.43 |
| | CVC | 8C<br>11C<br>13C | 0.44<br>0.37<br>0.39 | 0.40 | |

As can be seen from the chart, there is no distinction in the transfer lengths between the two strand sizes. If anything, the transfer lengths found with the ½-inch 'special' strands were shorter than those found with the ½-inch strand, the opposite of what it was predicted to be. However, it must also be noted that the strand size was not independent from the size of the girder as the larger strand was only used in the larger girders. Also, the different sized strands came from different manufacturers, which might also have had an impact on the results.

The results were also grouped in terms of similar concretes rather than similar strand diameters and can be seen in Table 5-17. As can be seen from the table, the effect of the type of concrete has a much larger effect on the number of strand diameters required to estimate the transfer length than does the strand diameter.

**Table 5-17: Transfer Lengths as a Function of Strand Diameter and Concrete Type**

<table>
<tr><th rowspan="2">Concrete</th><th rowspan="2">Strand<br>$\phi$ (in.)</th><th rowspan="2">Girder<br>ID</th><th>$\alpha$</th><th colspan="2">Average $\alpha$</th></tr>
<tr><th>(ksi$^{-0.5}$-in.)</th><th>(ksi$^{-0.5}$-in.)</th><th>(ksi$^{-0.5}$-in.)</th></tr>
<tr><td rowspan="6">SCC</td><td rowspan="3">0.5</td><td>2S</td><td>0.51</td><td rowspan="3">0.50</td><td rowspan="6">0.45</td></tr>
<tr><td>4S</td><td>0.56</td></tr>
<tr><td>7S</td><td>0.41</td></tr>
<tr><td rowspan="3">0.52</td><td>2S</td><td>0.50</td><td rowspan="3">0.40</td></tr>
<tr><td>4S</td><td>0.48</td></tr>
<tr><td>7S</td><td>0.40</td></tr>
<tr><td rowspan="6">CVC</td><td rowspan="3">0.5</td><td>8C</td><td>0.42</td><td rowspan="3">0.40</td><td rowspan="6">0.40</td></tr>
<tr><td>11C</td><td>0.39</td></tr>
<tr><td>13C</td><td>0.39</td></tr>
<tr><td rowspan="3">0.52</td><td>8C</td><td>0.44</td><td rowspan="3">0.40</td></tr>
<tr><td>11C</td><td>0.37</td></tr>
<tr><td>13C</td><td>0.39</td></tr>
</table>

### 5.1.10 Effect of Cross-Section Size

As mentioned, it is impossible to separate the strand diameter from the size of the girder. Evidence has shown that as cross-sections become larger, transfer lengths become shorter (Levy 2007, Boehm 2008). Consequently, a table comparing the α values for the two different sized girders produces the same table as that shown in Table 5-16, which supports the trend showing that larger cross-sections produce shorter transfer lengths.

As part of the extended study, several cross-sectional sizes were examined, with each study using significantly larger test specimen. Both Swords (2005) and Levy (2007) tested small, simple cross sections in order to obtain a better understanding of the different factors contributing to the transfer length. The members utilized included prisms and T-beams, respectively, and can be seen in detail in Sections 2.5.1 and 2.5.2. It then became necessary to consider girder cross sections that would typically be used in the field, resulting in the study by Boehm (2008) and the examination of AASHTO Type I girders. The latest study continued the investigation by looking at the larger bulb-tee cross-section, as well as considering two different sized bulb-tee girders.

To investigate this idea further, all phases of the investigation were compared. In order to make the most meaningful comparisons, only the prisms containing two prestressing strands from the research done by Swords (2005) were considered. In addition, given the range of ages at which the concrete was sampled, a similar age was used from each phase of experimentation. The transfer lengths from Swords (2005), Boehm (2008), and the current study were taken at seven days after release. Given that the longest time period considered by Levy (2007) was four days, the corresponding results from this concrete age were used. In addition, only transfer zones in SCC containing no more than 30 percent GGBF slag and CVC mixtures were considered, of which only those adjacent to flame-cutting of strands were included. When looking at the results of the AASHTO Type I study, the difference in lengths between the live-end and dead-end results that defined the live or dead end classification was deemed insignificant and all of the zones were included despite the result of the flame-cutting process. Consequently, four zones considered by Swords (2005), eight zones considered by Levy (2007), and twelve zones considered by Boehm (2008) were compared to twelve fully bonded BT-54 zones and twelve fully bonded BT-72 zones obtained from this study. The results can be seen in Figure 5-4.

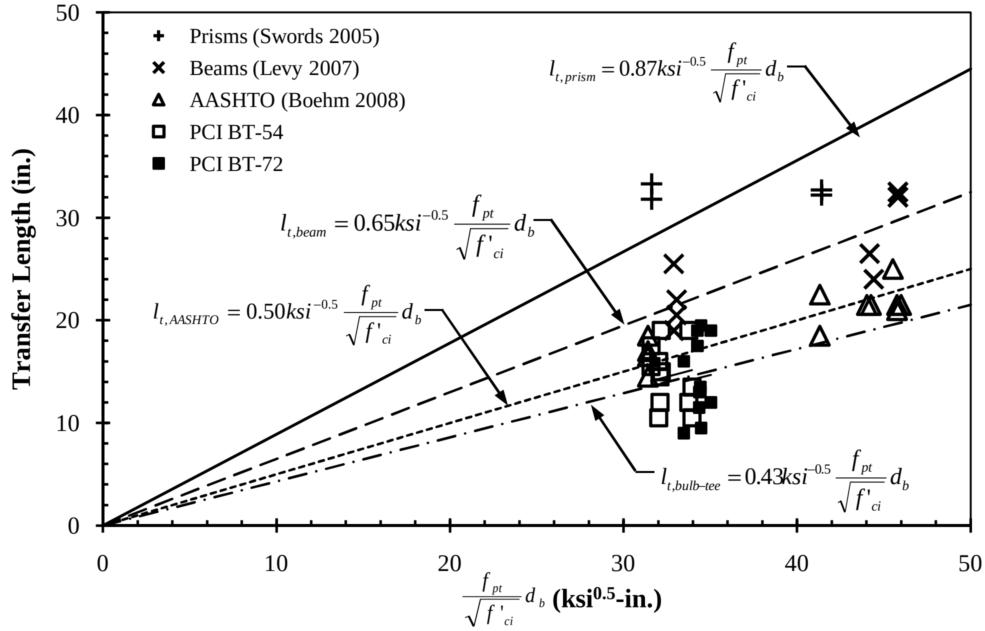



**Figure 5-4: Effect of Cross-Section Size on Transfer Lengths**

As can be seen from the graph, as well as the average proportionality constants for each specimen type, the transfer lengths became smaller as the magnitude of the cross section increased. However, all of the full-scale girders tested, including the AASHTO Type I, the PCI BT-54, and the BT-72 girders, resulted in similar results, which can be seen by the grouping that occurred amongst the full-scale girders in Figure 5-4. Further comparison and breakdown of the average α values can be seen in Table 5-18 with a graphical comparison of the SCC and CVC α values shown in Figure 5-5.

**Table 5-18: Summary of Normalized α Values**

| | α Value (ksi$^{-0.5}$-in.) | |
|---|---|---|
| **Specimen** | **SCC** | **CVC** |
| Prisms (Swords 2005) | 1.03 | 0.78 |
| T-Beam (Levy 2007) | 0.66 | 0.64 |
| AASHTO Type I (Boehm 2008) | 0.50 | 0.49 |
| PCI BT-54 (current study) | 0.49 | 0.41 |
| PCI BT-72 (current study) | 0.46 | 0.37 |

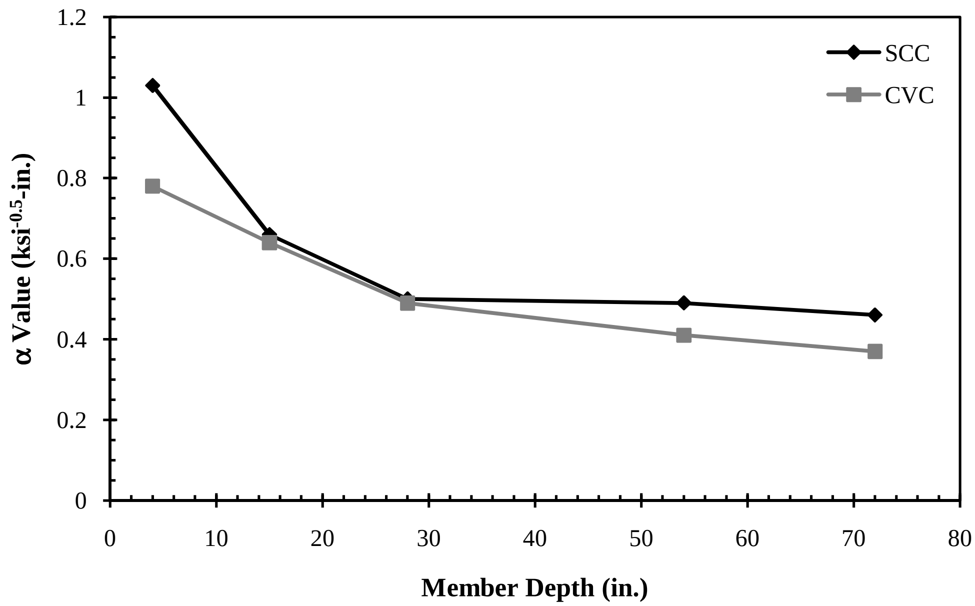



**Figure 5-5: Normalized α Values with Respect to Member Depth**

Each of the specimens produced a slightly smaller proportionality factor than the next smaller specimen. The smaller of the bulb-tee girders produced a proportionality factor about 2 percent shorter than the AASHTO Type I girders in SCC, while the BT-72 girders produced even shorter transfer lengths than the BT-54 girders (a further 6 percent reduction). This was a fairly insignificant change when compared to the decrease seen from the small-scale SCC prism α value of 1.03 to 0.49 for the bulb-tee girders, as well as the fact that a variation of 10% in these transfer lengths is almost within the limits of precision of the transfer-length determination process. This confirms the statements made by Russell and Burns (1993) in which they conclude that larger cross sections will produce shorter transfer lengths than those produced in the small sample prisms that had typically been tested up to that point as a result of the larger mass being able to absorb more energy.

Relooking at the information presented by Girgis and Tuan (2005), they attribute the presence of VMA to the longer transfer lengths measured in the SCC girders when compared to the CVC girders. However, several other important variables can be seen between the two projects. First, in addition to the absence of VMA, the CVC mixture used contained a reduced amount of HRWR admixture. Second, the cross section used in conjunction with the shorter CVC transfer lengths was 50 percent taller than the one used with SCC. Finally, the girders cast with SCC contained concrete that was 1000 psi weaker at transfer than the CVC girders. From the results found in this thesis and previous works, particularly Swords (2005), Levy (2007), and Boehm (2008), both the larger cross section and the stronger concrete in the CVC girders examined by Girgis and Tuan (2005) could account for the difference in transfer lengths they saw. Also, as part of this research, the amount of VMA was increased from the amount used in the BT-54 girders to the amount used in the BT-72 girders. Of all the trends seen, the BT-72 girders produced shorter transfer lengths than the BT-54 girders, the opposite of what Girgis and Tuan (2005) suggest would be seen due to a larger quantity of VMA. Consequently, there was no evidence to suggest that the presence or amount of VMA used in an SCC mixture has any impact on the transfer lengths.

## 5.2 Comparison of Test Data with Design Expressions

It was necessary to re-evaluate the accuracy with which the design codes predicted the transfer length values and the potential for the need to adjust the current code provisions. As done previously, both the ACI 318-08 and the AASHTO LRFD and AASHTO Standard code provisions were examined. In addition, recommendations made by Levy (2007) for transfer length predictions were also considered.

### 5.2.1 ACI 318-08, Section 12.9

The first expression considered was that found in Section 12.9 of ACI's *Building Code Requirements for Structural Concrete (ACI 318-08)* in which the transfer length is described to be:

$$l_t = \left(\frac{f_{pe}}{3000}\right) d_b \qquad \textbf{\textit{Equation 5-5}}$$

Equation 5-5 was used to predict the transfer lengths based upon the parameters in association with each girder. A value of 165 ksi was estimated to be the effective stress in the prestressing steel after all loses ($f_{pe}$). While two different strand diameters were used, the effective prestressing was constant for this relationship. Consequently, two values were obtained through the equation, with there being only a difference of 1.2 in. between them. In order to consider the relationship between the measured transfer lengths and those predicted by Equation 5-5, a ratio was taken between the two values by dividing the measured transfer length by the predicted transfer length. As the ratio approaches one, the more unconservative the expression becomes. Figure 5-6 and Figure 5-7 show the ratios of both the fully bonded and the debonded-strand transfer lengths, respectively, as no special provisions are made for debonded strands.

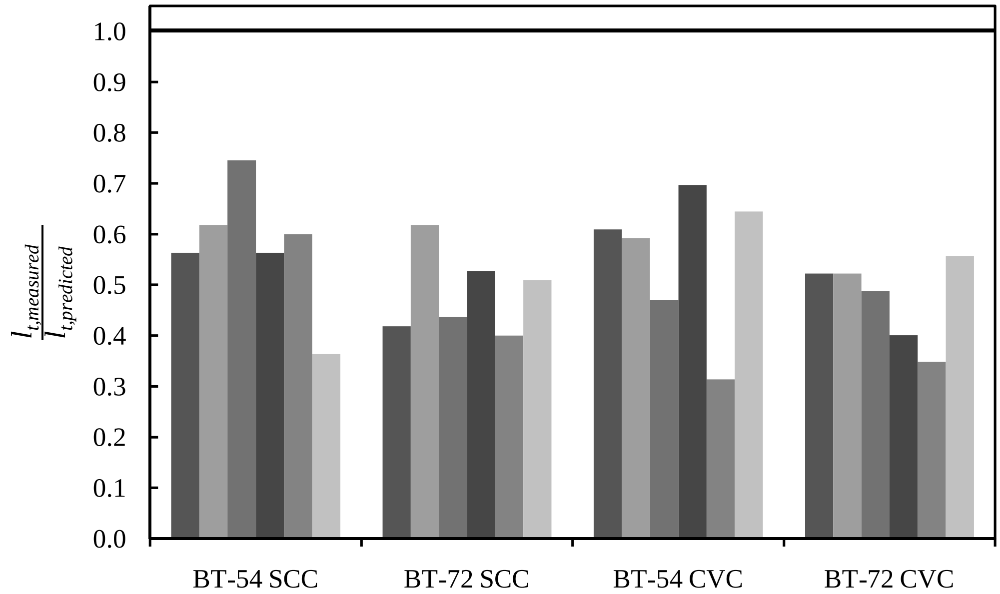



**Figure 5-6: Comparison to ACI Expression for Fully Bonded Strands**

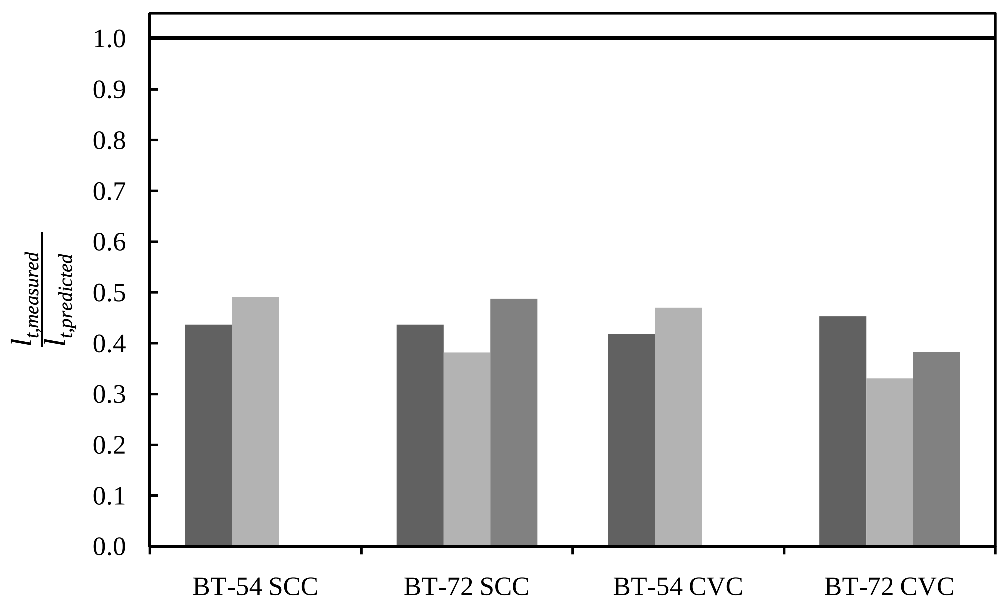



**Figure 5-7: Comparison to ACI Expression for Debonded Strands**

As can be seen, Equation 5-5 is very conservative in comparison to the collected results. On average, the measured values were 52% that of the predicted lengths. The ratios ranged from 0.31 to 0.75, values that are well below the threshold. The debonded-strand transfer lengths were even shorter than their fully bonded counterparts as they were only 43% that of the predicted values.  Unlike other code provided expressions in use, Equation 5-5 does consider the effective stress in the strand, not just the diameter of the strands.

### 5.2.2 AASHTO Standard and ACI 318-08 Shear Provisions

Both the *Standard Specifications for Highway Bridges* (2002) by AASHTO and the shear provisions in Sections 11.3.4 and 11.3.5 of ACI's *Building Code Requirements for Structural Concrete (ACI 318-08)* dictate that the transfer length be taken as:

$$l_t = 50d_b \qquad \textbf{\textit{Equation 5-6}}$$

As before, the transfer lengths were calculated based upon this expression and compared to the measured values via ratios, the level of conservatism being illustrated by Figure 5-8 and Figure 5-9.

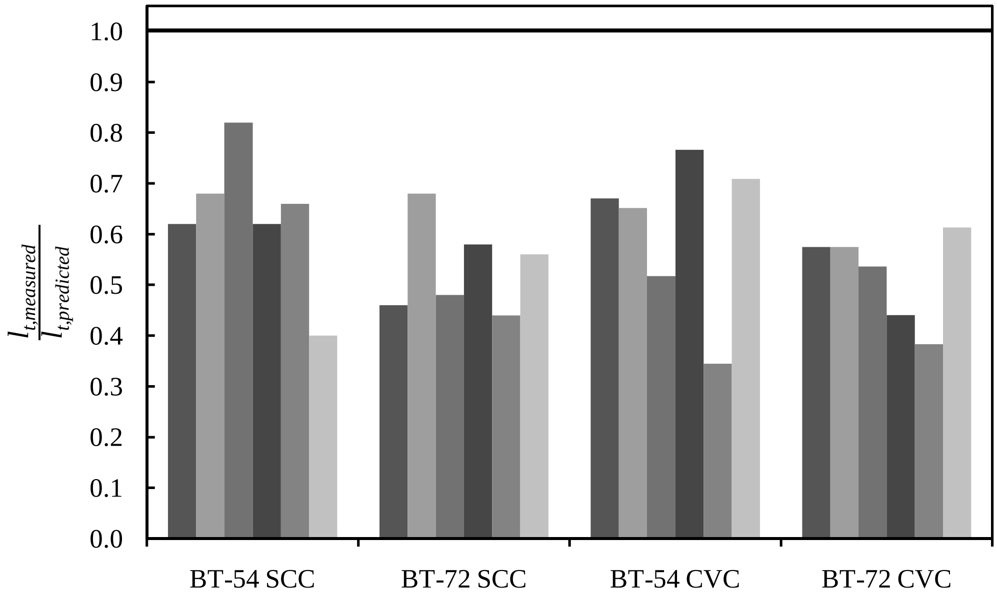



**Figure 5-8: Comparison to AASHTO Standard and ACI 318-08 Section 11.3.4 Expression for Fully Bonded Strands**

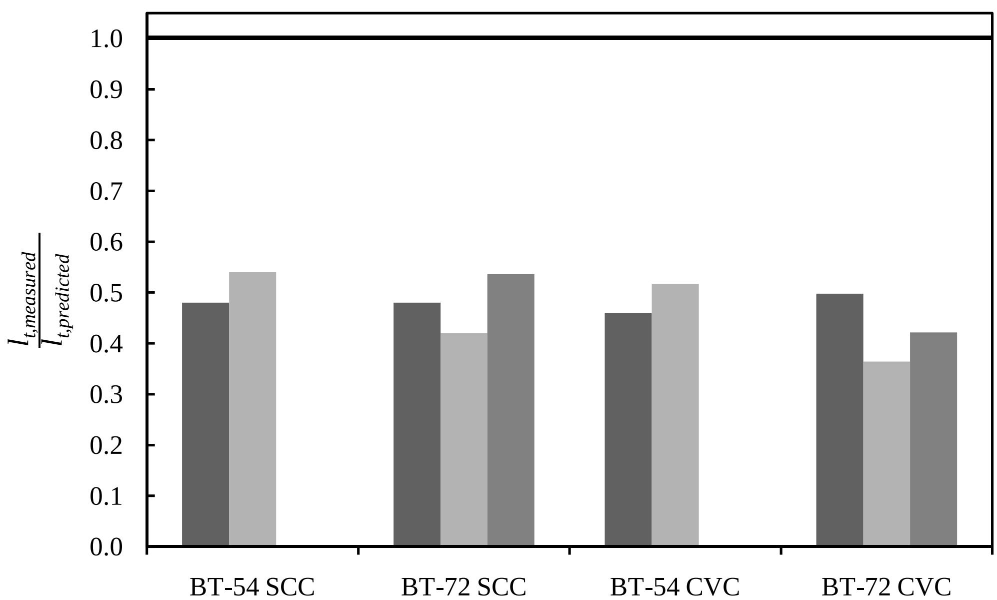



**Figure 5-9: Comparison to AASHTO Standard and ACI 318-08 Section 11.3.5 Expression for Debonded Strands**

While closer to the line of equality than Equation 5-5, and thus, technically, more accurate, it is still clear that employing Equation 5-6 to calculate the transfer length is still very conservative and did not reflect the trend of the results.  In addition, both Levy (2007) and Boehm (2008) found the expression to be unconservative for moderate-strength concrete mixtures.  This was not an issue for this study as moderate-strength concrete was not tested, and further investigation into this trend may be warranted. While developed for concretes with moderate strength (Hanson and Kaar 1959), the expression was shown to be more than adequate and a conservative approximation for strength design of the transfer length in high-strength concretes, although it was inaccurate.

