CIVIL ENGINEERING STUDIES
Illinois Center for Transportation Series No. 09-038
UILU-ENG-2009-2009
ISSN: 0197-9191

# TRANSFER AND DEVELOPMENT LENGTH OF PRESTRESSING TENDONS IN FULL-SCALE AASHTO PRESTRESSED CONCRETE GIRDERS USING SELF-CONSOLIDATING CONCRETE

Prepared By
**Bassem Andrawes**
**Moochul Shin**
**Andrew Pozolo**
University of Illinois at Urbana-Champaign

Research Report ICT-09-038

A report of the findings of

**ICT-R27-36**
**Transfer and Development Length of Prestressing Tendons in Full-Scale AASHTO Prestressed Concrete Girders using Self-Consolidating Concrete**

Illinois Center for Transportation

March 2009

**1. Report No.**
FHWA-ICT-09-038

**2. Government Accession No.**

**3. Recipient's Catalog No.**

**4. Title and Subtitle**
Transfer and Development Length of Prestressing Tendons in Full-Scale AASHTO Prestressed Concrete Girders using Self-Consolidating Concrete

**5. Report Date**
March 2009

**6. Performing Organization Code**

**8. Performing Organization Report No.**
ICT-09-038
UILU-ENG-2009-2009


**7. Author(s)**
Bassem Andrawes, Moochul Shin, and Andrew Pozolo

**9. Performing Organization Name and Address**
Illinois Center for Transportation
Department of Civil and Environmental Engineering
University of Illinois at Urbana-Champaign
205 N. Mathews Ave., MC-250
Urbana, IL 61801


**10. Work Unit ( TRAIS)**

**11. Contract or Grant No.**
ICT R27-36

**13. Type of Report and Period Covered**


**12. Sponsoring Agency Name and Address**
Illinois Department of Transportation
Bureau of Material and Physical Research
126 East Ash Street
Springfield, IL 62704-4766


**14. Sponsoring Agency Code**

**15. Supplementary Notes**


**16. Abstract**

Self-consolidating concrete (SCC) is a highly workable concrete that flows through densely reinforced or complex structural elements under its own weight. The benefits of using SCC include: **a)** Reducing labor costs by eliminating the need for mechanical vibration, **b)** Improving constructability, **c)** Providing a virtually flawless finish, **d)** Providing uniform and homogenous concrete, and **e)** Easily filling a complex shape formwork. Even though SCC is comparable to conventional concrete in terms of strength, the comparability of its bond to steel is less well-defined. This disparity of knowledge becomes more critical when using SCC in prestressed members due to the impact that bond strength has on the transfer and development lengths of prestressing tendons.

The increasing interest among Illinois precasters in using SCC in bridge girders has motivated the Illinois Department of Transportation (IDOT) and the Illinois Center for Transportation (ICT) to sponsor this synthesis study, which reviews and combines information from literature discussing the impact of using SCC on the transfer and development lengths of prestressing tendons in AASHTO bridge girders. The primary objectives of this study include: **(1)** Utilizing the results of previous research to evaluate the effect of using SCC on the transfer and development lengths of prestressing tendons and evaluate how SCC compares with conventional concrete, **(2)** Investigating the feasibility of using SCC in AASHTO bridge girders without the need for changing current design provisions recommended by the ACI and AASHTO, and **(3)** Providing IDOT with recommendations regarding the application of SCC in prestressed bridge girders.



**17. Key Words**
Self-Consolidating Concrete, AASHTO Girders, Prestressing Tendons, Transfer Length, Development Length


**18. Distribution Statement**
No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161.

| 19. Security Classif. (of this report) | 20. Security Classif. (of this page) | 21. No. of Pages | 22. Price |
|---|---|---|---|
| Unclassified | Unclassified | 62 | |

## ACKNOWLEDGEMENT / DISCLAIMER

This publication is based on the results of ICT-R27-36, *Transfer and Development Length of Prestressing Tendons in Full-scale AASHTO Prestressed Concrete Girders using Self-Consolidating Concrete*. ICT-R27-36 was conducted in cooperation with the Illinois Center for Transportation; the Illinois Department of Transportation; and the U.S. Department of Transportation, Federal Highway Administration.

The contents of this report reflect the view of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Illinois Center for Transportation, the Illinois Department of Transportation, or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

Trademark or manufacturers' names appear in this report only because they are considered essential to the object of this document and do not constitute an endorsement of product by the Federal Highway Administration, the Illinois Department of Transportation, or the Illinois Center for Transportation.

# EXECUTIVE SUMMARY

Self-consolidating concrete (SCC) is a highly workable concrete that flows through densely reinforced or complex structural elements under its own weight. The benefits of using SCC include: **a)** Reducing labor costs by eliminating the need for mechanical vibration, **b)** Improving constructability, **c)** Providing a virtually flawless finish, **d)** Providing uniform and homogenous concrete, and **e)** Easily filling a complex shape formwork. Even though SCC is comparable to conventional concrete in terms of strength, the comparability of its bond to steel is less well-defined. This disparity of knowledge becomes more critical when using SCC in prestressed members due to the impact that bond strength has on the transfer and development lengths of prestressing tendons.

The increasing interest among Illinois precasters in using SCC in bridge girders has motivated the Illinois Department of Transportation (IDOT) and the Illinois Center for Transportation (ICT) to sponsor this synthesis study, which reviews and combines information from literature discussing the impact of using SCC on the transfer and development lengths of prestressing tendons in AASHTO bridge girders. The primary objectives of this study include: **(1)** Utilizing the results of previous research to evaluate the effect of using SCC on the transfer and development lengths of prestressing tendons and evaluate how SCC compares with conventional concrete, **(2)** Investigating the feasibility of using SCC in AASHTO bridge girders without the need for changing current design provisions recommended by the ACI and AASHTO, and **(3)** Providing IDOT with recommendations regarding the application of SCC in prestressed bridge girders.

## TABLE OF CONTENTS

# CHAPTER 1 INTRODUCTION

## 1.1 PROBLEM DESCRIPTIOIN

Self-consolidating concrete (SCC) is a highly workable concrete that can flow through densely reinforced or complex structural elements under its own self weight. SCC is capable of adequately filling voids without segregation or excessive bleeding and without the need for mechanical or manual vibration (PCI 2003a). SCC was originally introduced in the mid 1980s by Okamura at Kochi University of Technology, Japan (Okamura 1999). The application of SCC has gradually increased since then due to its unique properties. The benefits of using SCC include: **a)** Reducing labor costs by eliminating the need for mechanical vibration, **b)** Improving constructability, **c)** Providing a virtually flawless finish, **d)** Providing homogenous and uniform concrete, and **e)** Easily filling a complex shape formwork. Given its potential benefits, the physical and mechanical properties of SCC have been the subject of extensive research. In the United States, the precast concrete industry is playing an important role in advancing the knowledge and application of SCC. A number of State Departments of Transportation such as those in Nebraska, Virginia, New York, and Kansas have already adopted the technology of SCC (Girgis and Tuan, 2004; Larson et al. 2007). Furthermore, the application of SCC is expected to grow rapidly in the next few years.

Despite the fact that SCC is comparable to conventional concrete in terms of its strength, the comparability of its bond to steel is less well-understood. To achieve the high flowability and stability characteristic of SCC, fabricators use more fine aggregates in conjunction with less or smaller coarse aggregates in SCC mix design. Changing the amounts and sizes of aggregates used in SCC from the amounts used in typical concrete is a primary reason why many researchers and practitioners are concerned about the bond strength of SCC. These concerns become more prominent when SCC is used in prestressed members because of uncertainty regarding the impact that SCC has on the transfer and development lengths of prestressing strands. The increasing interest among Illinois precasters in using SCC in bridge girders has motivated the Illinois Department of Transportation (IDOT) to sponsor this synthesis study to review and combine literature discussing the impact of using SCC on the transfer and development length of prestressing strands.

## 1.2 OBJECTIVES

The primary objectives of this synthesis are: (1) Utilize the results of previous research studies to evaluate the effect of using SCC in prestressed girders on the transfer and development lengths of prestressing strands and compare SCC to conventional concrete, (2) Investigate the feasibility of using SCC in typical AASHTO bridge girders without the need for changing the design provisions recommended by the ACI and AASHTO for the transfer and development lengths in prestressed girders, and (3) Provide IDOT with final recommendations regarding the application of SCC in prestressed bridge girders. In order to achieve these goals, a thorough literature review was conducted to summarize and combine the data and conclusions from previous research studies into one comprehensive report.

## 1.3 REPORT OUTLINE

This report consists of five chapters. Chapter 1 presents an introduction and background on the subject and the objectives of the research. The second chapter includes a general background discussion on SCC and the definitions of transfer and development lengths. Chapter 2 also describes the primary test methods used in assessing the transfer and development lengths in prestressed concrete girders. A summary of previous studies evaluating the transfer and development lengths of tendons in prestressed SCC members comprises Chapter 3. The fourth chapter presents two SCC mix designs slated for use in further research and discusses current Illinois SCC mix design standards for precast members. The fifth and final chapter illustrates the conclusions of the study and provides recommendations for future use of SCC in Illinois bridge girders.

# CHAPTER 2 GENERAL BACKGROUND

## 2.1 SELF-CONSOLIDATING CONCRETE

### 2.1.1 Physical Properties

As defined in Section 1.1, self-consolidating concrete is a highly workable concrete that can flow easily through densely reinforced or complex structural elements under its own weight and adequately fill voids without significant segregation or excessive bleeding, without the need for vibration (PCI 2003a). For a concrete mix to be considered as self-consolidating, the Precast/Prestressed Concrete Institute (PCI) suggests a minimum of three physical properties (criteria): a) flow ability, b) passing ability, and c) resistance to segregation. Several recommended tests, summarized in Table 2-1, can assess these criteria (PCI 2003a).

Table 2-1. Test Methods for Evaluating the Physical Properties of SCC

| Test | Purpose |
|---|---|
| Slump flow test | Flow ability |
| VSI rating | Resistance to segregation |
| T20 measurement | Filling ability or viscosity |
| L-Box, U-Box and J-ring | Passing ability |

### 2.1.2 SCC Mixes

Typical SCC mixes have higher paste volume, less coarse aggregate, and higher sand-to-coarse aggregate ratios than conventional concrete mixtures. Therefore, controlling the gradation of aggregates requires the use of special admixtures such as superplasticizers (SPs), viscosity modifying admixtures (VMAs), mineral fillers, or combinations thereof. Fly ash, slag, or limestone powder could be also used as mineral filler. High-Range Water-Reducing (HRWR) admixtures such as Polycarboxylate are typically utilized to improve the flowability of SCC without increasing the water-to-cement (W/C) ratio. Thus, SCC generally consists of cement, water, coarse aggregate, fine aggregate, additives (fly ash, slag, or limestone), and admixtures (SP, HRWR, or VMA). Using each component in the proper proportions is essential to achieve the physical properties characterizing SCC.

The design of SCC mixes varies significantly throughout the world depending on the concrete manufacturer. Figure 2-1 compares the volume percentage of the constituents used in SCC and those used in traditional concrete. Table 2-2, similarly, details examples of SCC mixes typically used in Japan, Europe, and the United States. Generally, each country adheres to the same guidelines in terms of the given percentage except the United States, which uses more fine aggregates (7-8%) compared to other countries.

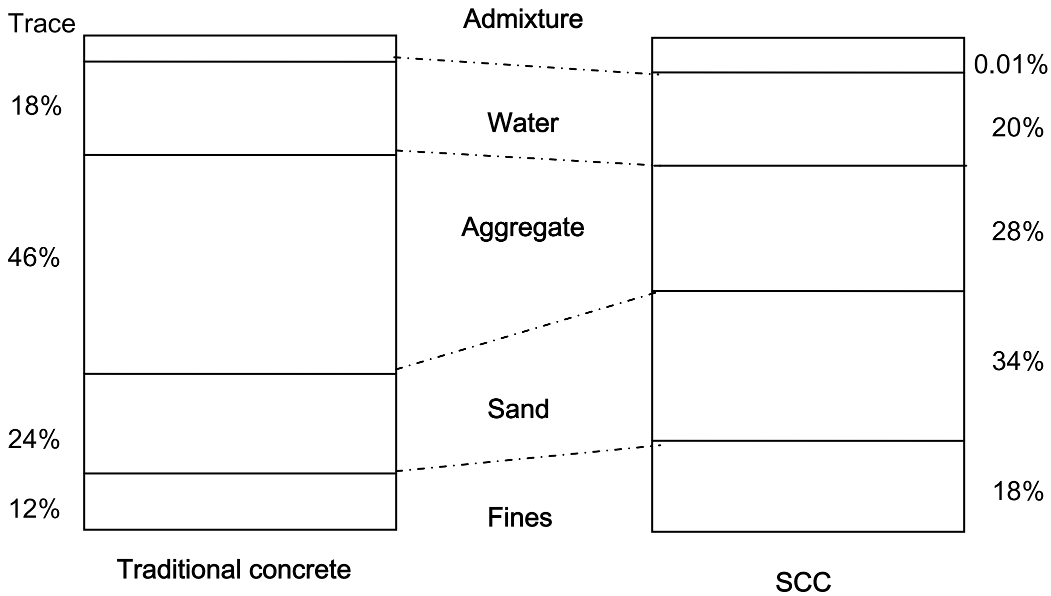


Figure 2-1. Typical volume percentage of constituents in SCC and traditional concrete (Gaimster and Gibbs, 2001).

### 2.1.3 Mechanical Properties

Previous studies have illustrated that hardened SCC shares similar mechanical characteristics with conventional concrete in terms of strength and modulus of elasticity (Persson 2001). As in conventional concrete, the mechanical performance is controlled primarily by paste content, aggregate proportions, and water-to-cement ratio. Many researchers (Khayat et al. 1996 & 2001; Persson 2001) have investigated the differences between the mechanical properties of SCC and conventional concrete. Most of the mechanical properties were not significantly different, except for creep and shrinkage. SCC mixtures are designed to have higher paste content or fines compared to typical concrete mixtures, which would likely cause an increase in the concrete shrinkage.

Table 2-2. SCC Mix Designs from Japan, Europe, and the United States
(JSCE 1999; Ouchi et al. 2003; PCI 2003a)

| Location – Mix # | Cement (%) | Coarse Agg. (%) | Fine Agg. (%) | Water (%) | Addictives (%) | Admixtures (%) | W/C |
|---|---|---|---|---|---|---|---|
| Japan – Mix 1 | 22.81 | 33.95 | 32.31 | 7.53 | 3.01 | 0.39 | 0.29 |
| Japan – Mix 1 | 9.51 | 35.66 | 37.82 | 7.13 | 9.51 | 0.37 | 0.38 |
| Japan – Mix 1 | 13.17 | 38.49 | 31.03 | 7.73 | 9.10 | 0.47 | 0.35 |
| Average | 15.16 | 36.03 | 33.72 | 7.47 | 7.21 | 0.41 | 0.34 |
| Europe – Mix 1 | 12.00 | 32.13 | 37.06 | 8.14 | 10.50 | 0.18 | 0.36 |
| Europe – Mix 1 | 14.06 | 31.95 | 37.06 | 8.18 | 8.52 | 0.23 | 0.36 |
| Europe – Mix 1 | 14.33 | 34.66 | 32.35 | 9.24 | 8.78 | 0.65 | 0.40 |
| Average | 13.46 | 32.91 | 35.49 | 8.52 | 9.27 | 0.35 | 0.37 |
| U.S. – Mix 1 | 17.77 | 26.82 | 45.81 | 7.58 | 1.96 | 0.07 | 0.38 |
| U.S. – Mix 1 | 15.67 | 30.02 | 41.08 | 7.90 | 5.22 | 0.11 | 0.38 |
| U.S. – Mix 1 | 16.77 | 35.97 | 40.92 | 6.21 | 0.00 | 0.13 | 0.37 |
| Average | 16.74 | 30.94 | 42.60 | 7.23 | 2.39 | 0.10 | 0.38 |

## 2.2 TRANSFER AND DEVELOPMENT LENGTH OF PRESTRESSING STRANDS

The pretensioning technique relies on the bond between steel strands and the surrounding concrete to transfer the stresses from the prestressing strands to the concrete. Figure 2-2 shows a schematic illustrating the variation in the strand stresses along the length of the beam starting from the free end of the strand. The stronger the bond strength, the shorter the length required to transfer a certain amount of stress between the steel and the concrete. Therefore, the strand length required for transferring the effective prestressing stress and developing its ultimate strength should be predicted with careful consideration. Otherwise, the prestressed member could fail prematurely due to splitting failure or pull-out failure (Collins and Michell, 1991).