In this comparison, the fully bonded strands averaged to be 57% of the values predicted by the expression.  While less conservative than Equation 5-5, Equation 5-6 is still a very conservative estimate for high-strength concretes.  The debonded strands were also compared to Equation 5-6 as Section 11.3.5 directly addresses the transfer length in debonded strands and declares that it too shall be taken as $50d_b$.  As seen before, the expression was even more conservative for the debonded strands, as they were found to be 47% of the predicted values.

### 5.2.3 AASHTO LRFD

AASHTO's *LRFD Bridge Design Specifications* (2010) take the most conservative approach for calculating the transfer length for strength design, specifying that the expression that should be used is:

$$l_t = 60d_b \qquad \textbf{\textit{Equation 5-7}}$$

The transfer lengths were again calculated based upon Equation 5-7 for the girders. Figure 5-10 and Figure 5-11, which show the comparison of the measured transfer lengths to the transfer lengths predicted by Equation 5-7.

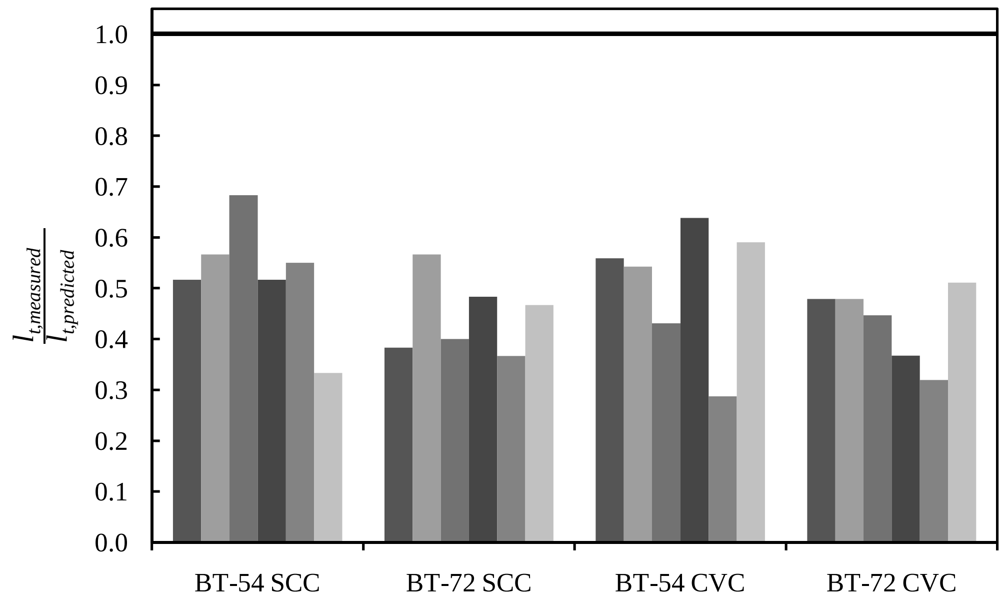



**Figure 5-10: Comparison to AASHTO LRFD Expression for Fully Bonded Strands**

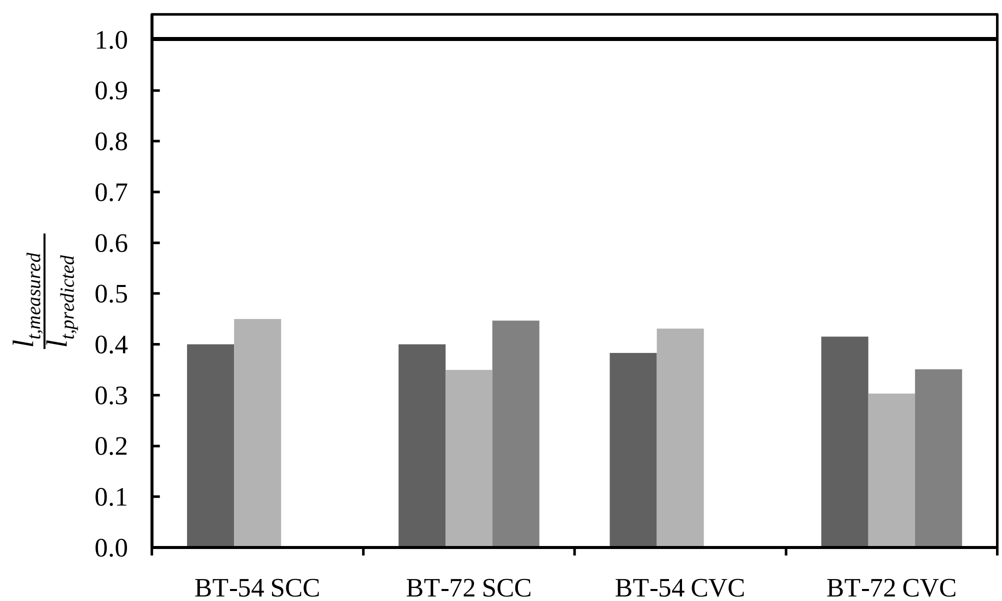



**Figure 5-11: Comparison to AASHTO LRFD Expression for Debonded Strands**

Not only was Equation 5-7 very conservative, but it also did not take into account any of the parameters that are shown to have an effect on the transfer length besides the strand diameter.  On average, the fully bonded strands measured to be only 48% that of the predicted length, with the largest ratio being 0.68, which was still more than 30% less than the predicted value.  The debonded strands measured an average of 60% less than the predicted values, and were the most conservative estimates except for those found by Equation 5-5.

### 5.2.4 Levy (2007)

Given the accuracy with which Equation 5-4 represented the data, Levy (2007) took the information she collected and determined values for the proportionality constant which could be used for design and strength calculations involving transfer lengths. From her testing, three general categories were formed based upon the properties of the specimen, as the properties of the member would alter the proportionality factor that would be needed.  First, it was determined that the method of release had an impact on the $\alpha$ value.  A gradual release method caused shorter transfer lengths, and thus could have a smaller $\alpha$ value.  This method of release had more impact on the transfer length than any of the concrete properties.  Consequently, only one $\alpha$ value is given for this category.  For members that underwent sudden release, the concrete composition did have an impact on the transfer length.  For concrete that contained unlimited amounts of GGBF slag but more than 30%, a larger $\alpha$ value was required.  A single $\alpha$ value was assigned to all other concretes, including SCC and CVC, which underwent sudden release.  The specimens in this study all fell into the last category.

Finally, dependent on the type of analysis being done, two different proportionality constants were used. As discussed previously, it is conservative to overestimate the transfer length for strength design purposes as it "results in an underestimation of the flexural strength and shear strength" (Levy 2007). Consequently, an upper-bound value was determined that encompassed 95% of the values collected by Levy (2007). Conversely, when calculating allowable stresses in the concrete immediately after transfer, a shorter transfer length is conservative. Consequently, an average α value was determined. The recommendations made by Levy (2007) to calculate the transfer length based on the two different analysis scenarios are detailed in Table 5-19.

**Table 5-19: Recommended Transfer Length Equations by Levy (2007)**

| Analysis Type | Transfer Length |
|---|---|
| Strength Design | $l_t = 0.78ksi^{-0.5}\dfrac{f_{pt}}{\sqrt{f'_{ci}}}d_b$ |
| Allowable Stress Limits | $l_t = 0.65ksi^{-0.5}\dfrac{f_{pt}}{\sqrt{f'_{ci}}}d_b$ |

Applying the transfer length expression used in association with strength design to the parameters of the specimens tested, the results were then compared to the experimentally determined lengths and can be seen in Figure 5-12. Unlike the results presented by Levy (2007) and Boehm (2008), the results of this study do not reflect a trend in relation to the predicted transfer lengths. This may be largely due to the fact that a small range of concrete strength was used, whereas an expression that accounted for the changes in concrete strength proved very effective in the previous studies. Despite this observation, Levy's (2007) equation was the only one that did account for the different parameters that effect transfer length, which was reflected in the fact that the equation did a slightly better job at predicting the trend of the data than any of the other expressions considered.

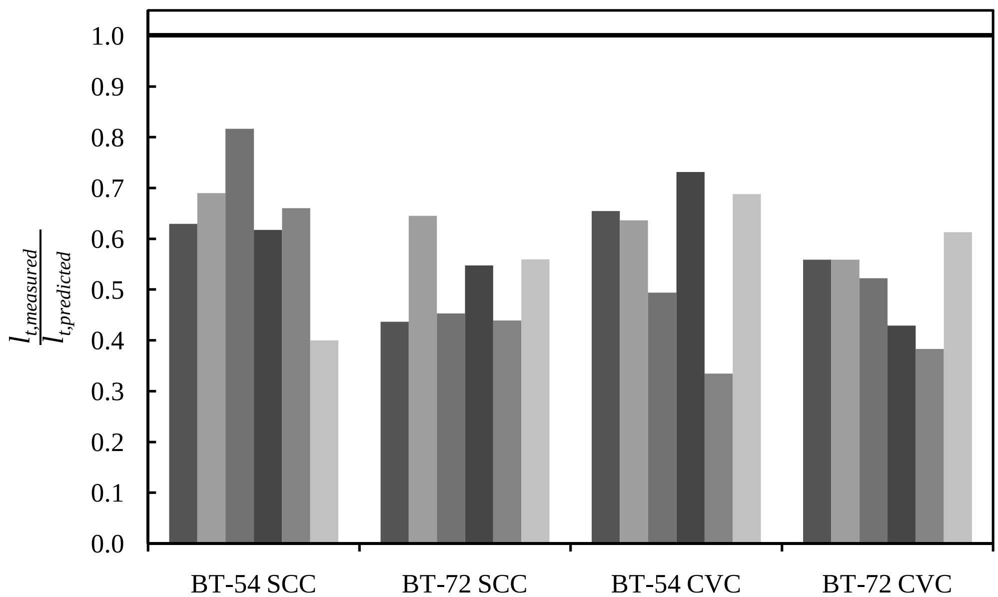



**Figure 5-12: Comparison to Levy Expression for Fully Bonded Strands**

As can be seen by the graph, the α value determined by Levy is very conservative and each measured transfer length lies well within its boundaries. While it might be appropriate to reduce the upper bound α value of 0.78 ksi$^{-0.5}$-in. slightly to a more general ¾ ksi$^{-0.5}$-in., the limited scope of this study could not account for effects that could result in relation to variations in concrete strength or other parameters not examined here that were considered by Levy (2007). In addition, as the equation still produces a conservative estimation for use in strength design, a change in the equation is not warranted. Ultimately, Levy's (2007) equation represents a conservative approximation for transfer lengths for strength design.

In research presented by Boehm (2008), there was concern that the expression may not adequately represent the transfer lengths in high-strength concrete to a satisfactory degree.  However, given that this study only considered high-strength concrete, it can be concluded that this is not an issue.  Figure 5-12 depicts the level of conservatism of the expression established by Levy (2007).  It can be seen that the expression is very conservative for the given set of results as the largest ratio was 0.82 with an average of 0.56, and the smallest ratio being 0.33.

As debonded strands were never specifically addressed in the code provisions, Levy's (2007) expression was also applied to the debonded-strand transfer length values. Figure 5-13 shows how the expression is even more conservative for the debonded strands than for the fully bonded strands.

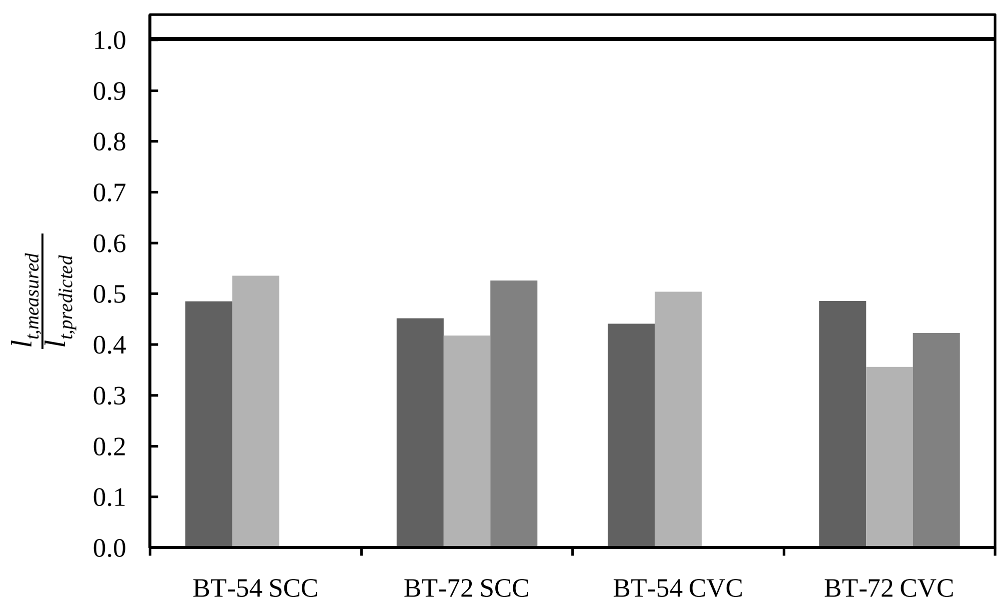



**Figure 5-13: Comparison to Levy Expression for Debonded Strands**

In situations in which the allowable stresses are being determined, it is appropriate to use an average value for α rather than an upper bound.  Levy (2007) specified that the average should be 0.65 ksi$^{-0.5}$-in as seen in Table 5-19, and corresponds with Figure 5-4. However, this value is still an overestimate, not an average of the full-scale girders. As the AASHTO Type I girders are the smallest girders that may be used in practice by ALDOT and the bulb-tee girders are the largest, an average value based upon these values is logical. As seen in Table 5-18, the α values range from 0.50 ksi$^{-0.5}$-in to 0.41 ksi$^{-0.5}$-in. As the AASHTO Type I girders produced the longest transfer lengths, an appropriate α value that is more reflective of full-scale girders for use in calculating the allowable stresses in a girder is ½ ksi$^{-0.5}$-in. Consequently, Equation 5-8 exhibits the modified equation.

$$l_t = \frac{1}{2} ksi^{-0.5} \frac{f_{pt}}{\sqrt{f'_{ci}}} d_b \qquad \textbf{\textit{Equation 5-8}}$$

### 5.2.5 Summary of Code Comparisons

The information presented in Section 5.2.1 through Section 5.2.4 is summarized in the following table:

**Table 5-20: Summary of Code Provision Comparison**

| | | $l_{t,measured}/l_{t,predicted}$ | | | |
|---|---|---|---|---|---|
| | | **Fully Bonded** | | **Debonded** | |
| **Source** | **Equation** | **Range** | **Average** | **Range** | **Average** |
| ACI 318-08 (12.9) | $\left(\frac{f_{pe}}{3000}\right)d_b$ | 0.31-0.75 | 0.52 | 0.33-0.49 | 0.43 |
| ACI 318-08 (11.3) & AASHTO Standard | $50d_b$ | 0.34-0.82 | 0.57 | 0.36-0.54 | 0.47 |
| AASHTO LRFD | $60d_b$ | 0.29-0.68 | 0.48 | 0.30-0.45 | 0.39 |
| Levy | $0.78ksi^{-0.5}\frac{f_{pt}}{\sqrt{f'_{ci}}}d_b$ | 0.33-0.82 | 0.56 | 0.36-0.54 | 0.46 |

# Chapter 6 Summary and Conclusions

## 6.1 Summary

While the use of SCC is not widespread, it continues to gain popularity as its advantages for use in construction are continually recognized due to its ability to flow around and through reinforcement without the use of vibration.  Consequently, a movement to better understand its structural behavior is underway as multiple entities across the United States work to perform the research necessary to allow its implementation on a broad scale.  In doing so, the precast/prestressed industry would be able to take advantage of the many benefits of using SCC, including reduced labor costs, increased worker safety, increased productivity, and improved product aesthetics.

As part of this movement, the Auburn University Highway Research Center performed this study on precast/prestressed bridge girders cast with SCC in conjunction with the previous phases for the investigation sponsored by the Alabama Department of Transportation.  The three phases previously completed included an in-depth evaluation of bond behavior between prestressing strand and SCC in regards to transfer length, development length, and flexural behavior.  Complete and thorough discussions regarding the results and conclusions of these phases were reported by Swords (2005), Levy (2007), and Boehm (2008).  This phase of the investigation focused on results obtained from actual bridge girders, which can later be associated with the bridge's overall structural performance.

The scope of this investigation was to examine the transfer lengths found in twelve of the twenty-eight precast/prestressed girders cast for the SR 22 bridge over Hillabee Creek in Tallapoosa County, Alabama. Of these twelve girders, three BT-54 girders and three BT-72 girders were cast with SCC, as well as three BT-54 girders and three BT-72 girders which were cast with CVC. All of the girders contained Grade 270, seven-wire, low-relaxation prestressing strand, with ½-inch strand being used in the BT-54 girders and ½-inch 'special' strand being used in the larger BT-72 girders. The girders were then cast at Hanson Pipe & Prestress Plant in Pelham, Alabama. Production began September 21, 2010 and concluded October 28, 2010 and incorporated the cast-in-place transfer length testing apparatus created specifically for this project.

Once cast, surface compressive strain measurements were taken for each girder before transfer and after transfer, as well as seven days and twenty-eight days after transfer. Ten of the twelve girders contained three transfer zones, with one transfer length for the fully bonded strands located at each end of the girder, as well as a single debonded-strand transfer length that was measure at the mark end of each girder. Two of the girders only contained two transfer zones, one fully bonded zone at each end. The concrete compressive strain measurements were than analyzed according to the 95% AMS method and transfer lengths were determined for each transfer zone. These results were then looked at in light of the type of concrete used, the effects of time, the effect of cross-section size, concrete strength, and with consideration of the plant casting process and girder orientation. When appropriate, the results of this phase of testing was also compared to the results found by Swords (2005), Levy (2007), and Boehm (2008) to create a more comprehensive understanding of the bond behavior in SCC. Finally, these results were also compared to current code provisions set forth by both ACI and AASHTO, as well as recommendations made by Levy (2007) for appropriate predictive expressions.

**6.2 Conclusions**

The following conclusions concerning the transfer lengths found in SCC were made based on the results seen in this study.

- Transfer lengths continue to grow for weeks after the release of the prestressing force. Significant growth occurs between the initial and seven-day measurements with minimal growth occurring over the weeks that follow.
- Trustworthy estimates of long-term transfer lengths cannot be determined based solely on measurements of initial transfer length, because transfer lengths increased 18% on average over the course of twenty-eight days after prestress release.
- Transfer lengths experienced more growth when cast in CVC compared to SCC.
- Transfer lengths of partially debonded strands grew less over time when compared to transfer lengths associated with fully bonded strands.
- In general, the debonded-strand transfer lengths were found to be shorter than those of the fully bonded strands. There is no reason to adjust the

code provisions for the transfer length of debonded strands in concrete not subject to cracking within the transfer zones.