### 2.2.1 Transfer Length

The transfer length ($L_t$) of a prestressing strand is defined as the length from the end of the strand to the point where the effective stress ($f_{se}$) is developed (see Figure 2-2). After releasing or cutting pretensioned strands, stress is gradually transferred to the surrounding concrete through direct bond. Therefore, a certain amount of length ($L_t$) is required for the prestressing force to transfer from the strands to the concrete. A number of factors control the transfer length in pretensioned members, including: 1) type of prestressing strands, 2) strand diameter, 3) effective stress ($f_{se}$), 4) strand surface condition, 5) concrete strength, 6) type of loading, 7) method of releasing the prestressing force, 8) concrete cover around the strands, and 9) strand spacing.

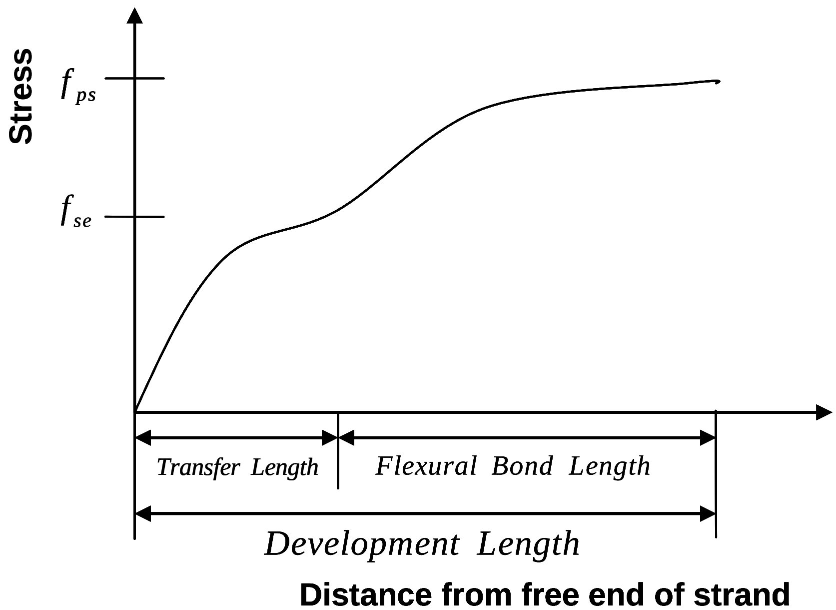


Figure 2-2. Stress variation along beam length.

The AASHTO-LRFD design specifications suggest that the transfer length for a strand should not exceed 60 times its diameter, while the flexural design guidelines in Section 12.9 of ACI 318-08 suggest using Equation 2-1 below for computing the recommended limit for the transfer length. $L_t$ is the transfer length in inches, $f_{se}$ is the effective stress in ksi, and $d_b$ is the diameter of the strand in inches.

$$L_t = \frac{1}{3} f_{se} d_b \qquad (2\text{-}1)$$

However, the ACI shear design guidelines in Section 11.3 state that the shear design of prestressed members is based on a transfer length equal to fifty times the strand diameter (ACI 2008). Even though there are many factors affecting the transfer length, both the AASHTO-LRFD and ACI recommendations suggest that the transfer length is primarily governed by either one or two parameters.

### 2.2.2 Development Length

Development length ($L_d$) is defined as the total embedment length of the strand required to reach a member's full design strength ($f_{ps}$). As illustrated in Figure 2-2, the development length is the summation of the transfer length and flexural bond length, which is the length required to resist flexural stresses. According to ACI 318-08, development length may be calculated using the following formula:

$$L_d = (f_{ps} - \frac{2}{3} f_{se}) d_b \qquad (2\text{-}2)$$

Where, $L_d$ and $d_b$ are both given in inches, while the stresses $f_{ps}$ and $f_{se}$ are given in ksi. The relationship in Equation 2-2 was further investigated in a study analyzing development length values in prestressed beams with epoxy-coated strands (Cousins et al. 1986). The outcome of the study indicated that Equation 2-2 is not conservative. Therefore, the Federal Highway Administration (FHWA) requires that the development length computed using Equation 2-2 be multiplied by a factor $K$ (greater than 1.0) until further research is conducted on the subject (PCI 2003b). As a result, the development length must satisfy Equation 2-3. For bonded strands in precast, prestress beam, $K$ = 1.6.

$$L_d \geq K(f_{ps} - \frac{2}{3} f_{se}) d_b \qquad (2\text{-}3)$$

## 2.3 TESTING METHODOLOGIES

### 2.3.1 Pull-out Test

The pull-out test, commonly referred to as the Moustafa test, is recommended by the PCI as an auxiliary test to evaluate the bond characteristics of prestressing strands (Logan 1997; PCI 2003a). Although the results of the pull-out test are related to bond behavior, the PCI suggests the test could be used as a tool to evaluate transfer and development lengths in prestressed concrete members. A detailed description of the pull-out test may be found in references (Logan 1997) and (Haq 2005). In his experimental study, Logan recommended a minimum of 16 kips for the first slip load and 36 kips for the maximum pull-out load, assuming a conventional concrete mixture and a 0.5-inch diameter strand. A summary of recommendations and guidelines for conducting the test is presented below:

a) The test is recommended for concrete with compressive strength between 3500 psi and 5900 psi. Table 2-3 presents the concrete mix design used by Logan for the pull-out test. The size of the concrete block used in the test is typically 24" x 24" with a 36" length and 18" embedment. Despite these recommendations, the block dimensions are flexible and are dependent on the number of tested strands. Figure 2-3 shows a schematic of the pull-out test setup.

Table 2-3. Normal Concrete Mix Design Used in Logan's Pull-out Test (Logan 1997)

| **Material** | **Quantity** (per $yd^3$) |
|---|---|
| Portland Cement (Type III) | 660 lb |
| Concrete Sand | 1100 lb |
| Crushed Gravel | 1900 lb |
| Water Reducer | 26 oz |
| Air-Entraining Agent | 0 oz |
| High-Range Water Reducer | 0 oz |
| Water | 35 gal |
| W/C | 0.44 |

b) A hydraulic jack with a minimum travel length of 12 inches should be used to pull out the strands. The maximum load used in pulling the strands shall not exceed 50 kips.

c) The jacking load is applied gradually (20 kips/min.) until the load gauge shows that the strand cannot carry any additional load. The maximum load shall be recorded before the load suddenly changes and the strand cannot sustain any more loads.

d) Four types of data should be recorded during the test: 1) maximum load capacity, 2) approximate load at first slip, 3) approximate pull-out distance at maximum load, and 4) a general depiction of failure. Typically, a poorly bonded strand would slip 8-10 inches before reaching its ultimate load, but a well-bonded strand would move only 1-2 inches. The maximum load recorded provides an indication of the bond capacity.

e) The test should be repeated as many times as needed and the data obtained should be used to compute an average failure load and standard deviation for each strand group.

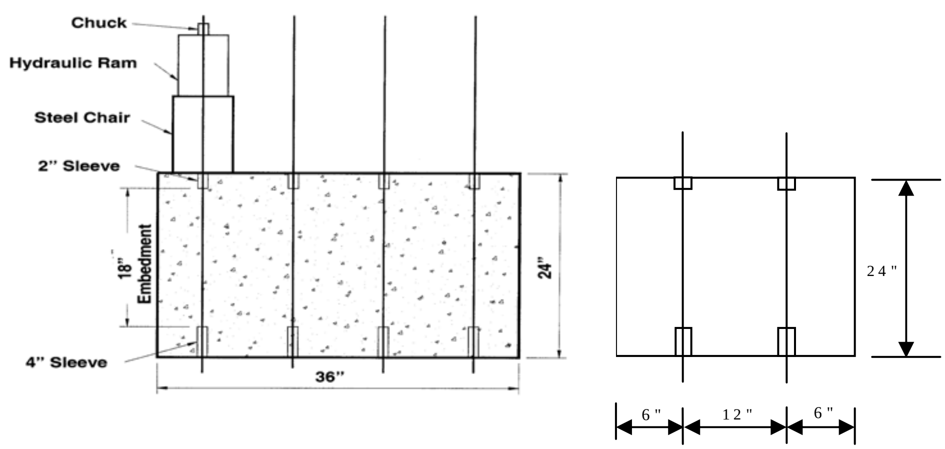


Figure 2-3. Schematic of the pull-out test setup (Haq 2005).

### 2.3.2 95% Average Maximum Strain Method

The 95% Average Maximum Strain (AMS) method is a common method used to evaluate the transfer length of prestressing strands. The test procedure is described below:

a) Mount gauge points and attach strain gauges to the concrete surface along the center line of strands at the end zone of the beam.
b) Record the strain at each point after releasing strands until a certain target date (usually up to 28 days).
c) Use the following formula to smooth the data, where $\varepsilon_{i-1}$, $\varepsilon_i$, and $\varepsilon_{i+1}$ are the recorded strains at *i-1*th, *i*th and *i+1*th gauge (Russel and Burns, 1993).

$$\varepsilon_{i,s} = \frac{\varepsilon_{i-1} + \varepsilon_i + \varepsilon_{i+1}}{3} \qquad (2\text{-}4)$$

d) Plot the strain vs. distance from the end of the beam relationship, as in Figure 2-4.
e) Mark the line of 95% of average maximum strain.
f) The transfer length is the length from the end of the beam to the point where the 95% average strain is first reached.

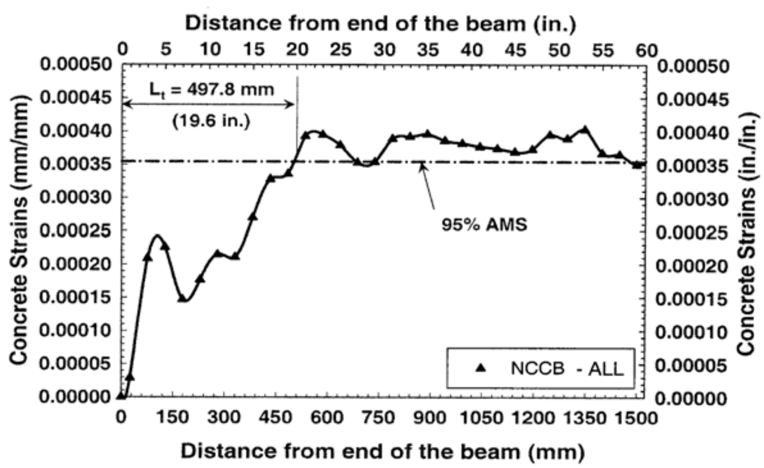


Figure 2-4. Strain vs. distance relationship from 95% AMS method data (Haq 2005).

### 2.3.3. End Slip Method

The end slip method, also referred to as the “draw-in method,” is another technique commonly used to evaluate the transfer length of prestressing strands (Haq 2005). This method is based on relating the transfer length of a strand with the amount of slippage measured at the end of the strand upon the release of the prestressing force. First, the strand draw-in $\Delta_d$ is calculated as follows:

$$\Delta_d = \delta_s - \delta_c \quad (2\text{-}5)$$

Here, $\delta_s$ is the change in the strand's length in the stress transfer zone due to prestress release, and $\delta_c$ is the elastic shortening of the concrete in the stress transfer zone due to prestress release. By integrating the strains of the strand and the concrete along the transfer length, $\delta_s$ and $\delta_c$ can be calculated as follows:

$$\Delta_d = \int_{L_t} (\Delta\varepsilon_s - \Delta\varepsilon_c)dx \quad (2\text{-}6)$$

In Equation 2-6, $\Delta\varepsilon_s$ is the change in the strand strain due to prestress release, and $\Delta\varepsilon_c$ is the change in the concrete strain due to prestress release. If the change in the strand and concrete strains is linear, Equation 2-6 can be expressed in the following, simpler form:

$$\Delta_d = \frac{f_{si}}{\alpha E_{ps}} L_t \quad (2\text{-}7)$$

In Equation 2-7, $f_{si}$ is the initial stress in the strand, $E_{ps}$ is the Young's modulus of the strand, α is the stress distribution constant, and $L_t$ is the transfer length. Balazs (1993) reported a value of 2 for parameter α in the case of constant stress distribution and a value of 3 in the case of linear stress distribution. Typically, the stress distribution is assumed to be constant. Thus, the transfer length can be calculated as follows:

$$L_t = \frac{2E_{ps}\Delta_d}{f_{si}} \quad (2\text{-}8)$$

### 2.3.4 Flexural Test

The flexure test (three point bending or four point bending) is typically used to determine the development length in prestressed concrete members. An iterative testing process is often needed to evaluate accurately the development length of prestressing strands. The distance between the applied load and the beam end is referred to as the embedment length. If the beam fails due to bond failure, the embedment length is increased. Otherwise, if the beam fails in flexure, the embedment length is decreased. The procedure is repeated until bond failure and flexural failure occur simultaneously. In this case, the embedment length is taken as the development length. Figure 2-5 shows a schematic describing the test procedure using the three point bending test. As shown in the figure, if the beam failed due to bond failure, the load is moved to the right (direction ii). However, if the beam fails in flexure, the load is moved back to the left (direction i).

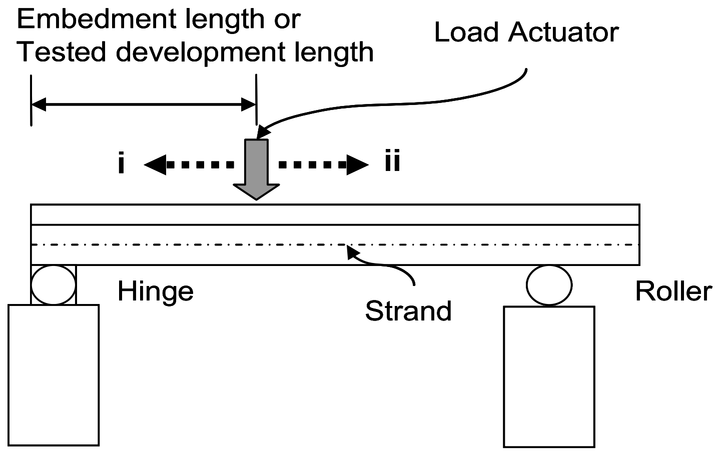


Figure 2-5. Flexural test used to evaluate development length of prestressing tendons.

# CHAPTER 3 SUMMARY OF PREVIOUS RESEARCH

A comprehensive literature review was conducted on the subject of transfer and development length of prestressed tendons used in SCC girders. The University of Illinois Library Databases (Grainger Engineering Library, NTIS, and Compendix, etc.) and the Transportation Research Information Services (TRIS, TRB) were the primary sources of the information presented in this chapter of the report. The search revealed seven major experimental studies conducted on the subject at different universities in the United States. This chapter presents a summary of these studies.

## 3.1 LARSON et al. (2006 & 2007)

### 3.1.1 Objectives

Larson et al. conducted a number of research studies sponsored by the Kansas Department of Transportation (KDOT) with the aim of characterizing the properties of pretensioned bridge girders cast with SCC (Larson et al. 2006 & 2007). A recently published report summarized the researchers' work and findings. The researchers evaluated transfer and development lengths using fifteen full-scale specimens cast with SCC, comparing the results to values recommended by the AASHTO and ACI design codes.