- Girders cast with SCC see larger discrepancies between transfer lengths found in the end of the girder in which concrete was placed first compared with the end placed second.
- Transfer lengths adjacent to long lengths of exposed strand are more likely to be longer than the transfer lengths of girders adjacent to short lengths of exposed strand.  This is independent of the use of SCC or CVC.
- While there appears to be evidence to support the influence of concrete placement order and girder orientation on the transfer lengths, the effects are interrelated and cannot be separated.  Consequently, further research on these topics are recommended.
- The effect of concrete strength and variability in prestress force on the transfer length can efficiently be accounted for by determining α values in terms of $\frac{f_{pt}}{\sqrt{f'_{ci}}} d_b$ in units of ksi$^{-0.5}$-in.
- For fully bonded and debonded strands examined in this study, SCC produced transfer lengths 20 percent and 18 percent longer than CVC, respectively.
- There is no apparent correlation to how much longer transfer lengths are in SCC when compared to CVC.  However, larger cross-sections have tended to produce larger differences in how much longer the SCC transfer lengths are than CVC transfer lengths.

- There is no significant difference between transfer lengths associated with different strand diameters; the type of concrete (SCC or CVC) has a much larger effect.
- Cross-sectional size does play a role in the transfer length as larger cross-sections produce shorter transfer lengths.
- The effect larger cross sections have on the transfer length is less pronounced among specimens of varying full-scale girders sizes than was seen between test prisms and the full-scale girders.
- While additional transfer length was associated with SCC, all methods of predicting the transfer length were conservative enough that the additional length was always accounted for.
- There is no evidence to suggest that the presence or amount of VMA used in an SCC mixture has any impact on the transfer lengths
- All recommended equations used by both ACI and AASHTO are conservative but inaccurate for high-strength concrete. Based upon previous research, it was seen that the equations were often unconservative for lower-strength concretes.
- Levy's (2007) expression for transfer length in concrete with no more than a 30 percent replacement of GGBF slag,

  $$l_t = 0.78ksi^{-0.5}\frac{f_{pt}}{\sqrt{f'_{ci}}}d_b$$

  for strength design provided the most accurate representation of the trend of the transfer lengths for this, and previous, studies as it accounted for release method, concrete strength, and stress in the prestressing strand. While still very conservative for the lengths collected in association with this investigation, justification cannot be made to decrease the α values dictated by Levy (2007).

- For calculating allowable stresses, Levy's (2007) expression was modified as follows:

$$l_t = \frac{1}{2} ksi^{-0.5} \frac{f_{pt}}{\sqrt{f'_{ci}}} d_b$$

## 6.3 Recommendations for Future Study

After completion of the study, the following recommendations can be made for future investigations:

- The effects of the sequence of concrete placement and orientation of the girder on the prestressing beds should be further evaluated.
- *$50d_b$* was found to be unconservative for moderate-strength concrete (Levy 2007). This trend should be investigated further.
- The accuracy of the modification made to Levy's (2007) allowable strength transfer length relationship should be confirmed.

## References

AASHTO. 2010. *AASHTO LRFD Bridge Design Specifications: Customary U.S. Units.* 5th ed. Washington D.C.: American Association of State Highway and Transportation Officials (AASHTO).

AASHTO. 2002. *Standard Specifications for Highway Bridges.* 16th ed. Washington D.C.: American Association of State Highway and Transportation Officials (AASHTO).

ACI Committee 318. 1963. *Building Code Requirements for Reinforced Concrete (318-63).* Detroit: American Concrete Institute (ACI).

ACI Committee 318. 1999. *Building Code Requirements for Reinforced Concrete (318-99) and Commentary (318R-99).* Farmington Hills, Michigan: American Concrete Institute (ACI).

ACI Committee 318. 2008. *Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary*. Farmington Hills, Michigan: American Concrete Institute (ACI).

ASTM. 2002. *Standard Test Method for Creep of Concrete in Compression*. *ASTM C512-02*. West Conshohocken, Pennsylvania: American Society for Testing and Materials.

Barnes, R. W., N. H. Burns, and M. E. Kreger. 1999. "Development Length of 0.6-Inch Prestressing Strand in Standard I-Shaped Pretensioned Concrete Beams." *Research Report No. 1388-1.* Austin: Center for Transportation Research, The University of Texas at Austin.

Barnes, R. W., J. W. Grove, and N. H. Burns. 2003. Experimental Assessment of Factors Affecting Transfer Length. *ACI Journal* 100, no. 6: 740 – 748.

Bennett, W. B., Jr. 1963. Preliminary Draft of the Proposed Revision of Section 211 – Bond anchorage of the Tentative Recommendations for Prestressed Concrete. Letter to the member of ACI Committee 423.

Boehm, Kurtis. 2008. *Structural Performance of Self-Consolidating Concrete in AASHTO Type I Prestressed Girders*. M.S. Thesis, Auburn University.

Canfield, Scott. 2005. *Full Scale Testing of Prestressed, High Performance Concrete, Composite Bridge Girders*. M.S. Thesis, Georgia Institute of Technology.

Girgis, A. F. M., C. Y. Tuan. 2005. Bond Strength and Transfer Length of Pretensioned Bridge Girders Cast with Self-Consolidating Concrete. *PCI Journal* 50, no. 6: 72-87.

Hanson, N. W. and P. H. Kaar. 1959. Flexural Bond Tests of Pretensioned Prestressed Beams. *ACI Journal. Proceedings* 55, no. 7: 783 – 802.

Kaar, P. H., R. W. LaFraugh, and M. A. Mass. 1963. Influence of Concrete Strength on Strand Transfer Length. *PCI Journal* 8, no. 5: 47 – 67.

Kaar, P., and D. Magura. 1965. Effect of Strand Blanketing on Performance of Pretensioned Girders, *PCI Journal* 10, no. 6: 20-34.

Levy, Kelly. 2007. *Bond Behavior of Prestressed Reinforcement in Beams Constructed with Self-Consolidating Concrete*. M.S. Thesis, Auburn University.

Logan, D. R. 1997. Acceptance Criteria for Bond Quality of Strand for Pretensioned Prestressed Concrete Applications. *PCI Journal* 42, no. 2: 52-90.

Martin, L. D., and N. L. Scott. 1976. Development of Prestressing Strand in Pretensioned Members. *ACI Journal. Proceedings* 73, no. 8: 453-456.

Mitchell, D., W. D. Cook, A. A. Kahn, and T. Tham. 1993. Influence of High-Strength Concrete on Transfer and Development Length of Pretensioning Strand. *PCI Journal* 38, no. 3: 52–66.

Naito, C. J., G. Parent, and G. Brunn. 2006. Performance of Bulb-Tee Girders Made with Self-Consolidating Concrete. *PCI Journal* 51, no. 6: 72-85.

Okamura, H. and Ouchi. 1999. “Self-Compacting Concrete, Development, Present Use and Future.” In *Self-Compacting Concrete: Proceedings of the First International RILEM Symposium*, ed. A. Skarendahl and O. Petersson. Cachan Cedex, France: RILEM Publications, 3-14.

Ozyildirim, C. 2008. “Bulb-T Beams with Self Consolidating Concrete on the Route 33 Bridge Over the Pamunkey River in Virginia.” *Final Research Report No. FHWA/VTRC 09-R5*. Charlottesville: Virginia Transportation Research Council.

PCI. 2003. *Interim Guidelines for the Use of Self-Consolidating Concrete in Precast/Prestressed Concrete Institute Member Plants*. 2nd Ed. United States of America: Precast/Prestressed Concrete Institute (PCI).

Pozolo, A., and B. Andrawes. 2011. Analytical Prediction of Transfer Length in Prestressed Self-Consolidating Concrete Girders using Pull-Out Test Results. *Construction and Building Materials* 25, no. 2: 1026-1036.

Rabbat, B. G., P.H. Kaar, H. G. Russell, R. N. Bruce Jr. 1979. Fatigue Tests of Pretensioned Girders with Blanketed and Draped Strands. *PCI Journal* 24, no. 4: 88-114.

Russell, Bruce W., and Ned H. Burns. 1993. Design Guidelines for Transfer, Development and Debonding of Large Diameter Seven Wire Strands in Pretensioned Concrete Girders. Research Report 1210-5F. Austin: Center for Transportation Research, The University of Texas at Austin.

Staton, B. W., N. H. Do, E. D. Ruiz, W. M. Hale. 2009. Transfer Lengths of Prestressed Beams Cast with Self-Consolidating Concrete. *PCI Journal* 54, no. 2: 64-83.

Swords, Jesse. 2005. *Transfer Length in Prestressed Self-Consolidating Concrete*. M.S. Thesis, Auburn University.

Tabatabai, H. and T. J. Dickson. 1995. "The History of the Prestressing Strand Development Length Equation." *Report No. FHWA-RD-93-076*. McLean, Virginia: Federal Highway Administration.

Ziehl, P. H., D. C. Rizos, J. M. Caicedo, F. Barrios, R. B. Howard, A. S. Colmorgan. 2009. "Investigation of the Performance and Beefits of Lightweight SCC Prestressed Concrete Bridge Girders and SCC Materials." *Report No. FHWA-SC-09-02*. Columbia, South Carolina: University of South Carolina.

## Appendix A: Notation

| | |
|---|---|
| $d_b$ | Diameter of prestressing strand |
| $f'_{ci}$ | Concrete strength when the prestressing steel was released |
| $f_{pe}$ | Effective prestress force |
| $f_{pj}$ | Jacking force |
| $f_{ps}$ | Stress which produces $M_n$ |
| $f_{pt}$ | Stress in the prestressing steel after release |
| $l_d$ | Development length |
| $l_t$ | Transfer length |
| $l_{fb}$ | Flexural bond length |
| $M_n$ | Nominal flexural strength |
| $\phi$ | Diameter |

## Appendix B: Special Provision

## ALABAMA DEPARTMENT OF TRANSPORTATION

DATE: August 10, 2009 Special Provision No. 08-0498

SUBJECT: Prestressed Concrete Bridge Members (SCC and Testing),

Project Number BR-0204(508), Tallapoosa County.

Alabama Standard Specifications, 2008 Edition, shall be amended by replacing Section 513 and adding a new Section 516 as follows:

## SECTION 513
## PRESTRESSED CONCRETE BRIDGE MEMBERS

**513.01 Description.**

This Section shall cover the furnishing and installation of precast prestressed concrete bridge members. The required details of the members and the required details of the installation of the members in the structure will be shown on the plans.

Testing for research purposes will be performed during the production and after the installation of the girders. The Contractor shall provide the assistance described in Section 516 and the assistance required by the Engineer in performing the testing. The

Contractor shall perform the work in a manner that will accommodate the delays in the girder production and installation process due to the testing.

**513.02 Materials.**

(a) ALDOT PROCEDURE.

Material requirements for the production of precast prestressed concrete bridge members are given in this Section and also in ALDOT-367 "Production and Inspection of Precast Non-Prestressed and Prestressed Concrete". Modifications to these material requirements for the production of precast prestressed concrete bridge members utilizing self-consolidating concrete (SCC) are given in Section 516.

(b) REINFORCING STEEL AND PRESTRESSING STEEL.

Reinforcing steel and prestressing steel shall meet the requirements given in SECTION 835. Reinforcing steel shall be Grade 60 {Grade 420}. Prestressing steel strands and bars shall be the type shown on the plans.

(c) CONCRETE MIX DESIGN FOR CONVENTIONAL CONCRETE.

The concrete producer shall establish the proportion of materials for each class and type of concrete following the guidelines given in ALDOT-170, "Method of Controlling Concrete Operations for Structural Portland Cement Concrete" and ALDOT-367, with the exception that the following criteria shall be used instead of the "Master Proportion Table".

The 28-day compressive strength for prestressed concrete bridge members shall be 5000 psi {35 MPa} if the required compressive strength is not shown on the plans.

The concrete producer shall submit the proposed concrete mix design to the Materials and Tests Engineer for approval following the requirements given in ALDOT-170. The distribution of the approved concrete mix design will be in accordance with the requirements given in ALDOT-170 and ALDOT-367. Any changes of the materials and/or proportions of the mix design will require a concrete mix re-submittal.

The mix design shall be based on the following requirements:

| MIX DESIGN CRITERIA | ALL MEMBERS EXCEPT PILES | CONCRETE PILES |
|---|---|---|
| Minimum Cementitious Materials Factor (Lbs/Yd$^3$) {kg/m$^3$} | 550 {330} | 600 {356} |
| Maximum Water/Cementitious Materials Ratio | 0.45 | 0.45 |
| Maximum Slump (prior to admixture) (in) {mm} | 4.0 {100} | 4.0 {100} |

The maximum total air content is 6.0 % by volume. The mix design shall be based on a target total air content of 4.5 %.

Chemical admixtures may be used to increase the slump of the concrete to a maximum of 9 inches {225 mm} if this is proposed in the mix design submittal and approved for inclusion in the mix. The approved water to total cementitious materials ratio shall not be exceeded in order to increase the slump.

Cement for piles shall be Type II and shall be low tricalcium aluminate. If requested by the Contractor and approved by the Materials and Tests Engineer, Type I or Type III cement containing a maximum of 8 % tricalcium aluminate may be used.

The concrete for piles shall contain Class "F" fly ash and microsilica as components of the cementitious materials. The requirements for fly ash and microsilica are given in Section 806, Mineral Admixtures. Fly ash shall meet the requirements given in AASHTO M 295. Fly ash shall also meet the Supplementary Chemical Requirement given in Table 2 of AASHTO M 295.The percentage of cement, fly ash and microsilica in piles shall be the percent by weight {mass} of the total cementitious materials content shown in the following table:

| CEMENTITIOUS CONTENT IN PILES | |
|---|---|
| Cement (Type II) | 70% |
| Fly Ash (Class “F”) | 20% |
| Microsilica | 10% |
| Total Cementitious Content | 100% |

(d) CONCRETE MIX DESIGN FOR SELF CONSOLIDATING CONCRETE (SCC).

Concrete mix design requirements for the production of precast prestressed concrete bridge members utilizing self-consolidating concrete (SCC) are given in Section 516.

**513.03 Construction Requirements.**

(a) MANUFACTURER'S PLANT, LABORATORY AND PERSONNEL REQUIREMENTS.

The concrete bridge member manufacturing plant shall be certified by the Precast/Prestressed Concrete Institute (PCI) Plant Certification Program. Certification of the production plants shall be at least Category B4 (Prestressed Deflected Strand Bridge Members). The manufacturer shall submit proof of the plant certification to the Materials and Tests Engineer prior to the start of production.

The manufacturing plant shall have on site, at the time of manufacturing bridge components for ALDOT, at least one technician that is certified as an ALDOT Concrete

Technician. This technician shall also be certified as PCI Level I/II. The manufacturer shall submit proof of this certification to the Materials and Tests Engineer prior to the start of production and during production when required by the Engineer.

The manufacturer's laboratory and laboratory personnel shall be qualified in accordance with the requirements given in ALDOT-405, "Certification and Qualification Program for Concrete Technicians and Concrete Laboratories".

(b) SHOP DRAWINGS AND NOTIFICATION OF MANUFACTURER.

The Contractor shall submit shop drawings to the Bridge Engineer for approval prior to production. The complete details of prestressed concrete members shall be submitted as Shop Drawings in accordance with the requirements given in Section 105.02. The submittal shall include the proposed tensioning and de-tensioning procedures.

Within 30 days after the award of the contract, the Contractor shall notify the State Materials and Tests Engineer in writing of the manufacturer's proposed fabrication schedule.

(c) CAMBER OF GIRDERS AND BRIDGE DECK SPANS.

The theoretical camber of girders will be shown on the plans. The camber of girders shall be a minimum of 1/2 inch {13 mm} at the time of shipment. The camber of span sections shall be a minimum of 1/4 inch {6 mm} at the time of shipment.

(d) SURFACE FINISH.

All surfaces shall have a Class 1 surface finish in accordance with the requirements given in Section 501.

The outside of all exterior girders shall have a Class 2 surface finish in accordance with the requirements given in Section 501 if a Class 3 is not shown to be required on the plans. A Class 2 surface finish shall be the same finish that is applied to other portions of the bridge structure. The final Class 2 finish shall not be applied until after the completion of the construction of the bridge deck on the girders.

A Class 2 surface shall be applied to the final exposed surface of concrete piles if shown to be required on the plans.

The riding surface of bridge deck span sections shall be finished with either a wood float finish or with a broom finish done with a broom with medium to stiff bristles. The surface shall not vary more than 1/8 of an inch {3 mm} from a 10 foot {3 m} straight edge.

The bonding surface of bridge deck span sections (surface covered by an overlay) shall be raked in a transverse direction to provide a roughened surface for the application of the overlay. The roughened surface shall have a minimum of 1/4 inch {6 mm} ridges raised in the surface at the time of the initial set of the concrete.

Where self-consolidating concrete is used for casting the girders the Contractor shall submit the proposed method of providing a roughened or mechanical bonding surface on the tops of the girders. The proposed method shall be shown on the girder shop drawings submitted for approval.

(e) HANDLING, STORING, AND TRANSPORTING MEMBERS.

The Contractor shall be fully responsible for handling, storing and transporting prestressed concrete bridge members in a manner that will prevent damage to the members.

Girders shall be handled and stored in an upright position. Lifting hooks or similar devices for lifting shall be placed at points close to each end of each member or at the locations shown on the plans. Devices shall be of sufficient strength and embedment to provide safe handling of the members. Blocking under units during storage and handling shall be placed to prevent damage.

Piles shall be lifted, stored, transported, and placed in the pile driving leads in a manner that will eliminate the possibility of damaging bending stresses, cracking and spalling. Piles shall be lifted by means of a suitable bridle or sling attached to the pile at pickup points designated on the plans. Cracked piles will be rejected and shall be immediately removed and replaced without additional compensation.

All prestressed concrete bridge members except piles shall be held at the plant for a minimum of 4 days after casting. Piles shall be held at the plant for a minimum of 21 days after casting. All prestressed concrete bridge members shall not be transported until the minimum 28 day compressive strength is obtained and verified by test cylinders.

(f) INSTALLATION OF PRESTRESSED CONCRETE MEMBERS.

1. DAMAGED MEMBERS.

Members that are damaged in any way shall be replaced or repaired without extra compensation.

2. PRESTRESSED CONCRETE GIRDERS.

Prestressed girders shall be lifted by attachment at the lifting points shown on the shop drawings. Girders shall be supported at the bearing points shown on the plans when they are put into the structure.

The Contractor shall be fully responsible for the stability of the girders during construction. The Contractor shall submit working drawings in accordance with the requirements given in Article 105.02 for temporary bracing installed to provide stability for the girders.

3. INSTALLATION OF DECK SPAN MEMBERS.

Deck span members that will not be covered by an overlay shall be installed so that the difference in the top surface of adjacent members does not exceed 1/4 of an inch {6 mm}. Deck members shall be replaced without extra compensation if the difference in the surface is not within the allowable 1/4 of an inch {6 mm} difference. Members not meeting the installation tolerance may be installed in other locations in the structure if this results in an acceptable deck surface.

Deck span members shall be bolted together as shown on the plans to provide snug tight fit. Beveled washers shall be provided if the flat washers, bolt heads and nuts are not in full bearing on each other after tightening. Snug tight is defined as the tightness that can be produced by one or two solid blows from an impact wrench or by full effort of a person using an ordinary 2 foot {610 mm} spud wrench. The threads of the bolts shall be burred to prevent removal after the members have been acceptably bolted together.

At the completion of the bolting together of the members, the concrete keyway shall be filled with an approved 4000 psi {28 MPa} compressive strength concrete mix. The Contractor shall obtain the approval of the mix design from the Materials and Tests Engineer prior to filling the keyways. The keyways shall be filled in accordance with the following:

- standard mixing of the concrete shall be completed a minimum of 45 minutes in advance of placement;
- the mix shall be retempered by remixing the concrete without additional water just prior to placing;
- the concrete mix shall be placed in the keyway, tamped, and packed as necessary to insure complete filling of the joint;
- the exposed surface of the joint shall be struck to the same elevation of the adjoining deck sections;
- the surface shall be given a wood float finish.

(g) PLACEMENT OF CRANES ON BRIDGE DECKS.

Cranes shall not be placed on a bridge deck unless approved by the Engineer. The Contractor shall submit a placement plan for review prior to placing a crane on a bridge deck. The placement plan shall be submitted in accordance with the requirements given in Section 510.

**513.04 Method of Measurement.**

(a) ITEM NO. 513-A.

Girders will be measured per each girder of each type and length.

(b) ITEM NO. 513-B.

Each type of girder will be measured per linear foot of casting length shown on the approved shop drawings minus the length of elastic shortening and shrinkage. This will be the length recorded on the Shipping Notice (BMT-139) prepared by the

Department's Plant Inspector. A copy of BMT-139 shall be sent with the shipment of the girders.