### 3.1.2 Material Properties

Prestressed Concrete Incorporated of Newton, Kansas provided the conventional concrete and SCC mix designs for the study. Table 3-1 shows the constituent materials of each mix. As shown in the table, a lower water to cement ratio was used in the SCC mix than in the conventional concrete mix. The maximum aggregate size used for both concrete types was 3/4 inches. No information was provided on the type of high-range water reducing admixture used in producing the SCC mix. The compressive strength of the concrete was obtained after various curing times, ranging from 0-70 days. At 28 days, the compressive strength of the SCC was found to be 7500 psi. The inverted-slump flow, visual stability index, J-Ring, and L-Box tests were performed on the concrete mixes used in the study. The results are presented in Table 3-2. As indicated in the table, three types of specimens were used: 1) Single-strand beams (SSB), 2) top-strand beams (TSB) and 3) T-beams (TB).

Table 3-1. SCC and Normal Concrete Mix Designs (Larson et al. 2007)

| Materials | SCC (Quantity per $yd^3$) | NC (Quantity per $yd^3$) |
|---|---|---|
| Cement (Type III) | 750 lbs | 650 lbs |
| Fine Aggregate | 1500 lbs | 1480 lbs |
| Coarse Aggregate | 1360 lbs | 1457 lbs |
| Air Entrainment | 5 oz | 6 oz |
| HRWR | 70 oz | 26 oz |
| VMA | 0 oz | 0 oz |
| Water | 27 gal | 31.6 gal |
| w/c | 0.3 | 0.41 |

Table 3-2. Test Results for SCC Mixes (Larson et al. 2007)

| | Specimen | Slump flow (inch) | VSI | J-Ring (in) | L-Box | Strength @ release (psi) | Strength @ testing (psi) |
|---|---|---|---|---|---|---|---|
| Bottom Strand | SSB A | 21 | 0.5 | 19 | 0.8 | 5,000 | 8,250 |
| | SSB C | 21 | 0.5 | 19 | 0.8 | 5,000 | 6,960 |
| | SSB D | 22 | 0.5 | 21 | 0.83 | 5,000 | 7,430 |
| | SSB E | 22 | 0.5 | 21 | 0.83 | 5,000 | 7,710 |
| | SSB F | 22 | 0.5 | 21 | 0.83 | 5,000 | 7,190 |
| Top Strand | TSB A | 28 | 0.5 | 26 | 0.88 | 3,600 | 6,570 |
| | TSB B | 28 | 0.5 | 26 | 0.88 | 3,600 | 7,150 |
| | TSB C | 28 | 0.5 | 26 | 0.88 | 3,600 | 6,940 |
| | TSB D | 28 | 0.5 | 26 | 0.88 | 3,600 | 7,790 |
| | TSB E | 28 | 0.5 | 26 | 0.88 | 3,600 | 7,330 |
| | TSB F | 28 | 0.5 | 26 | 0.88 | 3,600 | 6,100 |
| T-Beam | TB A | 17 | 0.5 | 14 | 0.78 | 5,200 | 7,550 |
| | TB B | 22 | 0.5 | 21 | 0.83 | 4,800 | 7,920 |
| | TB C | 21 | 1 | 18 | 0.83 | 5,200 | 8,300 |
| | TB D | 22 | 0.5 | 21 | 0.83 | 4,800 | 8,070 |

### 3.1.3 Pull-out Test

Using the concrete mix adopted in the Logan study (Table 2-3), pull-out tests were conducted to assess the adequacy of the 0.5-inch diameter strands used in the large-scale specimens. As previously mentioned, a minimum value of 16 kips for the first slip load and 36 kips for the maximum pull-out load were recommended. The test results showed that the average first slip load was 21.6 kips and that the average maximum pull-out load was 39.6 kips. Thus, researchers concluded that the strands had sufficient bond strength and could be used for the flexural tests.

### 3.1.4 Design of Test Specimens

The section dimensions of the three specimen types used in the study are presented in Figure 3-1. As depicted in the figure, both the SSB and TSB sections contained only one strand. To study the effect of using multiple strands with close spacing, researchers included the TB section in the study. The TSB section was considered in the study to investigate the possibility of encountering a reduction in the bond strength when using top strands.

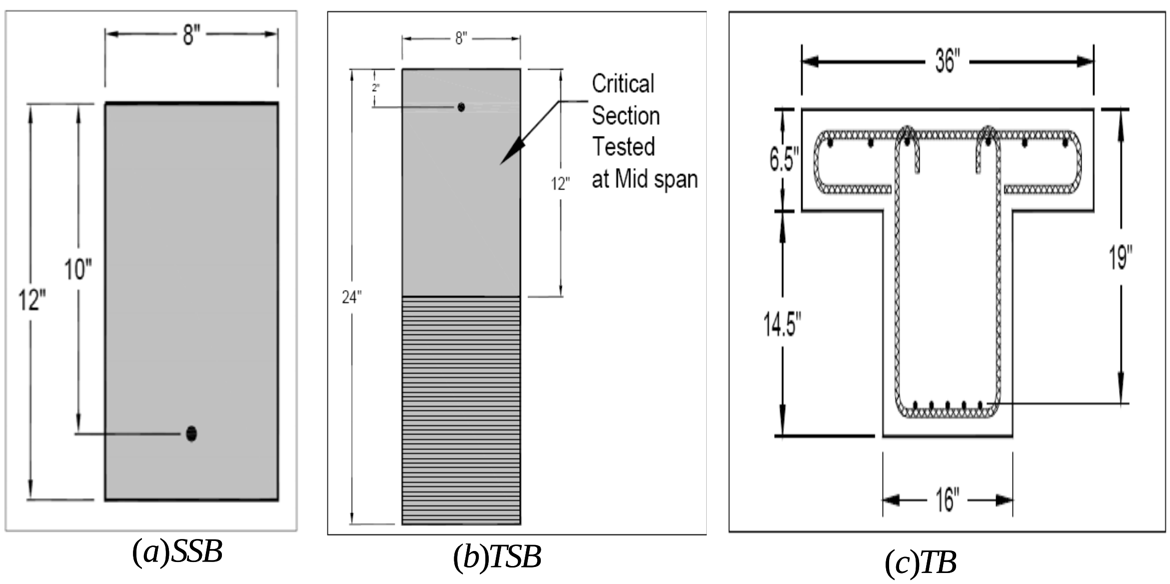


Figure 3-1. Cross-section of three specimen types (Larson et al. 2007).

Figures 3-2 and 3-3 show an elevation view of the beams tested in flexure during the project. The four point bending method was used as the flexural bending test. Three SSB and two TB specimens were cast with 13’-2” and 15’-6” span lengths. The embedment length was determined based on the load location. However, for the TSB specimens, the load location was fixed throughout the tests and the embedment length was varied based on the beam's span length. An embedment length of 6'-1'' was the base length from which the researchers started to evaluate the development length of tested beams. If a beam failed in flexure, the test was repeated with an embedment length equal to 80% of the base length (4'-10''). However, if a bond, slip, or shear failure was observed the embedded length was increased to 120% of the base length (7'-3'').

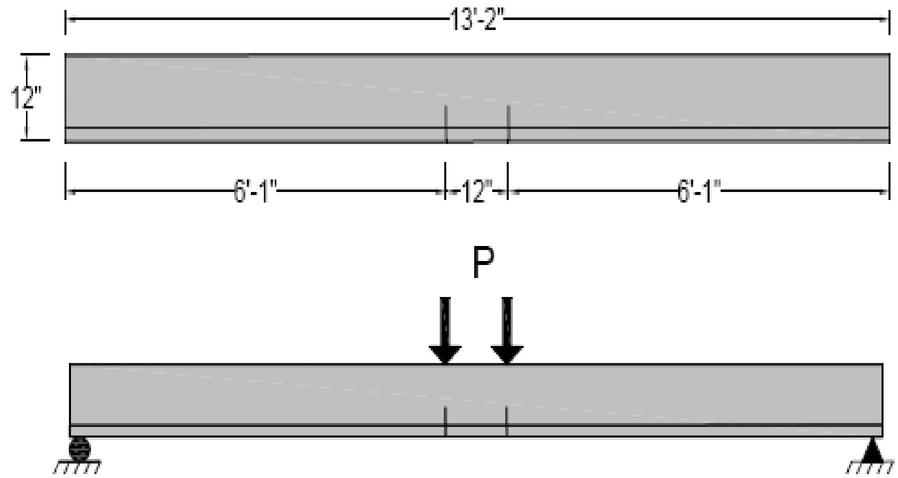


Figure 3-2. SSB and TB with embedment length of 6'-1'' (Larson et al. 2007).

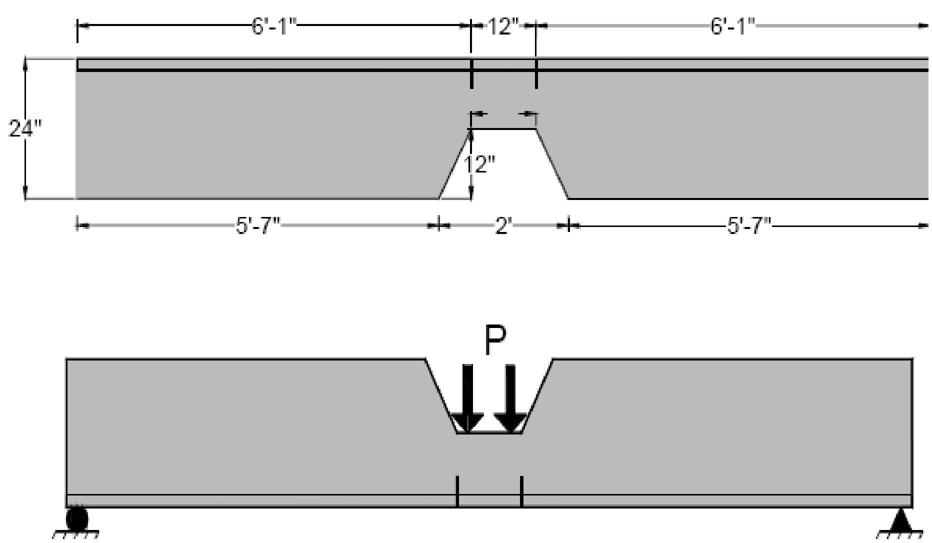


Figure 3-3. TSB with embedment length of 6'-1'' (Larson et al. 2007).

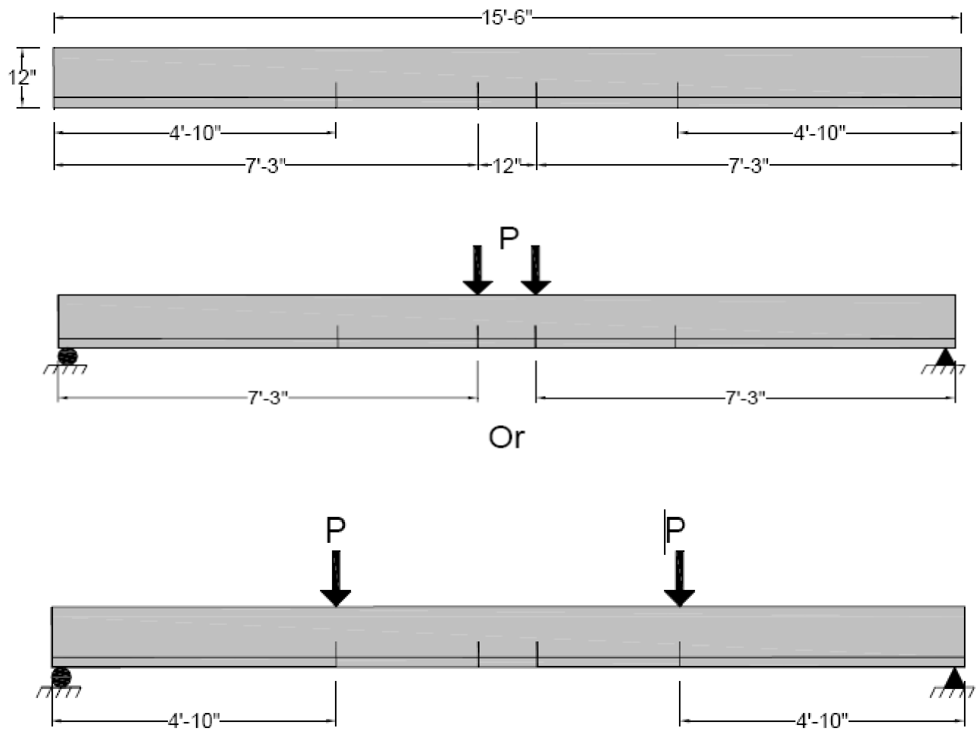


Figure 3-4. SSB or TB with embedment lengths of 7’-3’’ and 4’-10’’ (Larson et al. 2007).

### 3.1.5 Results

*3.1.5.1 Transfer Length*

The “end slip” method was used to evaluate the transfer length of the strands in the three specimen types studied. Based on an effective stress of 185 ksi and a strand diameter of 0.5 inches, the ACI code (Equation 2-1) recommends a maximum transfer length of 31 inches. However, according to the AASHTO-LRFD and ACI codes (shear design requirements), the recommended transfer length was $60d_b$ = 30 inches and $50d_b$ = 25 inches, respectively. Tables 3-3, 3-4, and 3-5 show the transfer length results of the SSB, TSB, and TB specimens, respectively, at the time of releasing the strands, 18 days after releasing the strands, and at the day the flexural tests were conducted. The average transfer length values for all tests are also shown at the bottom row of each table. As shown in Tables 3-3 and 3-4, six specimens (A, B, C, D, E and F) were tested for each of the SSB and TSB specimen types. The transfer length was evaluated at each end of the beam (ends A and B). For the TB specimen type, four specimens (A, B, C and D) were tested as illustrated in Table 3-5. The transfer lengths at both ends of the beam (ends 1 and 2) were recorded. Furthermore, the transfer length of each of the five strands used in the TB specimens was recorded and presented in the table (strands A, B, C, D, and E).

Table 3-3. Transfer Length Results for SSB Specimens (inches) (Larson et al. 2007)

| Beam | Embedment Length | Release | | 18 Days | | Flexural Test Day | |
|---|---|---|---|---|---|---|---|
| | | A | B | A | B | A | B |
| SSB A | 6'-1" | 16 | 17 | 26 | 30 | 27 | 30 |
| SSB B | 6'-1" | 18 | 24 | 16 | 30 | 16 | 30 |
| SSB C | 6'-1" | 17 | 7 | 24 | 11 | 23 | 11 |
| SSB D | 4-10" | 30 | 25 | 29 | 31 | 29 | 31 |
| SSB E | 4-10" | 19 | 19 | 13 | 17 | 14 | 17 |
| SSB F | 4-10" | 12 | 15 | 10 | 16 | 10 | 16 |
| Average | | 18 | | 21 | | 21 | |

The average results in Tables 3-3, 3-4, and 3-5 were below the ACI and AASHTO code requirements. However, some individual test results violated both the AASHTO and ACI codes. This observation is especially true in the case of TSB and TB specimens where top strands or closed-spacing strands were used. The maximum violations recorded were when the TSB and TB specimens exceeded the AASHTO code requirement (30 inches) by 20% and 46.7%, respectively, and the ACI code requirement (25 inches) by 44% and 76%, respectively. Moreover, the results show that the transfer length value was stable with time and experienced almost no or minimal increase after 18 days of releasing the strands.

Table 3-4. Transfer Length Results for TSB Specimens (inches) (Larson et al. 2007)

| Beam | Embedment Length | Release | | 18Days | | Flexural Test Day | |
|---|---|---|---|---|---|---|---|
| | | A | B | A | B | A | B |
| TSB A | 4'-10" | 17 | 19 | 30 | 34 | 30 | 34 |
| TSB B | 4'-10" | 21 | 13 | 30 | 24 | 30 | 25 |
| TSB C | 4'-10" | 15 | 13 | 34 | 31 | 34 | 31 |
| TSB D | 6'-1" | 15 | 17 | 22 | 19 | 23 | 19 |
| TSB E | 6'-1" | 8 | 21 | 20 | 31 | 22 | 31 |
| TSB F | 6'-1" | 8 | 15 | 32 | 23 | 36 | 25 |
| Average | | 15 | | 28 | | 28 | |

Comparing the average values presented in Tables 3-3 and 3-4 reveals that using top strands could have a negative impact on transfer length. Although the average transfer length of the TB specimens was below the 30” AASHTO requirement, the individual results for samples tested at the date of flexural testing failed to meet the same requirement 36.4% of the time. Furthermore, they failed to meet the 25” ACI requirement 48.5% of the time. Therefore, closely spaced strands could have affected the bond strength negatively.