(c) ITEMS 513-C and 513-D.

Concrete span sections will be measured per each type and size.

**513.05 Basis of Payment.**

(a) UNIT PRICE COVERAGE.

1. ITEMS 513-A and 513-B.

Concrete girders will be paid for at the contract unit price for each type of girder. This price shall be full compensation for furnishing all materials, accessories, tools and labor necessary to manufacture and install the girders except for the additional costs that results from the testing of the girders for research purposes and the utilization of SCC. Compensation for the cost of the testing of the girders and utilizing SCC as described in Section 516 will be paid for under the requirements given in that Section.

This price shall also be full compensation for premolded bituminous filler, for all items cast into the concrete including metal bearing plates and studs welded to these plates, and for obtaining a Class 2 surface finish on the outside of all exterior girders.

2. ITEMS 513-C and 513-D.

Concrete deck span sections will be paid for at the contract unit price for each type and size. This price shall be full compensation for furnishing all materials, accessories, tools and labor necessary to manufacture and install the span sections.

This price shall also be full compensation for all items cast into the concrete, for the tie bolts, for expansion and bearing materials, for cover concrete over fittings, for grout and grouting, for placement of keyways, and for surface finishing. Other structural steel items and handrail will be covered under other items of work.

3. PARTIAL PAYMENT.

Partial payments will be made in accordance with the following schedule:

- Fabrication and Delivery to Approved Storage Site. (Approved storage sites and partial payment for stored materials are addressed in Article 109.07);
- Erected and the Required Finish Applied to Girder Units or the Bolting Up and Casting Of Keyway on Deck Units - 100%.

Partial payments for members that are unacceptable because of damage, improper installation or any other reason will be recovered by the Department on the next monthly estimate or final estimate, whichever is applicable.

(b) PAYMENT WILL BE MADE UNDER ITEM NO.:

513-A _*_ Pretensioned-prestressed Concrete Girders, Type _**_ (SPECIALTY ITEM) - per each

513-B Pretensioned-prestressed Concrete Girders, Type _**_(SPECIALTY ITEM) - per linear foot {meter}

513-C Prestressed Concrete Interior Span Sections, ____ Wide by ____ Deep by ____ Long (SPECIALTY ITEM) - per each

513-D Prestressed Concrete Exterior Span Sections, ____ Wide by ____ Deep by ____ Long (SPECIALTY ITEM) - per each

* Length

** Type I, II, III, etc., as per AASHTO Classification.

## SECTION 516
## TESTING CONCRETE GIRDERS AND UTILIZING SELF-CONSOLIDATING CONCRETE

**516.01 Description.**

This Section shall cover the testing of prestressed concrete girders for research purposes and the utilization of self consolidating concrete (SCC) for the production the girders where shown to be required on the plans. The requirements given in this Section shall modify and supplement the material, production and compensation requirements given in Section 513.

SCC is a highly flowable, non-segregating concrete utilized in the fabrication of prestressed bridge components. SCC is utilized to fill the formwork and encapsulate the reinforcing steel without, or with very minimal, applied vibratory consolidation. The Engineer may require a minimal amount of applied vibratory consolidation.

Testing for research purposes will be performed during the production and after the installation of the girders. The Contractor shall provide the assistance described in this Section and the assistance required by the Engineer in performing the testing. The Contractor shall perform the work in a manner that will accommodate the delays in the girder production and installation process due to the testing.

**516.02 Materials.**

(a) ALDOT PROCEDURES.

Material requirements for the production of precast prestressed concrete bridge members are given in this Section and in the following ALDOT Procedures:

- ALDOT 170 "Method of Controlling Concrete Operations for Structural Portland Cement Concrete";
- ALDOT-367 "Production and Inspection of Precast Non-Prestressed and Prestressed Concrete";
- ALDOT 441 "Prestressing Strand Pullout Test".

The requirements given in this Section shall govern over similar requirements given in the ALDOT Procedures.

(b) DESIGN, SUBMITTAL AND APPROVAL OF SCC MIX DESIGN.

The concrete producer shall establish the proportion of materials for the SCC mix in accordance with the guidelines given in ALDOT 170 with the exception that the mix design criteria given in this Section shall be followed instead of the Master Proportion Table.

The concrete producer shall submit the proposed SCC mix design to the Materials and Tests Engineer for approval following the guidelines given in ALDOT-170. The distribution of the approved concrete mix design will be in accordance with the requirements given in ALDOT-170 and ALDOT-367. The producer shall submit all proposed changes to a previously approved mix design for reapproval. Any changes of the materials and/or proportions of the mix design will require a concrete mix re-submittal.

(c) CEMENTITIOUS MATERIALS FOR SCC.

The cementitious materials for SCC shall meet the requirements given in Sections 806 and 815. The following combinations of mineral admixtures may be substituted for a portion of the Portland cement that is required for the total cementitious materials content of the mix.

| MAXIMUM PERCENT OF ALLOWABLE MINERAL ADMIXTURE SUBSTITUTION FOR PORTLAND CEMENT (BY WEIGHT) | | | |
|---|---|---|---|
| Substitution Option | Class C or Class F Fly Ash | Ground Granulated Blast Furnace Slag | Microsilica |
| 1 | 30 % | - | - |
| 2 | - | 50 % | - |
| 3 | - | - | 10 % |
| 4 | 20 % | - | 10 % |
| 5 | 20 % | 30 % | - |

(d) CHEMICAL ADMIXTURES FOR SCC.

Chemical Admixtures for the production of SCC shall be selected from List II-1 of the Department’s "Materials, Sources, and Devices with Special Acceptance Requirements" Manual. Refer to Subarticle 106.01(f) and ALDOT-355 concerning this list.

Approved viscosity modifying admixtures (VMA) may be used as a part of the chemical admixtures if they are shown in the approved mix design.

(e) PREPARATION OF SCC TEST SAMPLES.

All SCC test specimen molds, air content buckets, and unit weight buckets shall be filled in one continuously poured lift using a suitable container without vibration, rodding, or tapping. The SCC shall be dropped from a height of 6 inches ± 2.0 inches above the mold or container top into the center of the container until the concrete is slightly above the top of the mold or bucket. The SCC shall be struck off level with the top of the mold or bucket.

(f) REQUIREMENTS FOR THE DESIGN OF THE SCC MIX.

The design of the SCC shall be in compliance with the requirements given in the following table.

| **REQUIRED DESIGN PROPERTIES OF THE SCC MIX<br>Properties shall be measured from a minimum size batch of 3 cubic yards.** | | |
|---|---|---|
| **Property** | **Requirement** | **Test** |
| Compressive Strength at 28 days | The compressive strength shown on the plans or 5000 psi if it is not shown. | AASHTO T 22 |
| Cementitious Materials Content | Minimum 600 pounds per cubic yard of concrete. | |
| Water/Cementitious Materials Ratio | Maximum 0.40. | |
| Nominal Aggregate Size | Maximum 0.75 inches. | |
| Fine/Total Aggregate Ratio | 0.45 to 0.55 by volume. | |
| Total Air Content | Maximum 6.0 % by volume. The design of the mix shall be based on a target total air content of 4.5 %. | AASHTO T 152 |
| Temperature | Freshly mixed concrete at the time of placing in the forms shall not be less than 50 °F or more than 95 °F. | AASHTO T 309 |
| 28-Day Drying Shrinkage | Maximum 0.04 % when prisms are exposed to drying at a concrete age of 7 days. | ASTM C 157 |

| | | |
|---|---|---|
| Slump Flow[1] | Minimum 25 inches, maximum 29 inches. Test shall be c*ompleted* within 10 minutes after completion of mixing. | ASTM C 1611 |
| Passing Ability[2] | Difference between slump flow and J-Ring flow 3.0 inches or less. Test shall be *completed* within 10 minutes after completion of mixing. | ASTM C 1621 |
| Visual Stability Index[3] (VSI) | Less than 2.0. Test shall be *completed* within 10 minutes after completion of mixing. | ASTM C 1611 |
| Static Segregation Index | Maximum 15.0 %. Test shall be *started* within 10 minutes after completion of mixing. | ASTM C 1610 |
| Robustness | Slump flow 29 inches or less.<br>VSI less than 2.0. | ASTM C 1611 |
| 1. The Slump Flow test shall be performed using the "Filling Procedure B" in ASTM C 1611.<br>2. The Passing Ability test shall be performed using the “Filling Procedure B” in ASTM C 1621.<br>3. A VSI of 1.5 is acceptable. A VSI of 1.5 corresponds to a stable SCC with a minimal mortar halo (< 0.25 inch), good aggregate distribution, and slight noticeable bleeding at the surface of the slump patty. | | |

(g) ROBUSTNESS TESTING.

The freshly mixed SCC shall be robust to ensure that segregation of the mixture does not occur during or after placement.

Robustness testing shall be performed on a minimum batch size of 3 cubic yards of mix. No water may be withheld from the minimum batch size of 3 cubic yards. A representative sample shall be taken and the unit weight shall be determined in accordance with the requirements given in AASHTO T 121. After completion of the unit weight test, a 2 cubic foot sample shall be taken for further testing. The concrete weight of the 2 cubic foot sample shall be calculated from the unit weight test results. The 2 cubic foot sample shall be added to a buttered rotating-drum mixer and additional water shall be added to the mixture. The additional water shall be equal to 2 % of the total fine aggregate saturated-surface dry weight in the 2 cubic foot sample. The concrete sample with the added water shall be mixed for 1 minute and then the tests for mixture robustness (Slump Flow and Visual Stability Index) shall be performed. Both the Slump Flow and VSI tests shall be *completed* within 5 minutes after completion of mixing with additional water. A 4 inch x 6 inch digital color photograph (printed with a minimum resolution of 300 dpi) of the slump flow patty obtained for the robustness test shall be submitted with the request for the mixture approval.

The mixture is acceptable if its robustness test results are a slump flow of 29 inches or less and a VSI less than 2.0.

(h) TESTING REQUIREMENTS FOR THE SCC DURING GIRDER PRODUCTION.

SCC shall be in compliance with the requirements given in the following table during the production of the girders. All of the tests shown in this table shall be considered as "one set" of tests. At least one set of tests shall be performed for every 50 cubic yards, or fraction thereof, of concrete placed.

| REQUIRED PROPERTIES OF THE SCC MIX DURING GIRDER PRODUCTION | | |
|---|---|---|
| **Property** | **Requirement** | **Test** |
| Compressive Strength at 28 days | The compressive strength shown on the plans or 5000 psi if it is not shown. | AASHTO T 22 |
| Temperature | Freshly mixed concrete at the time of placing in the forms shall not be less than 50 °F or more than 95 °F. | AASHTO T 309 |
| Total Air Content | Maximum 6.0 % by volume. | AASHTO T 152 |
| Slump Flow[1] | Minimum 25 inches, maximum 29 inches. | ASTM C 1611 |
| Visual Stability Index[2] (VSI) | Less than 2.0. | ASTM C 1611 |
| 1. The Slump Flow test shall be performed using the "Filling Procedure B" in ASTM C 1611.<br>2. A VSI of 1.5 is acceptable. A VSI of 1.5 corresponds to a stable SCC with a minimal mortar halo (< 0.25 inch), good aggregate distribution, and slight noticeable bleeding at the surface of the slump patty. | | |

The QC technician of the prestressed concrete producer shall be responsible for the performance of all concrete sampling and testing during girder production. The Department's QA technician will determine and document the point of sampling and testing and the schedule of testing for each line of production.

Sampling and testing shall be performed as close as possible to the casting bed. Fresh concrete will be accepted during concrete placement based on all of the tests shown in the preceding table except for compressive strength. Acceptance of a girder will be based on the 28-day compressive strength test results.

All tested SCC batches shall meet the 28-day compressive strength, temperature, total air content, and slump flow requirements outlined in the preceding table. Action to correct the lack of mixture stability is required immediately following an individual failing VSI test. The batch of SCC immediately following a batch that failed the VSI requirement shall be tested for all of the tests shown in the preceding table. If the VSI requirement is not met on any two consecutive batches of SCC, then the second batch of SCC will be rejected. After the rejection of a batch of SCC that did not meet the VSI requirement, testing of all subsequent batches of SCC shall continue until two consecutive batches of SCC meet the VSI requirement. A 4 inch x 6 inch digital color photograph (printed with a minimum resolution of 300 dpi) of the slump flow patty of the SCC that did not meet the VSI requirement shall be kept for documentation purposes.

(i) AGGREGATE MOISTURE CONTROL.

If moisture meters are not used, the free moisture content of aggregates shall be measured within one hour prior to each day's batching operations, at 2 hour intervals during continuous batching operations, and at any time a change in moisture content becomes apparent.

**516.03 Fabrication and Testing.**

The Contractor shall have the prestressed concrete girders fabricated by Hanson Prestressed Concrete in Pelham Alabama. The Contractor and Hanson shall allow testing for research purposes to be performed during the production of the girders made with conventional concrete and the girders made from SCC.

The Contractor and Hanson Prestressed Concrete shall provide access for the installation of instrumentation and recording of measurements. They shall also alter the schedule of production to allow the installation of instrumentation and recording of measurements.

The Contractor and Hanson Prestressed Concrete shall provide full cooperation as directed by the Engineer to insure that the testing is done in a manner that will allow the researchers to make sufficient measurements.

The following are anticipated delays that will be necessary to facilitate the research-related testing:

-Up to one hour at the casting bed between installation of girder reinforcement and placement of side forms;

- Up to three hours at the casting bed between the removal of side forms and strand detensioning;
- Up to one hour at the casting bed between completion of detensioning and removal of girders;
- One day for measurement of camber after girder erection and prior to placement of deck forms;
- One day for installation of instruments prior to casting of bridge deck;
- Four hours for installation of instruments prior to casting of barrier rails;
- Two days for load testing after completion of bridge construction.

Girders shall be stored in such manner as to facilitate measurements of camber and strain.

The Contractor shall notify the Engineer at least 7 calendar days prior to shipping any girder to the project site. The Engineer will inform the researchers of this delivery schedule to allow them the time to prepare for testing.

Instruments, sensors, or associated cables shall not be disturbed without prior written approval of the Engineer. The Engineer will be advised by the researchers if these devices can be disturbed by the Contractor.

(b) TIME ALLOWED FOR THE PLACEMENT OF THE SCC.

SCC delivery shall be timed so that consecutive lifts will combine completely without creating segregation, visible pour lines, or cold joints. Additional loads of SCC shall be placed within 15 minutes from the previous load. An SCC load outside the 15 minutes margin may be placed on top of a previous load if the ALDOT QA inspector authorizes it and if required, minimal vibration is applied to the previous load.

(c) DISCHARGE OF THE SCC INTO FORMS.

SCC shall be placed from one point only and be allowed to flow sideways. SCC may be also pumped from the bottom upward so as not to encapsulate air. Simultaneous opposing flows of SCC shall not be done.

(d) SEALING FORMS TO PREVENT LEAKAGE.

Sealing of form joints and bulkheads prior to placement of SCC is required. Paste leakage from forms and bulkheads may cause honeycombed areas. Any area where paste leakage has occurred shall be assessed for possible repairs in accordance with the requirements given in Section 2, Article 10 of ALDOT-367.

(e) ALLOWABLE MINIMAL VIBRATION.

Freshly placed SCC shall not be vibrated. If additional loads of SCC are placed over SCC that was placed more than 15 minutes earlier, then minimal vibration may be applied to the surface of the already placed SCC to improve mixing between the different loads of SCC.

**516.04 Method of Measurement.**

The cost of the delays for testing the concrete girders will be measured as a lump sum unit. The utilization of SCC for the fabrication of the girders will be measured as a lump sum unit.

**516.05 Basis of Payment.**

(a) UNIT PRICE COVERAGE.

Concrete girders will be paid for at the contract unit price for each type of girder in accordance with the requirements given in Section 513.

Payment for the cost of the delays for testing of the girders and the utilization of SCC for fabricating the girders shall be full compensation for all costs attributable to the testing and the utilization of SCC that are in excess of the costs of fabricating and installing prestressed concrete girders with conventional concrete. The price for the testing of the girders and utilizing SCC shall be full compensation for furnishing all materials, accessories, tools and labor necessary to manufacture, install, and facilitate the testing of the girders and strands (strand pullout) that are in excess of the costs of girder fabrication and installation utilizing the requirements given in Section 513 without SCC.

(b) PAYMENT WILL BE MADE UNDER ITEM NO.:

516-A Testing Concrete Girders – per lump sum.

516-B Utilization of Self-Consolidating Concrete – per lump sum.

## Appendix C: Moustafa/Logan Pullout Testing Procedure and Results

Alabama Department of Transportation

Auburn University Highway Research Center

Tests Performed September 14, 2010

### Strand Preparation Procedure

1. Ten strand samples were taken from a roll of Strand-Tech Martin ½-inch strand used in the PCI 54 bulb-tee girders constructed for the ALDOT SR 22 Hillabee Creek Bridge/Auburn University SCC research study. Samples were saw-cut to 34 in. lengths, and projections from the saw-cutting were removed. Strand samples were straightened by hand as needed.
2. Ten strand samples were taken from a roll of American Spring Wire ½-inch "special" strand used in PCI 72" bulb-tee girders constructed for the ALDOT SR 22 Hillabee Creek Bridge/Auburn University SCC research study. Samples were saw-cut to 34 in. lengths, and projections from the saw-cutting were removed. Strand samples were straightened by hand as needed.
3. The strand samples were clean and free of rust.

**Block Casting Procedure**

1. Details of the test block geometry and reinforcement are shown in the Appendix.
2. Test block forms were set up, and the reinforcement cage was installed and securely positioned.
3. The strand samples were then tied securely in place in accordance with the layout shown in Appendix.
4. After the strand locations and tying procedure were checked and approved, concrete placement began at 12:05 p.m. on September 13, 2010.
5. Concrete mixture proportions are given in Table 1. The concrete had a slump of 2 ¼ in. and an air content of 3.9 percent.
6. The concrete was vibrated well using an internal vibrator.
7. The top surface was smoothed using a one-pass trowel finish. No strand samples were moved after the vibration ceased.
8. Support racks were placed over the test blocks to keep the curing covers from coming in contact with the tips of the strand samples. Curing compound (Conspec *Aquafilm*) was sprayed on the tops of the blocks to prevent shrinkage cracks from occurring in the top surface.
9. Pullout block and test cylinders were tarp-cured together. Curing was not accelerated.

Table B-1: Concrete mixture proportions

| Materials | Quantity per cubic yard |
|---|---|
| Cement (Type III) | 655 lbs |
| Concrete Sand (Red Bluff) | 1230 lbs (SSD) |
| #78 Crushed Limestone | 2010 lbs (SSD) |
| Normal-Range Water Reducing Admixture | 0 oz |
| Air-Entraining Admixture | 0 oz |
| High-Range Water Reducing Admixture | 0 oz |
| Water | 235 lbs |
| Water/Cement Ratio | 0.43 |

**Pullout Testing Procedure**

1. Pullout testing was performed on September 14, 2010.
2. The average concrete compressive strength measured from tests of four tarp-cured, 6 in. x 12 in. cylinders during the pullout testing period was 4030 psi.
3. The hydraulic jack was a Hercules jack typically used for single-strand stressing in long-line pretensioning operations.
4. The bridging device was slipped over each strand to be tested and placed against the concrete surface. The strand chuck was slipped over the strand to the top of the bridge and light pressure is applied to the jack to seat the jaws of the chuck into the strand.
5. The jacking load was applied in a single increasing application of load at the rate of approximately 20 kips per minute until the maximum load was reached.

**Results**

The following data are reported in Table B-2 for the ½-inch strand and Table B-3 for the ½-inch "special" strand. Each table contains the following information for each strand sample:

- Maximum pullout force capacity ($P_{max}$)
- Approximate load at first noticeable movement ($P_{move}$)
- Approximate distance the strand pulled out at maximum load ($\Delta_{Pmax}$)
- General description of failure

Table B-2: Pullout Test Results for STD ½-inch Strand

| **Strand ID** | **$P_{max}$ (kips)** | **$P_{move}$ (kips)** | **$D_{Pmax}$ (in.)** | **Failure Description** |
|---|---|---|---|---|
| 1 | 36.0 | 15.0 | 2.0 | Pullout |
| 2 | 35.0 | 15.0 | 1.8 | Pullout |
| 3 | 34.0 | 15.0 | 2.3 | Pullout |
| 4 | 40.0 | 15.0 | 2.0 | Pullout |
| 5 | 37.0 | 17.0 | 2.0 | Pullout |
| 6 | 37.0 | 15.0 | 2.0 | Pullout |
| 7 | 33.5 | 13.0 | 2.3 | Pullout |
| 8 | 33.5 | 18.0 | 2.0 | Pullout |
| 9 | 47.5 | 16.0 | 1.5 | Pullout |
| 10 | 41.5 | 16.0 | 1.5 | 7-wire rupture |

The average maximum load for this group of 10 strand samples was 37.5 kips. Values ranged from 33.5 to 47.5 kips with a standard deviation of 4.4 kips (coefficient of variation = 11.8 percent). While these results would typically indicate inadequate results, it was determined that Strand #9 had an unusually strong bond, and thus could be discarded from the analysis. Consequently, the maximum values ranged from 33.5 to 41.5 kips with a standard deviation of 2.8 kips and a coefficient of variation equal to 7.8 percent. Peak load generally corresponded to a pullout-type failure at pullout distances ranging from 1.5 to 2.3 in.