Table 3-5. Transfer Length Results for TB Specimens (inches) (Larson et al. 2007)

| **Beam** | **Release** | | | | | **18 Days** | | | | | **Flexural Test Day** | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | **A** | **B** | **C** | **D** | **E** | **A** | **B** | **C** | **D** | **E** | **A** | **B** | **C** | **D** | **E** |
| A1 | | | 19 | 25 | 6 | | | 28 | 34 | 13 | | | 28 | 34 | 14 |
| A2 | 18 | 28 | 16 | 41 | 20 | 24 | 36 | 30 | 44 | 25 | 25 | 36 | 30 | 44 | 25 |
| B1 | | 11 | 40 | 19 | 7 | | 22 | 41 | 11 | 16 | | 22 | 41 | 15 | 17 |
| B2 | 11 | | | | 6 | 14 | | | | 8 | 18 | | | | 11 |
| C1 | 20 | 18 | 25 | 28 | 19 | 23 | 20 | 28 | 31 | 17 | 23 | 20 | 28 | 31 | 19 |
| C2 | 26 | 28 | 42 | 31 | 25 | 28 | 38 | 42 | 40 | 31 | 28 | 38 | 42 | 41 | 31 |
| D1 | 28 | 32 | | 11 | 10 | 30 | 35 | | 15 | 21 | 30 | 35 | | 16 | 22 |
| D2 | 22 | 28 | 17 | 30 | 20 | 25 | 22 | 19 | 31 | 21 | 25 | 24 | 20 | 31 | 22 |
| Average | 22 | | | | | 26 | | | | | 27 | | | | |

*3.1.5.2 Development Length*

Using the four point bending test, as shown in Figures 3-2, 3-3, and 3-4, allowed for the simultaneous testing of both ends of a specimen. According to Equation 2-2, the recommended development length for the specimens was equal to 6’-1’’. The test results showed that specimens with the 6’-1’’ and 4’-11’’ embedment lengths both failed in flexure. Therefore, it was concluded that the development lengths of the strands satisfied AASHTO and ACI code provisions. Table 3-6 gives a summary of the flexure test results including the experimental and theoretical moment capacities. The results indicate that in all cases, the moment capacity measured from the experiment was higher than that predicted theoretically, further validating that all strands were fully developed in the specimens.

Table 3-6. Results from Flexural Tests (Larson et al. 2007)

| Beam | Embedment Length | Nominal Moment | Experimental Moment | $M_{exp}/M_n$ | Strand Rupture | Strand Slip > 0.01in |
|---|---|---|---|---|---|---|
| SSB A | 6'-1" | 33 | 36.6 | 1.11 | Yes | No |
| SSB C | 6'-1" | 33 | 38.2 | 1.16 | Yes | No |
| SSB D | 4-10" | 29.4 | 39.6 | 1.35 | Yes | No |
| SSB E | 4-10" | 29.4 | 37.5 | 1.28 | Yes | No |
| SSB F | 4-10" | 29.4 | 38.8 | 1.32 | Yes | No |
| TSB A | 4-10" | 29.4 | 38.9 | 1.32 | Yes | No |
| TSB B | 4-10" | 29.4 | 39.1 | 1.33 | Yes | No |
| TSB C | 4-10" | 29.4 | 38.6 | 1.31 | Yes | No |
| TSB D | 6'-1" | 33 | 36.6 | 1.11 | Yes | No |
| TSB E | 6'-1" | 33 | 37.3 | 1.13 | Yes | No |
| TSB F | 6'-1" | 33 | 35.7 | 1.08 | Yes | No |
| TB A | 6'-1" | 319 | 370 | 1.16 | Yes | No |
| TB B | 6'-1" | 319 | 383 | 1.20 | Yes | No |
| TB C | 4-10" | 280 | 359 | 1.28 | Yes | No |
| TB D | 4-10" | 280 | 376 | 1.34 | Yes | No |

### 3.1.6 Conclusions

1) The transfer lengths in the study were near those suggested by ACI and AASHTO. However, in some cases the transfer lengths failed to meet the 25” ACI criterion by as much as 44% (TSB) and 76% (TB).
2) The results showed the transfer length increasing until the 18$^{th}$ day after releasing the strands, after which no further increase was observed.
3) The location of the strands (top or bottom) affected the transfer length. Specimens with top-strands had on average a greater transfer length compared to those with bottom strands. The development length was not affected by the location of the strand.
4) Using closely spaced strands could have a negative effect on the transfer length.
5) The development lengths measured were satisfactory with an approximate value of 80% of the values recommended by the AASHTO-LRFD and ACI code provisions.

## 3.2 TRENT (2007)

### 3.2.1 Objectives

The Shockey Precast Group of Winchester, VA sponsored research at the Virginia Polytechnic Institute and State University to investigate the effect of SCC on the bond strength of tendons in prestressed concrete beams. One of the primary objectives was to examine the transfer and development lengths in concrete girders cast with SCC and conventional concrete (CC). Three rectangular beams were tested to study transfer length; two of the beams were cast with SCC and the third was cast with conventional concrete. In addition, a total of twelve prestressed T-beams were cast with SCC and CC and were tested in flexure to evaluate the development lengths of their prestressing strands.

### 3.2.2 Material Properties

Three concrete mixes were used in the study, two of which were SCC mixes (S1CCM and S1CCM2), while the third was a conventional concrete mix (S1CRM). The constituents of each of the three mixes are presented in Table 3-7. All three concretes had the same initial target compressive stress of 3500 psi at 12 hours, and 6500 psi at 28 days. The main difference between S1CCM and S1CCM2 was the type of fine aggregate used in the mix. In S1CCM, manufactured sand was the only fine aggregate used, while in S1CCM2, manufactured sand represented 65% of the total fine aggregate and a natural sand blend comprised the remaining 35%. No information was provided regarding the amount of admixtures used in producing the mixes.

### 3.2.3 Properties of Test Beams

The beams used to evaluate the transfer length had the 6” x 6” square section shown in Figure 3-5. Each beam was prestressed using a single 9/16-inch diameter grade 270 low relaxation strand. Figure 3-6 shows the casting layout of the three beams used in evaluating the transfer length before releasing the strands. As expected, the two edge specimens (S1CRM and S1CCM2) absorbed most of the energy once the strands were released. The prestress force then transferred gradually and uniformly to the middle specimen (S1CCM).

The girders used in evaluating the development length had the cross section shown in Figure 3-7. The shape of the cross section was selected by the precaster to fit the forms that were available at the time. Figure 3-8 depicts the casting layout of the four girders for each concrete mixture used in the flexural tests. Thus, a total of twelve beams were tested to evaluate the development length.

Table 3-7. Constituents of the Three Concrete Mixes (Trent 2007)

| Material | S1CRM (WT1Beams) | S1CCM (WT2Beams) | S1CCM2 (WT3Beams) |
|---|---|---|---|
| Type I Cement (lb) | 705 | 750 | 745 |
| Coarse Aggregate (lb) | 1750 (#57stone) | 1625 (#67stone) | 1650 (#67stone) |
| Fine Aggregate (lb) | 1256 | 1340 | 1308 |
| Water (gal) | 30.5 | 34 | 34 |
| Air Entraining Agent-AEA-14(oz): no info. | | | |
| ASTMC494-Plastiment: no info. | | | |
| ASTM C494 -ViscoCrete 4100 (oz) : no info. | | | |
| Unit Weight (lb/ft$^3$) | 146.9 | 148.1 | 148 |
| Slump/Flow (in.) | 8 | 25 | 21.5 |
| Air Content (%) | 4.8 | 5.2 | 5.8 |
| W/C ratio | 0.36 | 0.38 | 0.38 |

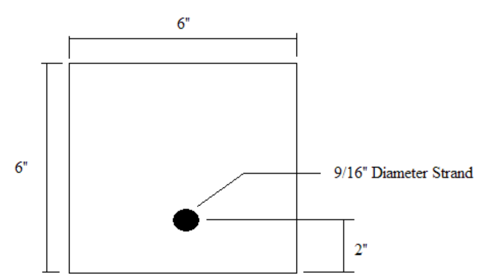


Figure 3-5. Cross section of the transfer length test beams (Trent 2007).