Table B-3: Pullout Test Results for ASW ½-inch "special" Strand

| **Strand ID** | **$P_{max}$ (kips)** | **$P_{move}$ (kips)** | **$D_{Pmax}$ (in.)** | **Failure Description** |
|---|---|---|---|---|
| 1 | 40.5 | NA | 2.0 | Pullout |
| 2 | 40.5 | 25.0 | 2.3 | 6-wire break |
| 3 | 39.0 | 24.0 | 1.5 | 1-wire break |
| 4 | 40.5 | 22.0 | 2.8 | Pullout |
| 5 | 40.5 | 22.0 | 1.8 | Pullout |
| 6 | 37.0 | 23.0 | 2.3 | Pullout |
| 7 | 37.5 | 26.0 | 1.5 | Pullout |
| 8 | 31.5 | 20.0 | 2.0 | Pullout |
| 9 | 31.0 | 19.0 | 1.8 | Pullout |
| 10 | 37.5 | 23.0 | 2.0 | Pullout |

The average maximum load for this group of 10 strand samples was 37.6 kips. Values ranged from 31.0 to 40.5 kips with a standard deviation of 3.6 kips (coefficient of variation = 9.6 percent). Peak load generally corresponded to a pullout-type failure at pullout distances ranging from 1.5 to 2.8 in.

**Discussion**

Based on "excellent transfer/development length performance", Logan (1997) recommends the following conditions for acceptable pullout capacity of *standard* ½-in. 7-wire strand:

- Average $P_{max}$ (six samples) of at least 36 kips
- Coefficient of variation not exceeding 10 percent

The pullout tests on which these recommendations are based were performed in a block with a concrete compressive strength of 4230 psi (Logan 1997).

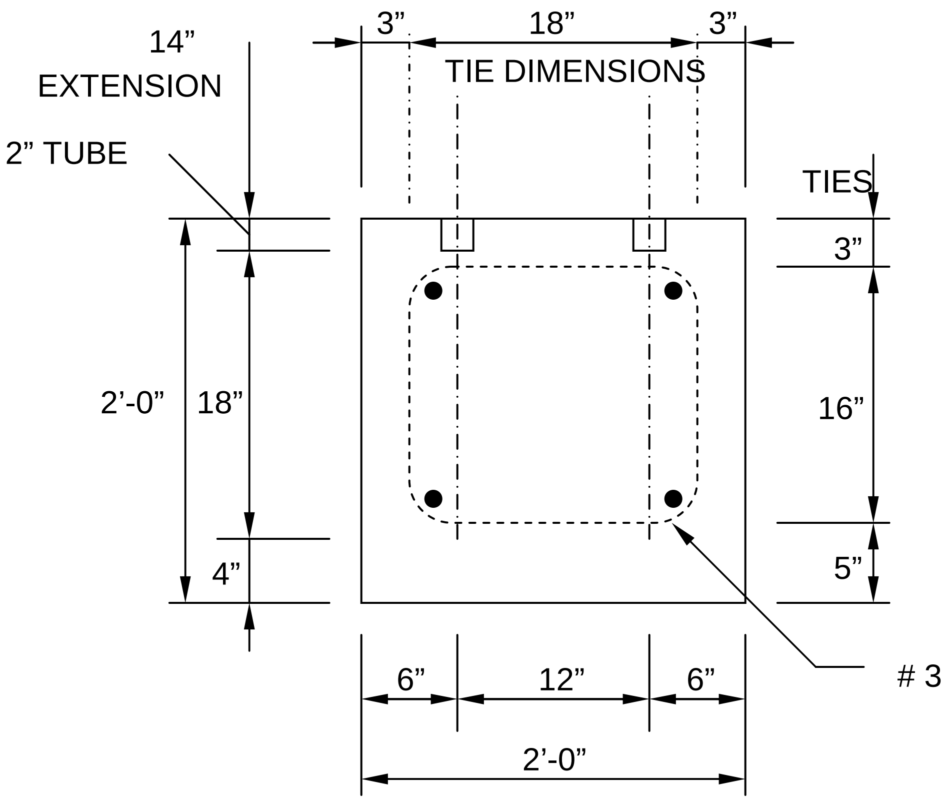
Assuming an equivalent average bond stress at failure for different size strands, the corresponding limiting $P_{max}$ value for ½-in. "special" strand is 37½ kips. The 10-sample average $P_{max}$ value from the tests reported here exceeds their corresponding value, and the coefficient of variation is within the recommended 10 percent limit.

**Reference**

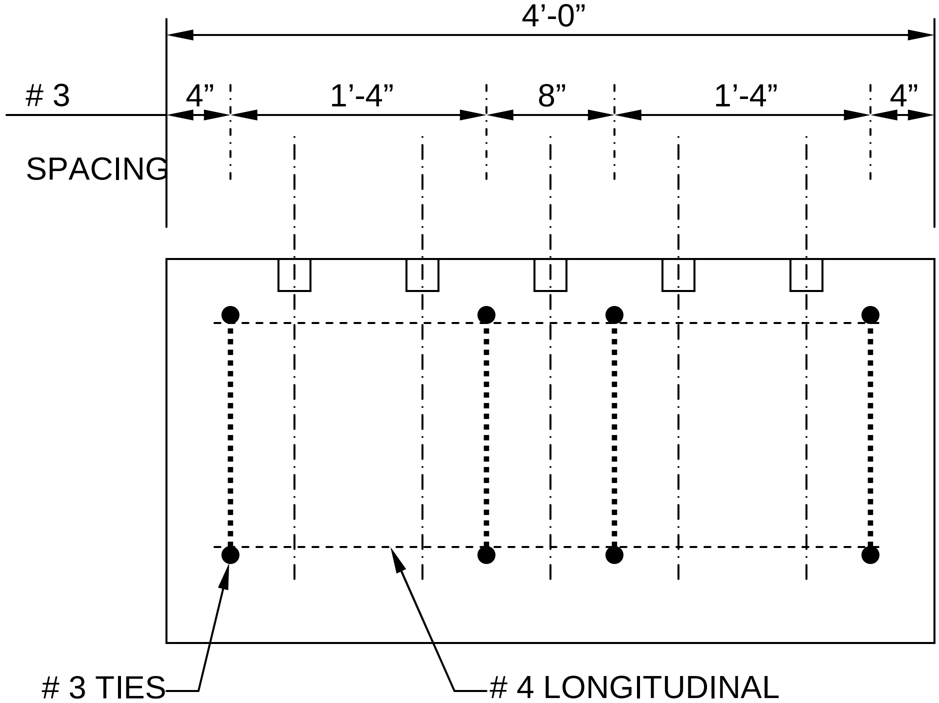
Logan, Donald R. 1997. Acceptance criteria for bond quality of strand for pretensioned prestressed concrete applications. *PCI Journal* 42 (2): 52–90.

APPENDIX

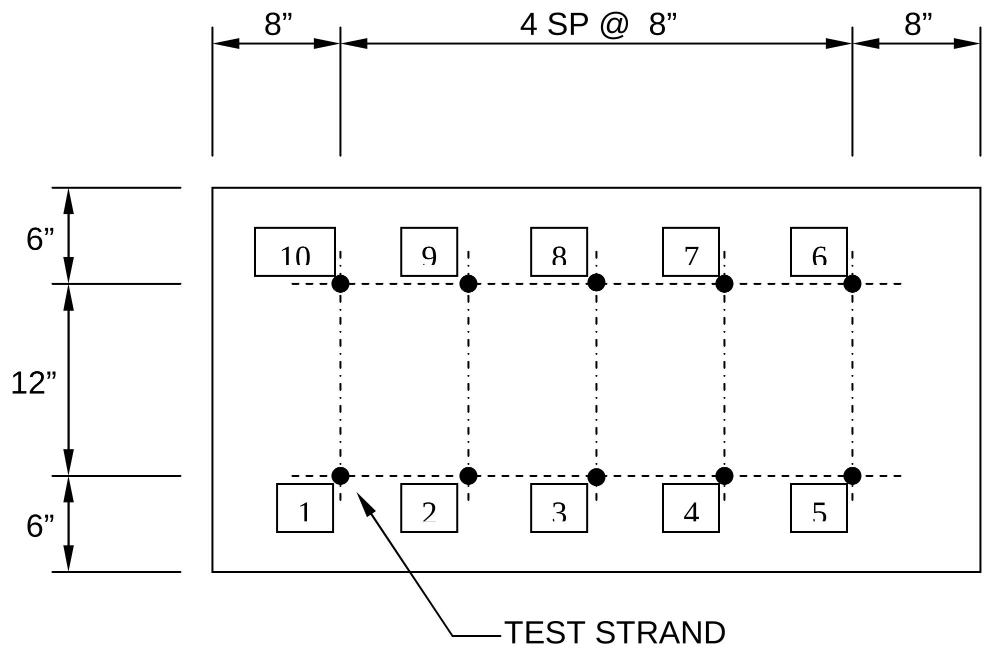
PULLOUT TEST BLOCK DETAILS



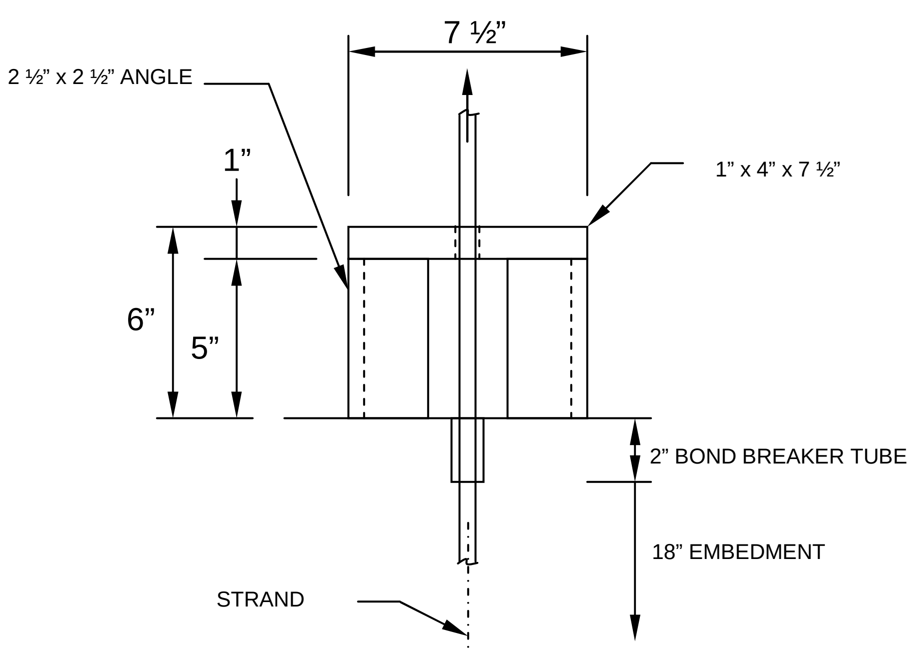

MOUSTAFA PULL-OUT BLOCK ELEVATION




MOUSTAFA PULL-OUT BLOCK ELEVATION




PLAN VIEW WITH STRAND NUMBERS




TEST BRIDGING DEVICE

## Appendix D: Fully Bonded Compressive Strain Profiles

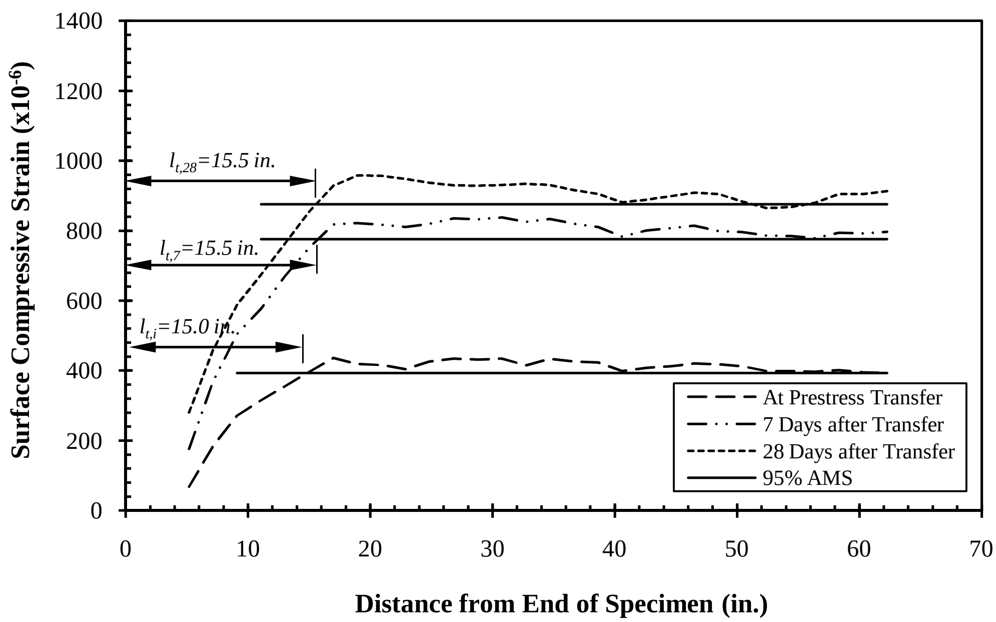



**Figure D-1: 54-2S-A-F Measured Initial and Long-Term Strain Profiles**

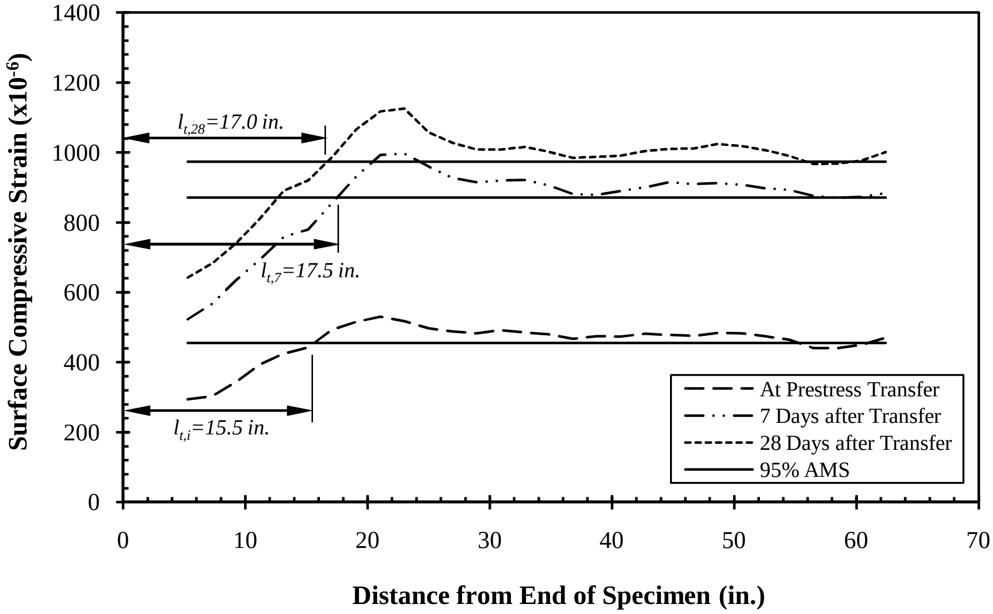



**Figure D-2: 54-2S-B-F Measured Initial and Long-Term Strain Profiles**

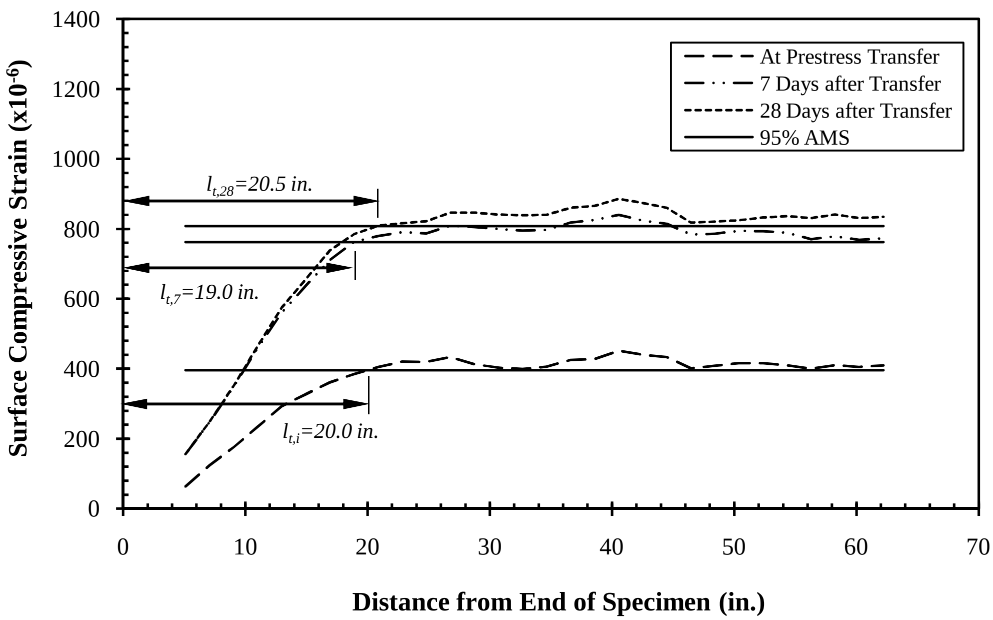



**Figure D-3: 54-4S-A-F Measured Initial and Long-Term Strain Profiles**

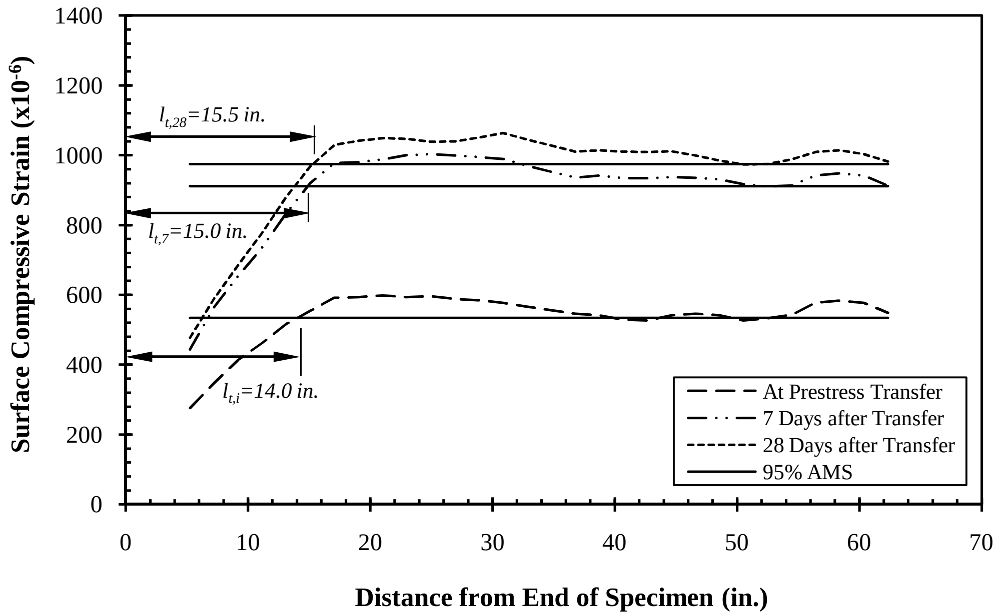



**Figure D-4: 54-4S-B-F Measured Initial and Long-Term Strain Profiles**

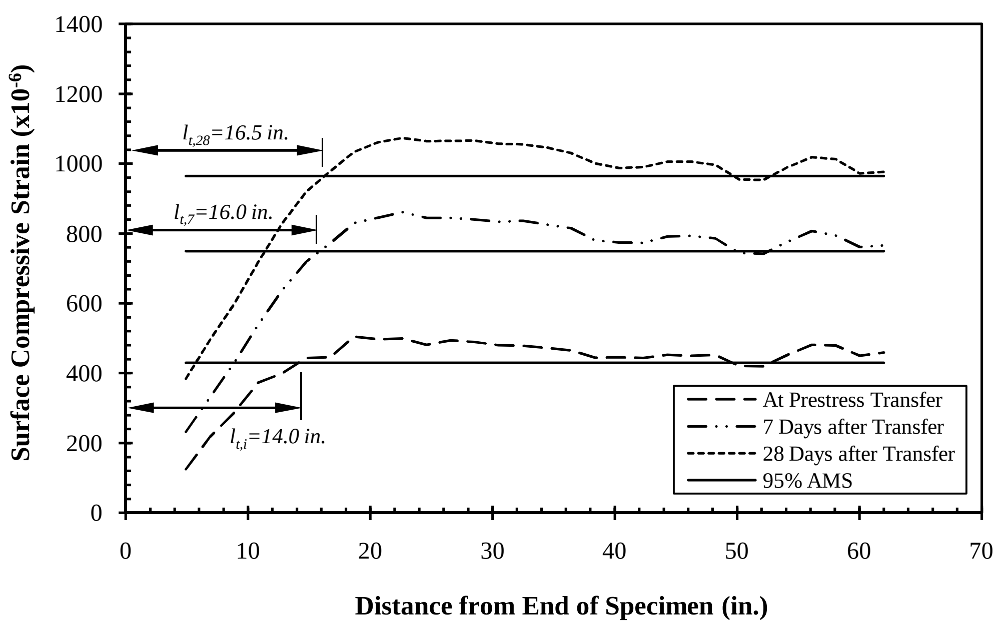



**Figure D-5: 54-7S-A-F Measured Initial and Long-Term Strain Profiles**

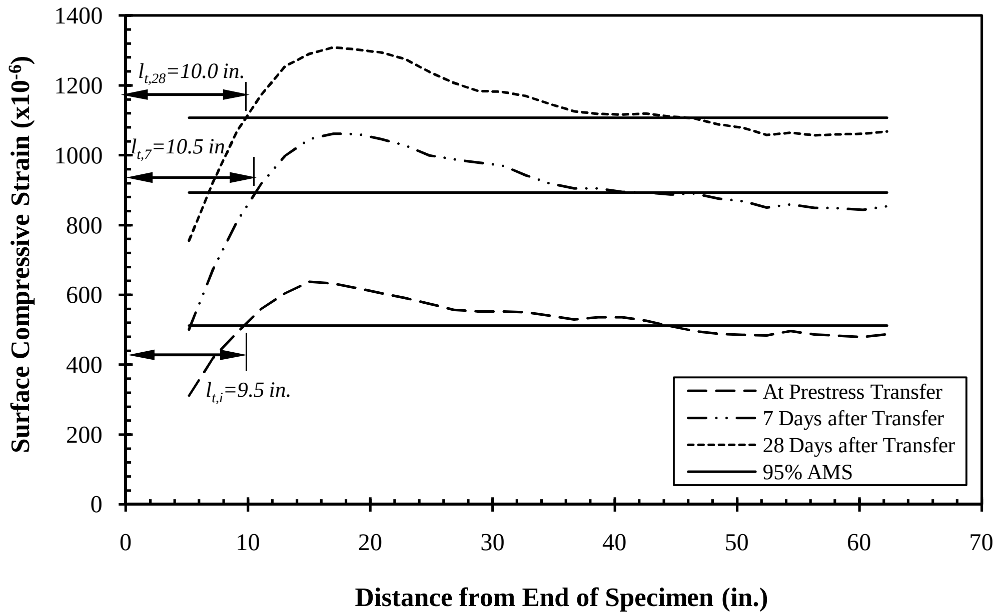



**Figure D-6: 54-7S-B-F Measured Initial and Long-Term Strain Profiles**

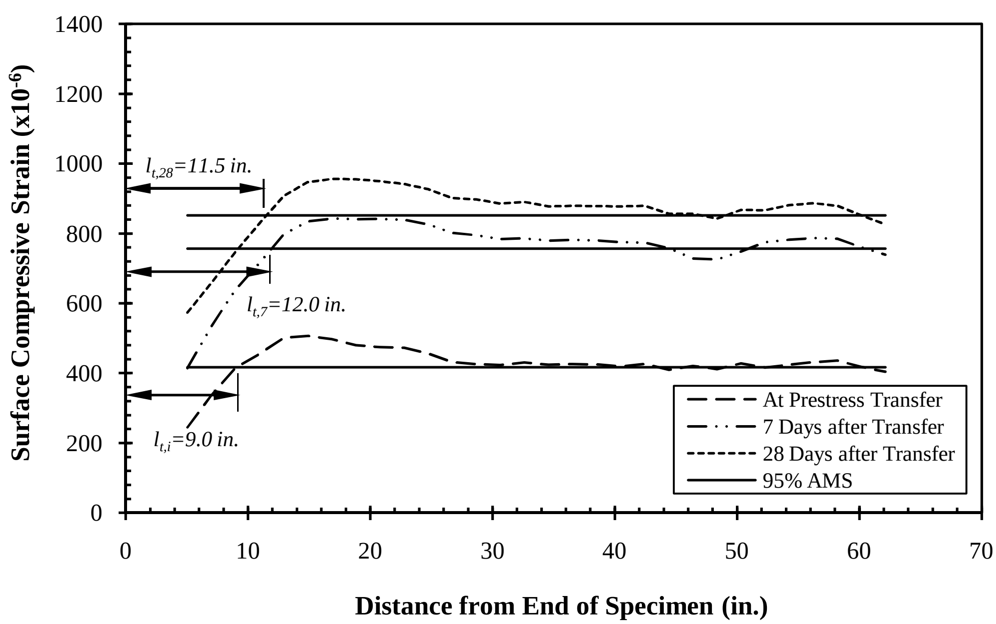



**Figure D-7: 54-8C-A-F Measured Initial and Long-Term Strain Profiles**

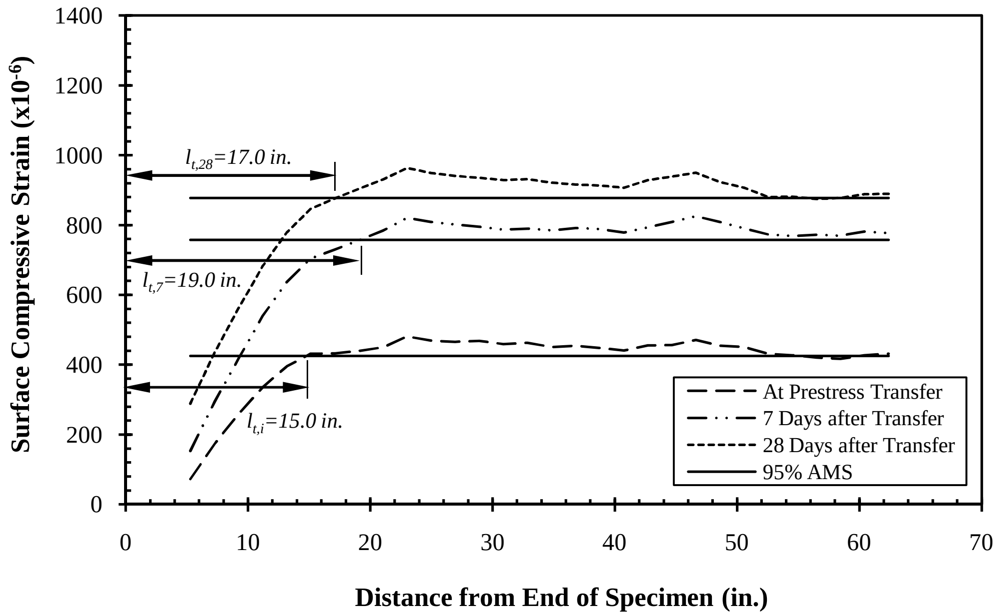



**Figure D-8: 54-8C-B-F Measured Initial and Long-Term Strain Profiles**

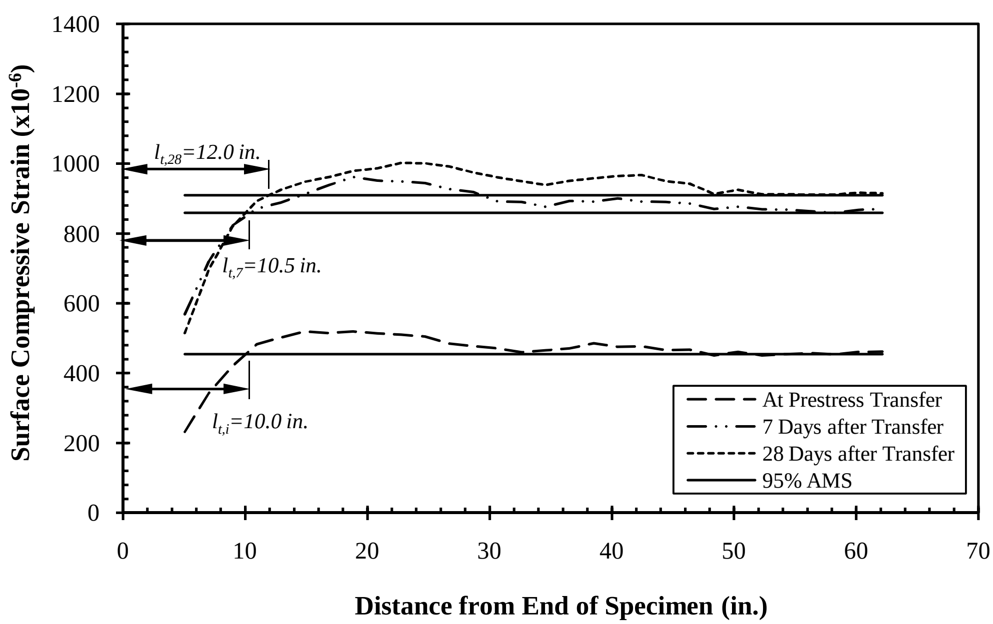



**Figure D-9: 54-11C-A-F Measured Initial and Long-Term Strain Profiles**

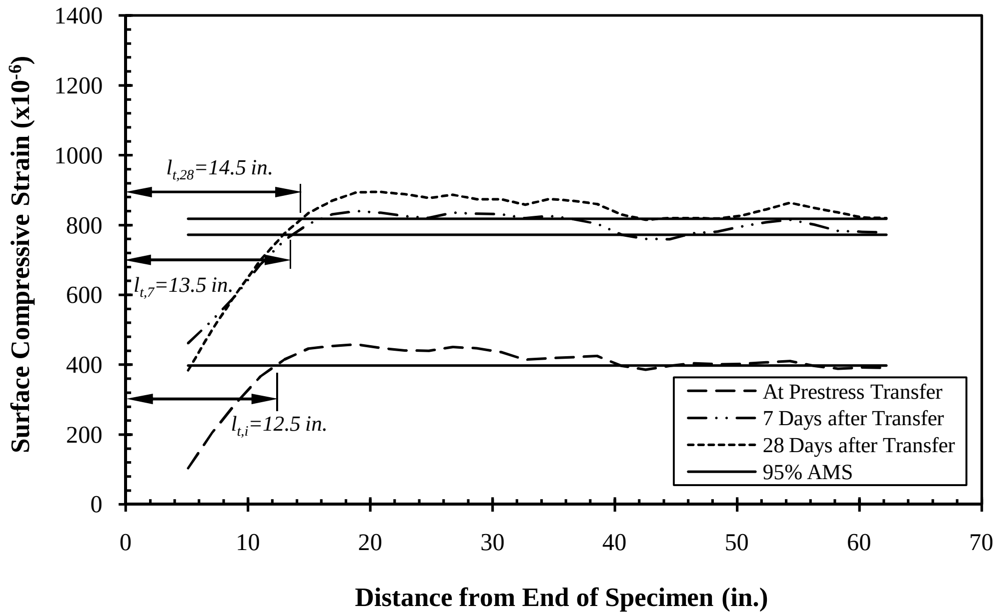



**Figure D-10: 54-11C-B-F Measured Initial and Long-Term Strain Profiles**

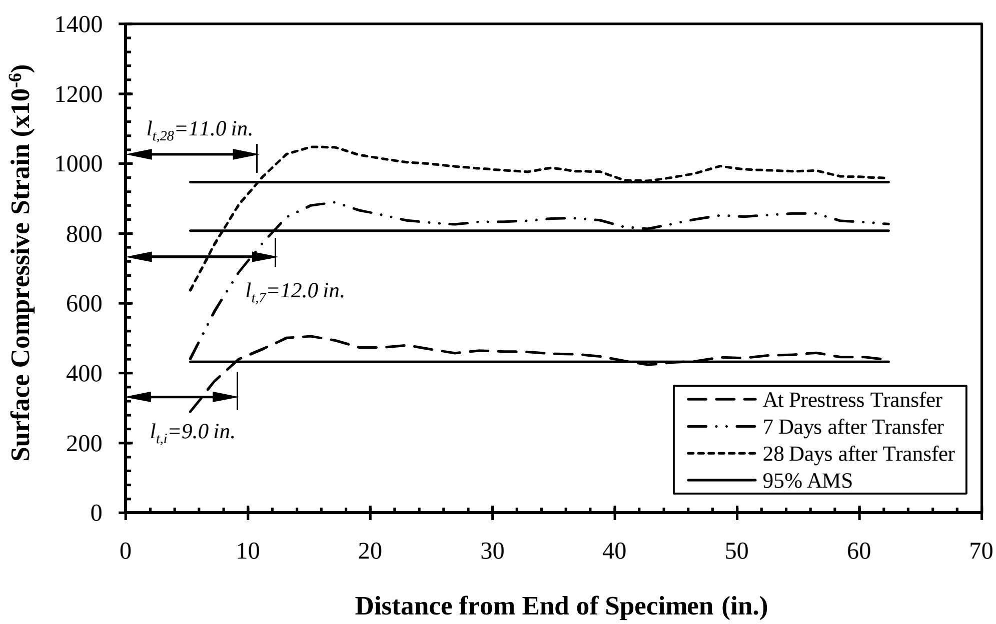



**Figure D-11: 54-13C-A-F Measured Initial and Long-Term Strain Profiles**

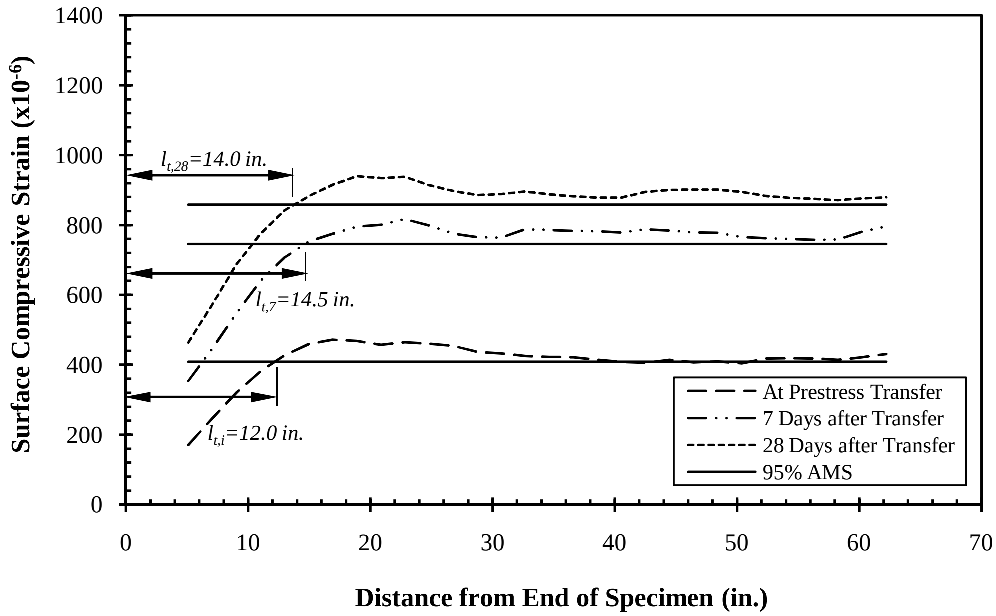



**Figure D-12: 54-13C-B-F Measured Initial and Long-Term Strain Profiles**

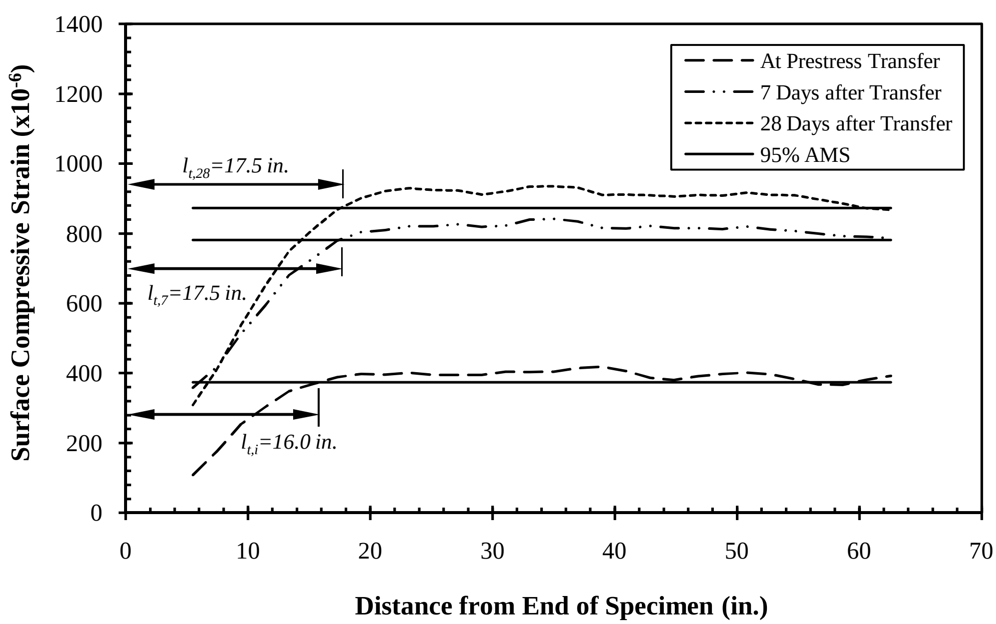



**Figure D-13: 72-2S-A-F Measured Initial and Long-Term Strain Profiles**

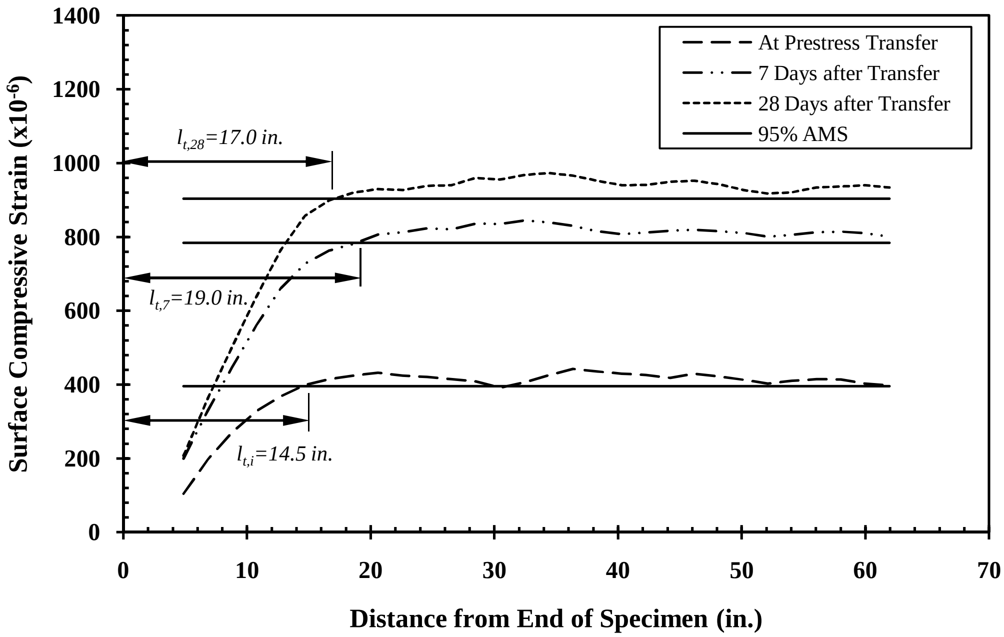



**Figure D-14: 72-2S-B-F Measured Initial and Long-Term Strain Profiles**

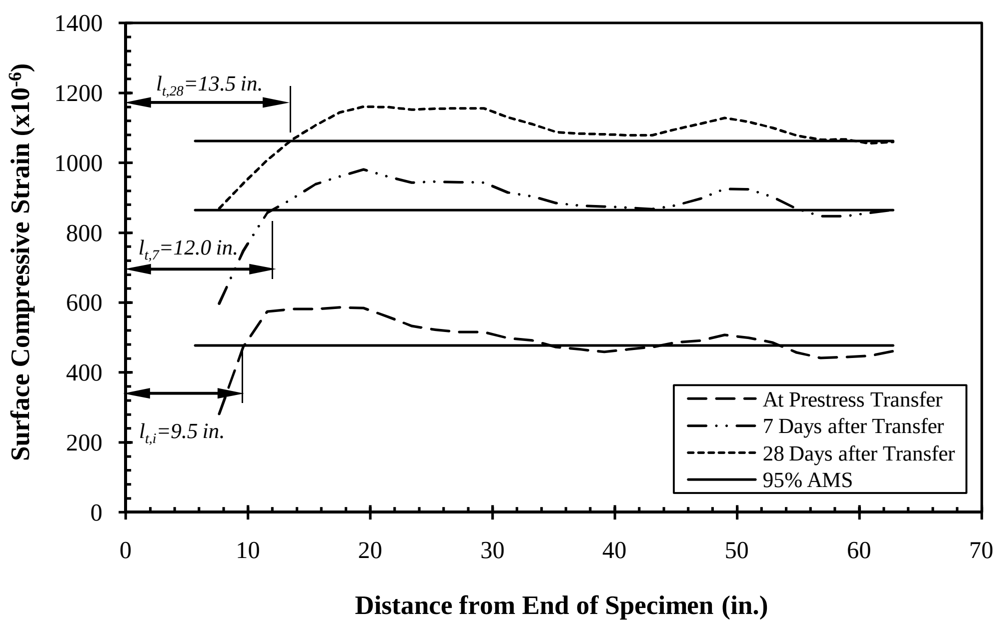



**Figure D-15: 72-4S-A-F Measured Initial and Long-Term Strain Profiles**

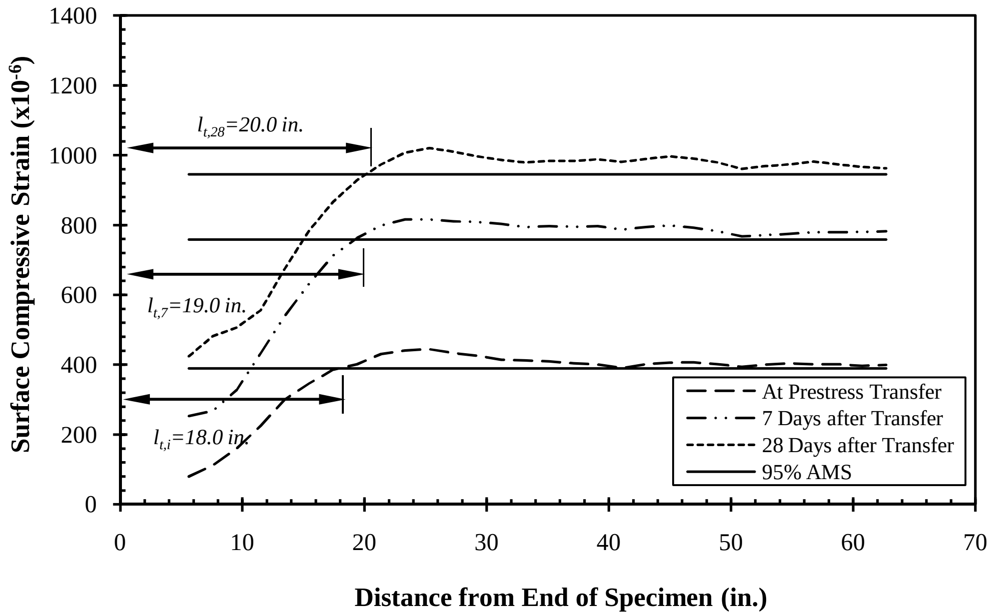



**Figure D-16: 72-4S-B-F Measured Initial and Long-Term Strain Profiles**

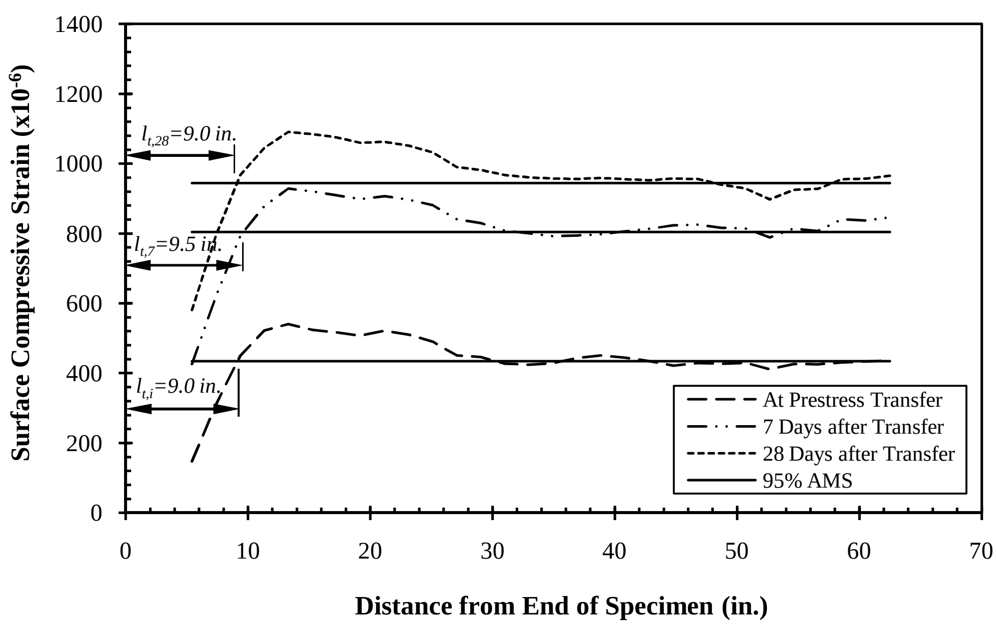



**Figure D-17: 72-7S-A-F Measured Initial and Long-Term Strain Profiles**

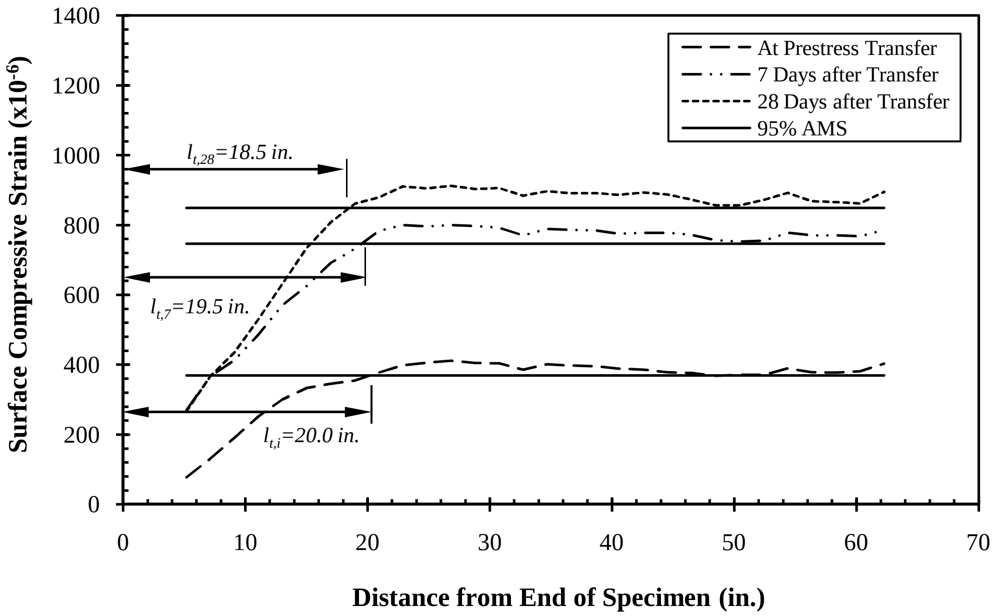



**Figure D-18: 72-7S-B-F Measured Initial and Long-Term Strain Profiles**

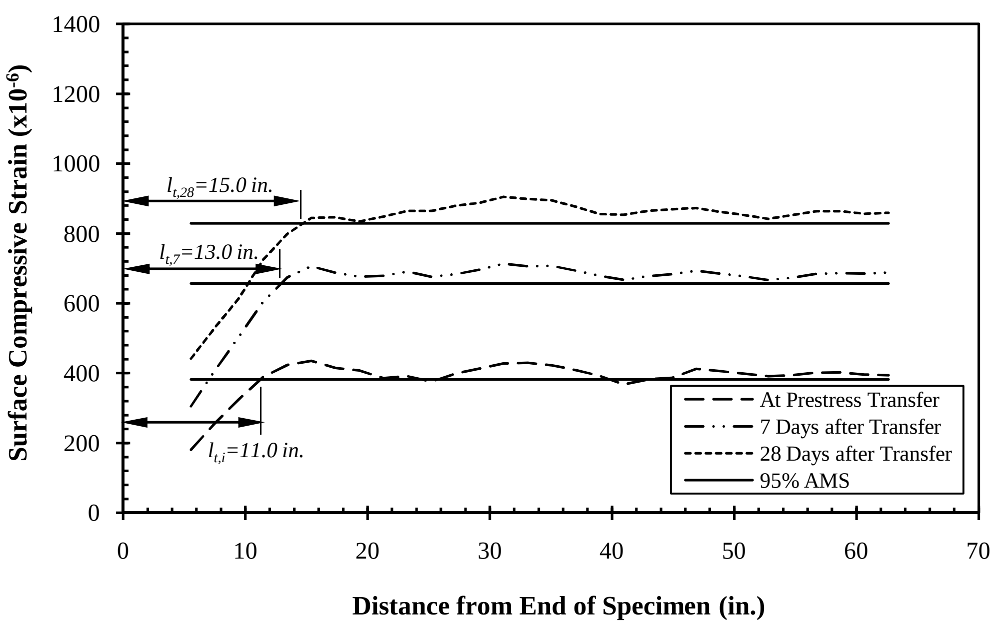



**Figure D-19: 72-8C-A-F Measured Initial and Long-Term Strain Profiles**

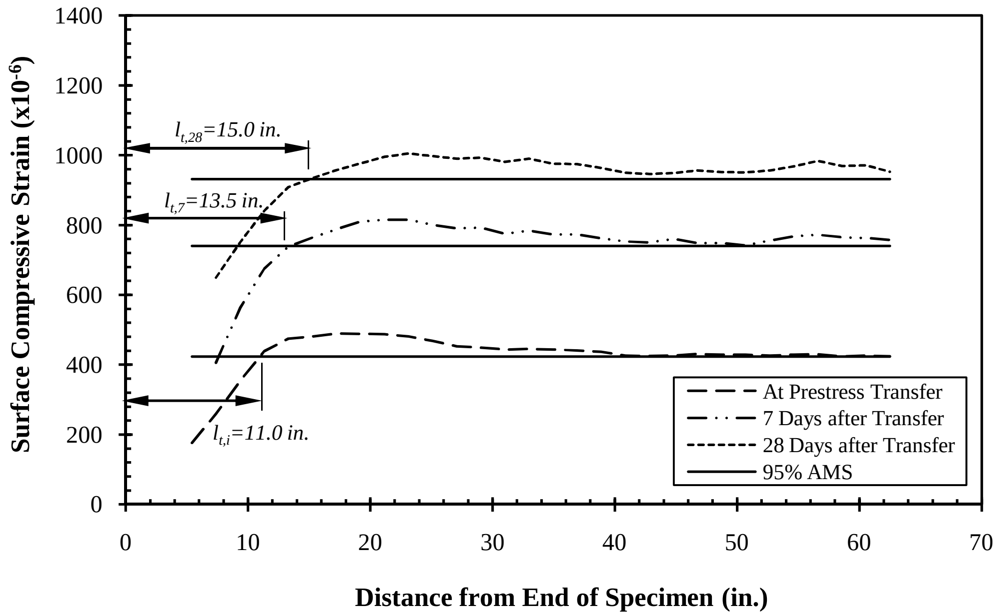



**Figure D-20: 72-8C-B-F Measured Initial and Long-Term Strain Profiles**

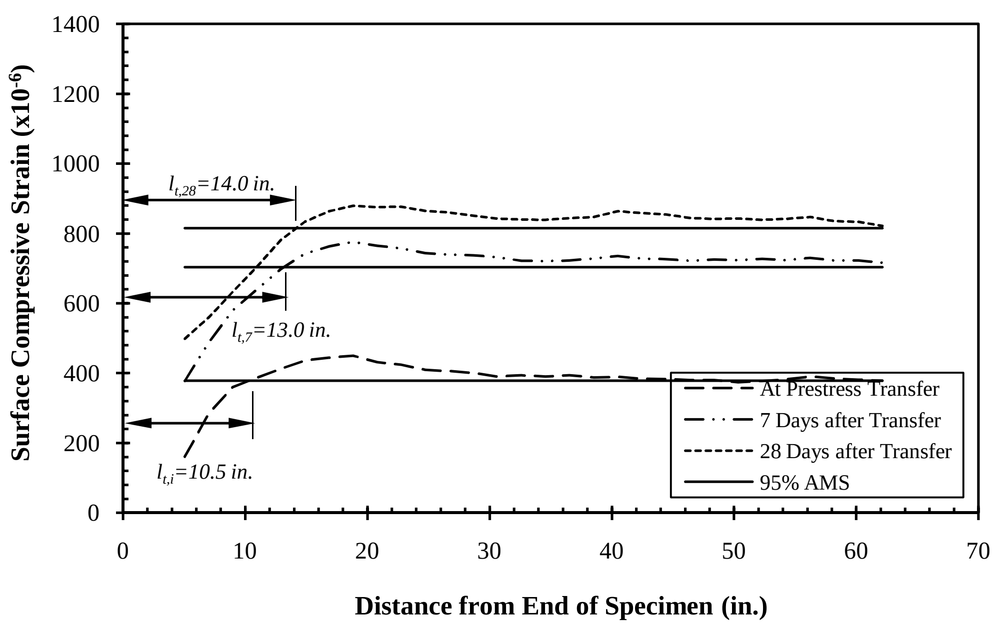



**Figure D-21: 72-11C-A-F Measured Initial and Long-Term Strain Profiles**

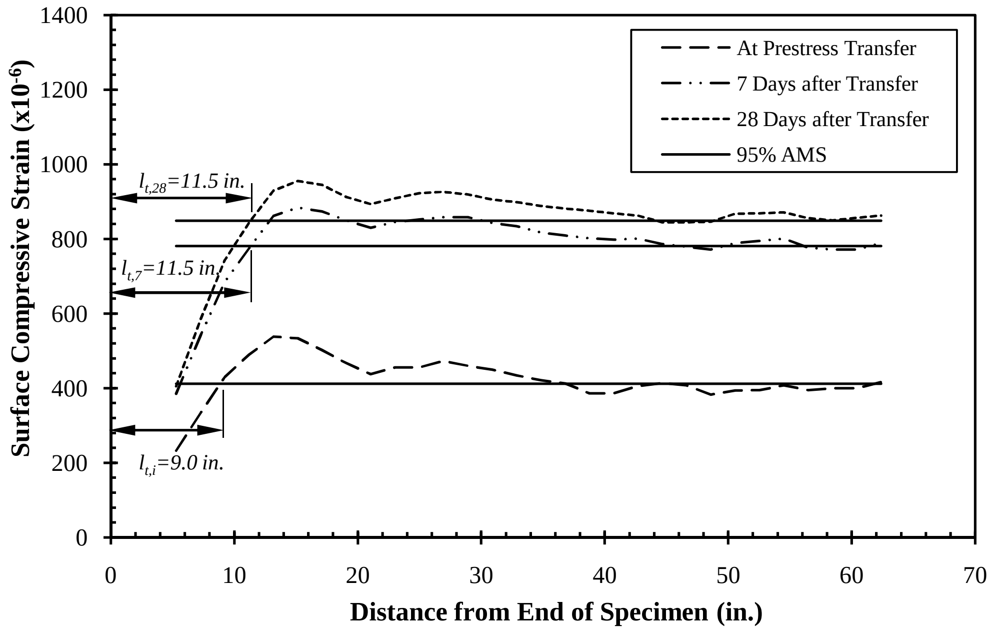



**Figure D-22: 72-11C-B-F Measured Initial and Long-Term Strain Profiles**

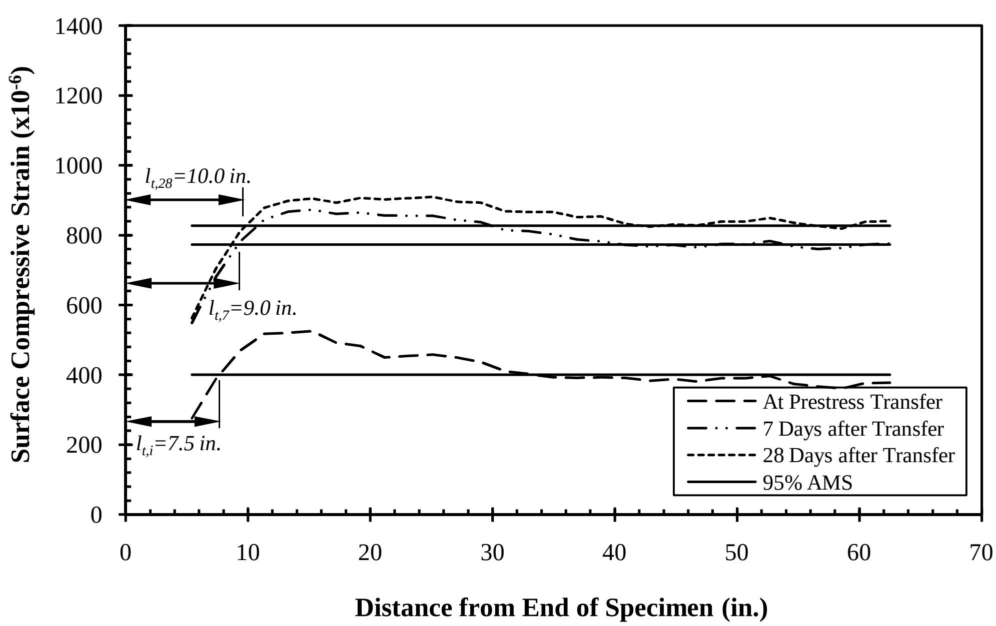



**Figure D-23: 72-13C-A-F Measured Initial and Long-Term Strain Profiles**

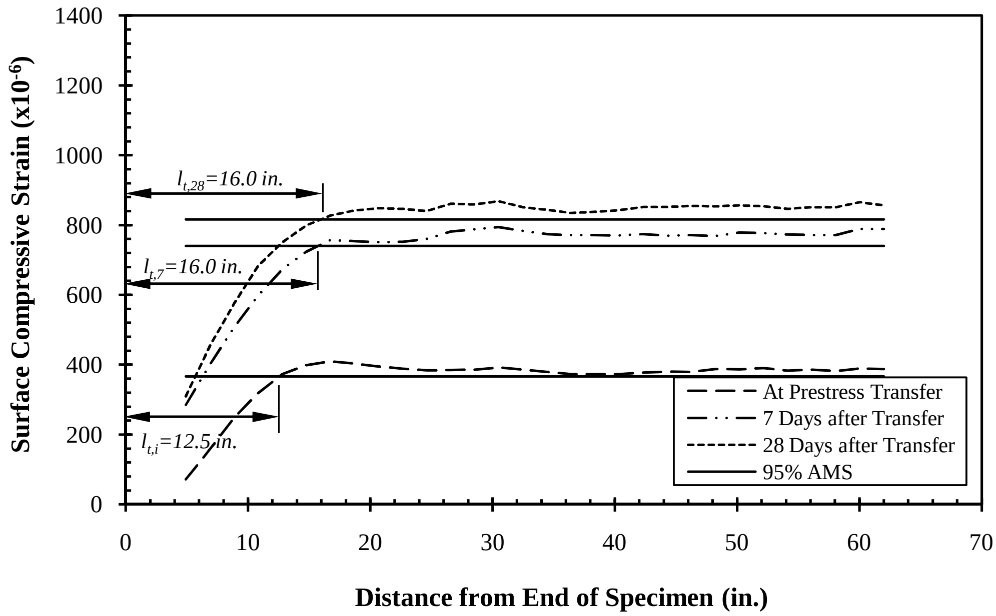



**Figure D-24: 72-13C-B-F Measured Initial and Long-Term Strain Profiles**

## Appendix E: Debonded Strand Compressive Strain Profiles

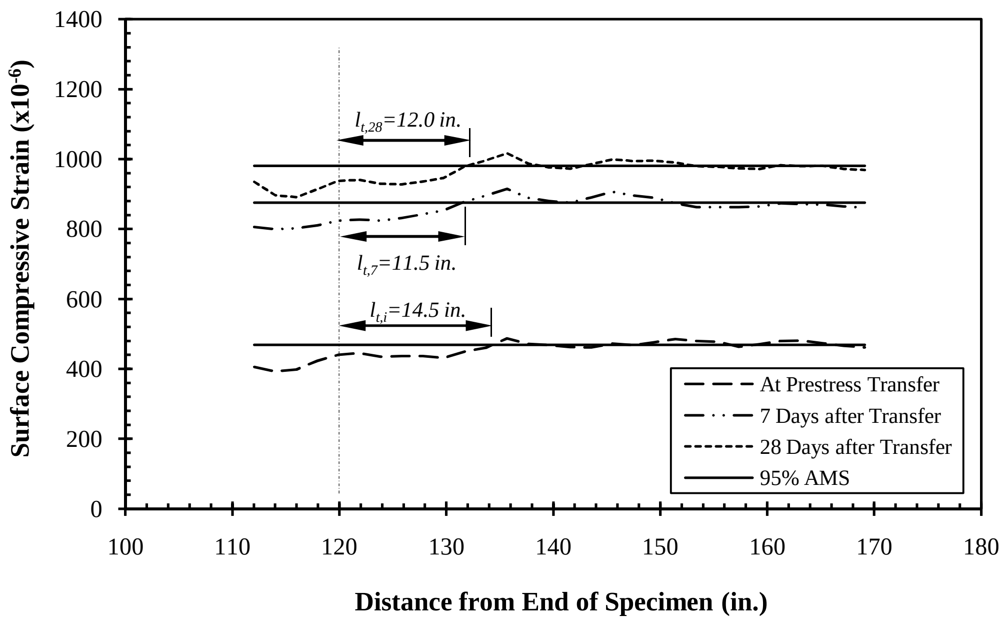



**Figure E-1: 54-2S-A-D Measured Initial and Long-Term Strain Profiles**

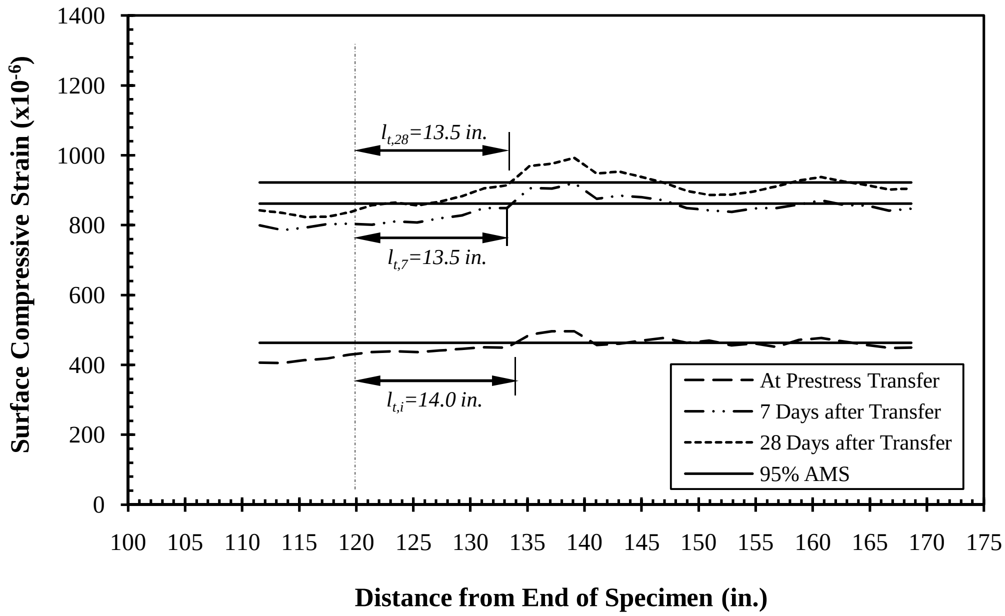



**Figure E-2: 54-4S-A-D Measured Initial and Long-Term Strain Profiles**

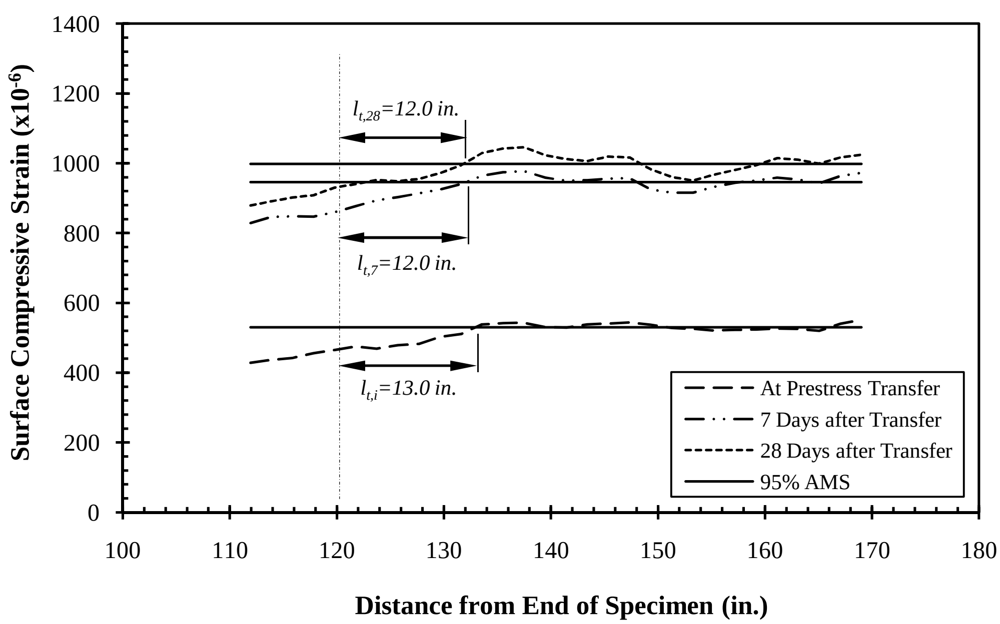



**Figure E-3: 54-11C-A-D Measured Initial and Long-Term Strain Profiles**

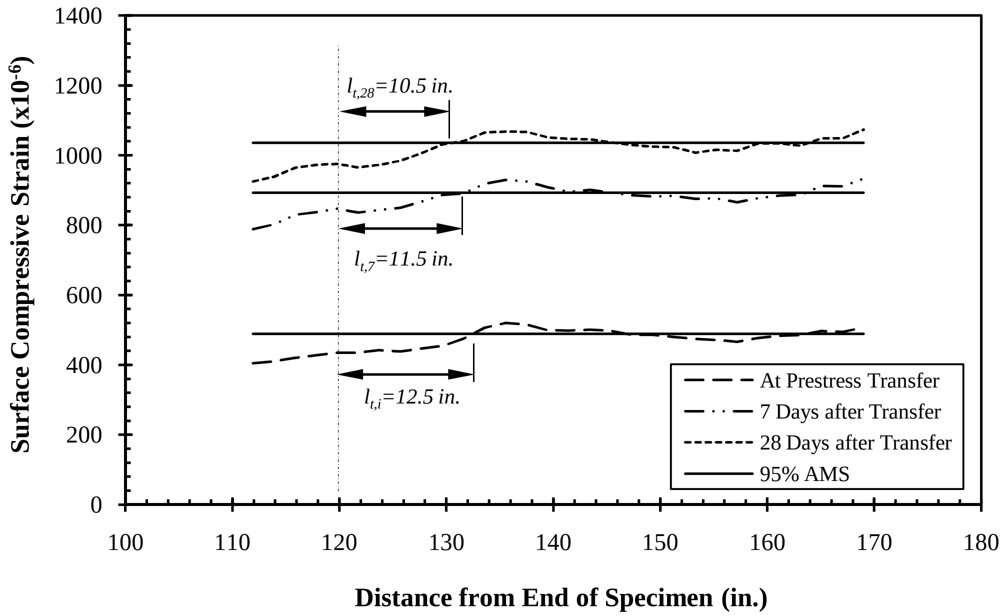



**Figure E-4: 54-13C-A-D Measured Initial and Long-Term Strain Profiles**

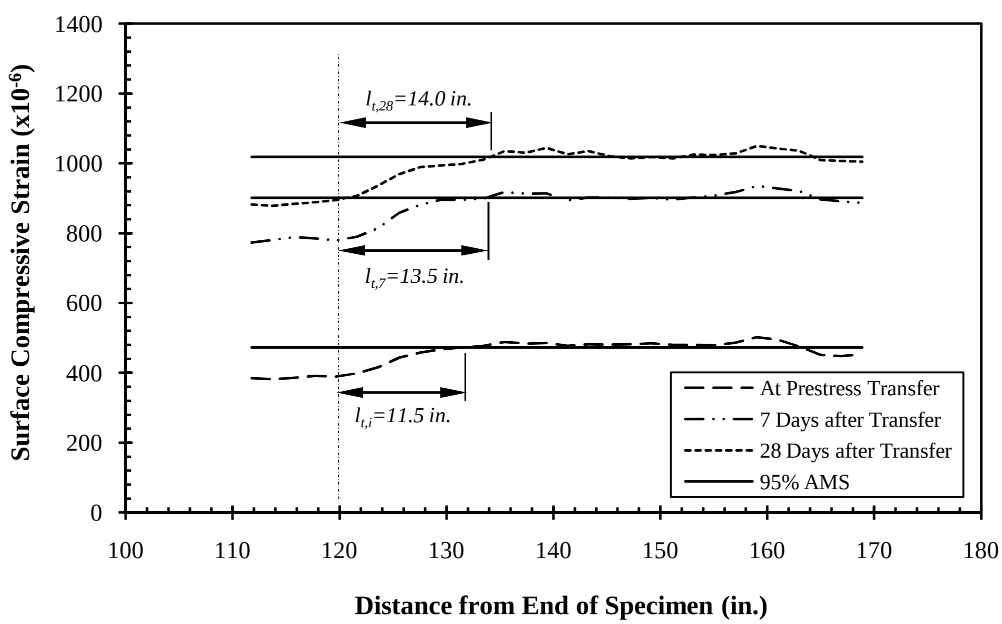



**Figure E-5: 72-2S-A-D Measured Initial and Long-Term Strain Profiles**

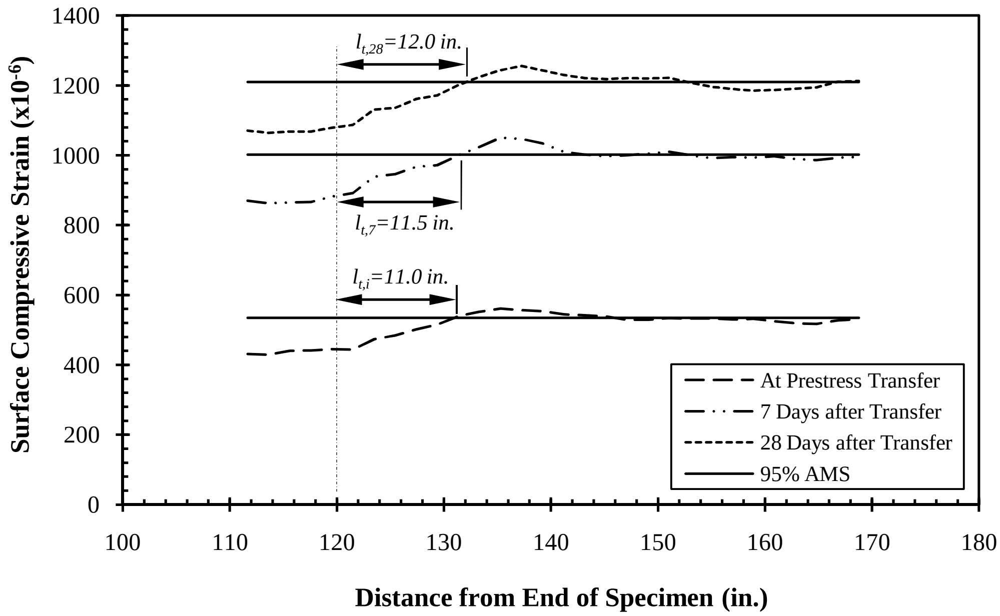



**Figure E-6: 72-4S-A-D Measured Initial and Long-Term Strain Profiles**

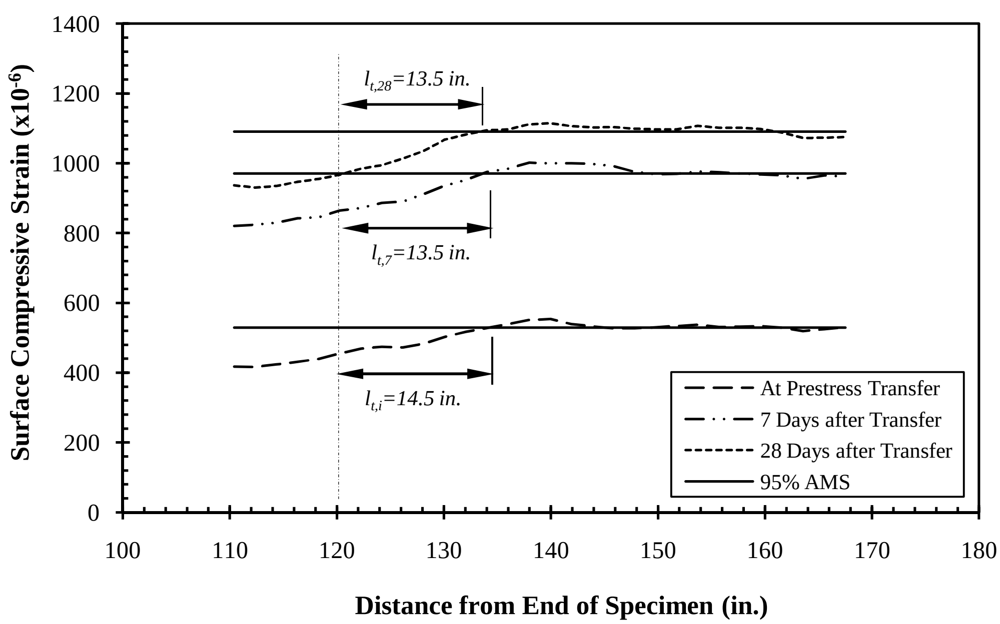



**Figure E-7: 72-7S-A-D Measured Initial and Long-Term Strain Profiles**

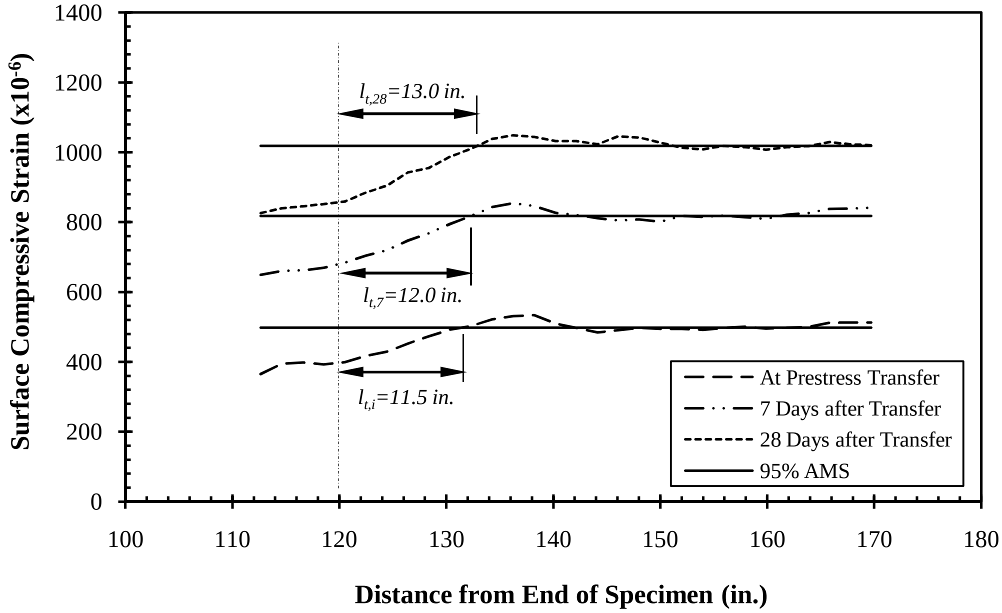



**Figure E-8: 72-8C-A-D Measured Initial and Long-Term Strain Profiles**

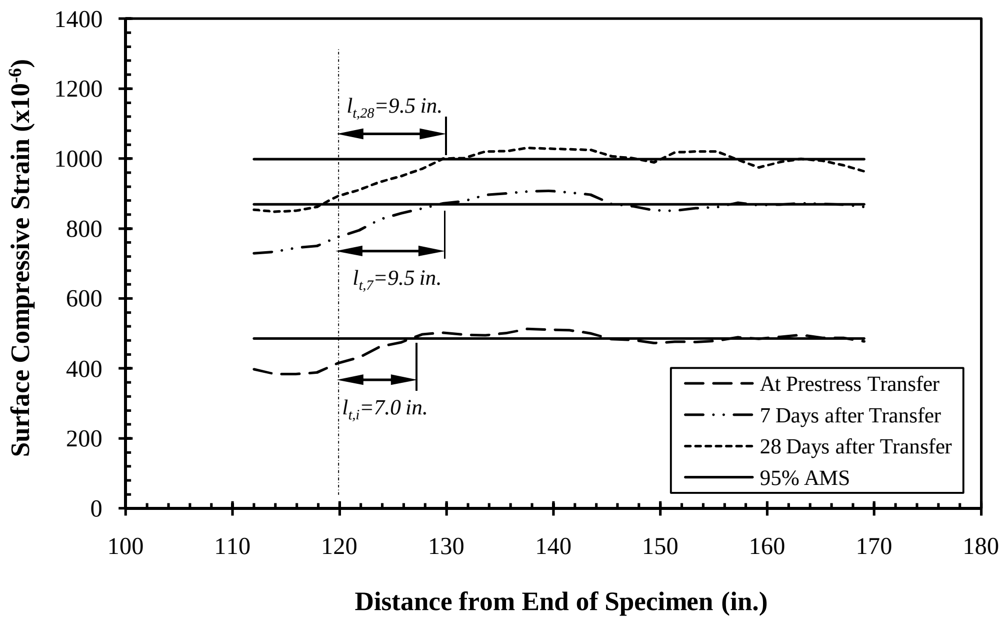



**Figure E-9: 72-11C-A-D Measured Initial and Long-Term Strain Profiles**

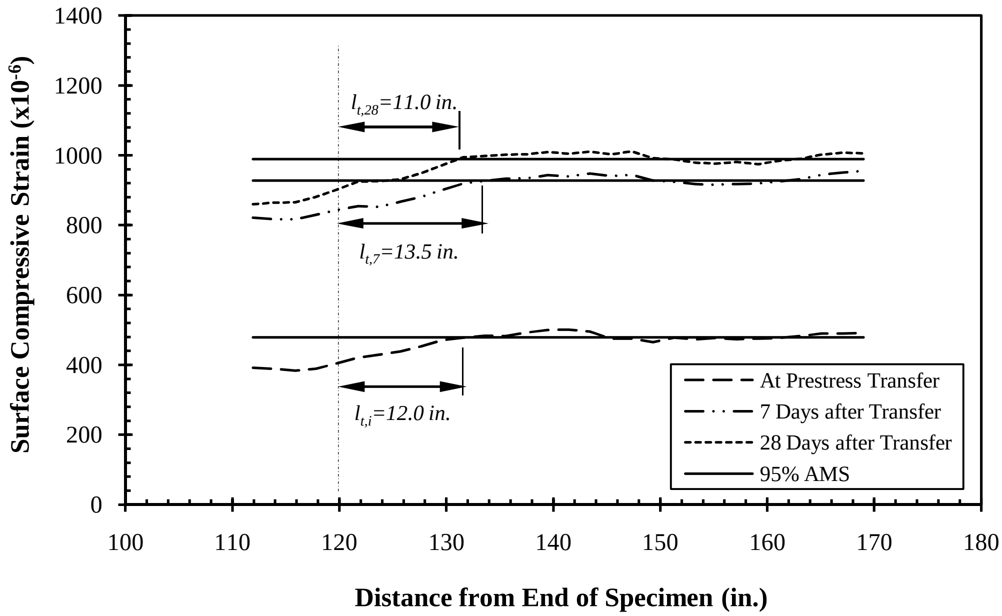



**Figure E-10: 72-13C-A-D Measured Initial and Long-Term Strain Profiles**