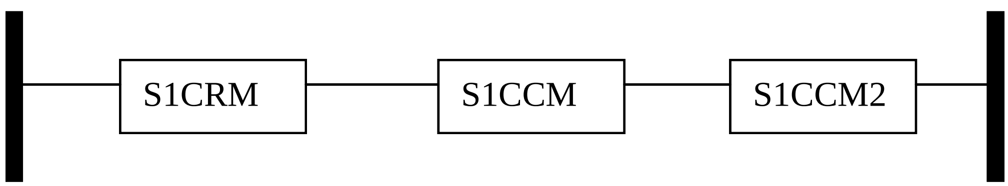


Figure 3-6. Casting bed layout of the transfer length test beams (Trent 2007).

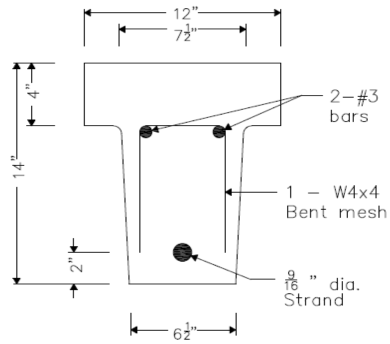


Figure 3-7. Cross-section of the beams used to evaluate development length (Trent 2007).

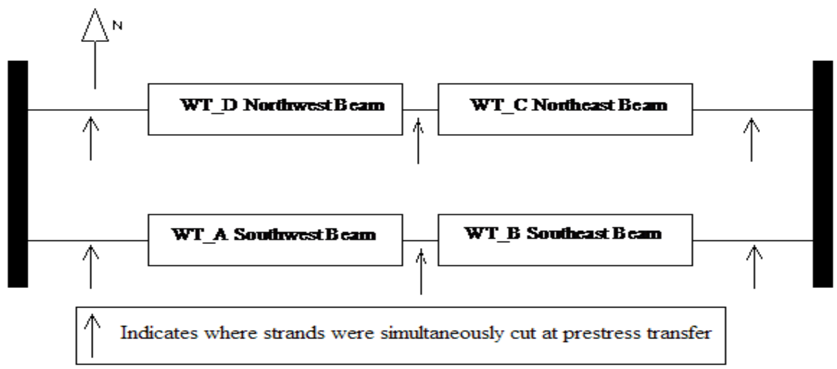


Figure 3-8. Casting bed layout of the development length test members (Trent 2007).

### 3.2.4 Results

*3.2.4.1 Transfer Length*

The 95% ASM method was utilized to determine transfer lengths in this study. The experimental transfer lengths were then compared with the values recommended by the ACI 11.3.4 (50$d_b$ = 28.1 inches) and AASHTO 5.11.4.1 (60$d_b$ = 33.7 inches) specifications. A summary of the transfer length values measured at the time of releasing the strands and after 7 and 28 days of releasing the strands are presented in Table 3-8. As seen in the table, the transfer length values of the strands used in specimen S1CCM2 at 7 days and 28 days were the only values that failed to meet ACI code recommendations, by approximately 17%. It might be worth mentioning that the transfer length of the strands used in the middle specimen (S1CCM) exhibited an increase in the transfer length after 7 days, while that of the edge specimens did not. Researchers attributed the increase to the location of the specimens with respect to the strands cutting point (see Figure 3-6). The two edge members absorbed most of the energy generated by releasing the strands. After the stresses transferred to those members were stable, the prestress force started to transfer gradually to the middle specimen (S1CCM).

Table 3-8. Comparison of Transfer Length Values

| Specimen | Release (in.) | 7 day (in.) | 28 day (in.) |
|---|---|---|---|
| S1CRM | 25 | 28 | 28 |
| S1CCM | 17 | 22 | 26 |
| S1CCM2 | 28 | 34 | 33 |

*3.2.4.2 Development Length*

An iterative testing scheme was conducted to determine the development length in the twelve 24-foot long specimens tested in flexure. The beams' overhang layout, as shown in Figure 3-9, allowed researchers to test the beam ends consecutively. Thus, a total of 24 flexure tests were conducted in the study. The results obtained from the tests indicated 10 bond failures and 14 flexural failures. While testing the members, a linear variable differential transducer (LVDT) was equipped at the end of the strand in order to record slippage distance. An end slip value of 0.01 inches was assumed to be the threshold for defining a bond failure.

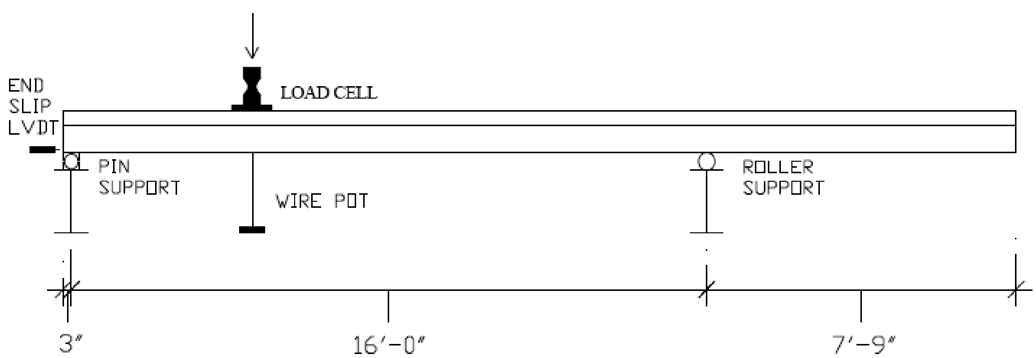


Figure 3-9. Flexural test setup for overhanging beam (Trent 2007).

Table 3-9 summarizes the development length results, where EE stands for "East End" and WE stands for "West End". The development length of the specimens cast with either SCC mix was 75 inches, about 80%-83% of the development length recommended by code provisions. The study failed to determine a development length for the members cast with conventional concrete.

Table 3-9. Development Length Comparison (Trent 2007)

| Member Name | Concrete Type | $L_d$ (in) from ACI Code | $L_d$ (in) Test | Failure Mode | *Test/ Code* |
|---|---|---|---|---|---|
| WT2B WE | S1CCM | 94 | 63 | Stand slip | 0.67 |
| WT2D WE | | | 63 | Stand slip | 0.67 |
| WT2A EE | | | 69 | Stand slip | 0.73 |
| WT2D WE | | | 69 | Stand slip | 0.73 |
| WT2A WE | | | 75 | Flexure | 0.80 |
| WT2B EE | | | 75 | Flexure | 0.80 |
| WT2C EE | | | 99 | Flexure | 1.05 |
| WT2C WE | | | 99 | Flexure | 1.05 |
| WT3A EE | S1CCM2 | 90 | 63 | Stand slip | 0.70 |
| WT3D WE | | | 63 | Stand slip | 0.70 |
| WT3C EE | | | 69 | Stand slip | 0.77 |
| WT3A WE | | | 69 | Flexure | 0.77 |
| WT3C WE | | | 75 | Flexure | 0.83 |
| WT3D EE | | | 75 | Flexure | 0.83 |
| WT3B EE | | | 99 | Flexure | 1.10 |
| WT3B WE | | | 99 | Flexure | 1.10 |

### 3.2.5 Conclusions

1) Two out of the three tested specimens yielded a transfer length value less than the values recommended by the ACI and AASHTO design provisions. The transfer length of the strands used in the third specimen exceeded the ACI recommended value by approximately 17%.
2) The transfer length values of the strands used in the SCC specimens were greater than those of strands in normal concrete specimens.
3) The development length values of strands in SCC specimens were 80%-83% of the development length values recommended by ACI and AASHTO code provisions.

## 3.3 HAQ (2005)

### 3.3.1 Objectives

The Precast/Prestressed Concrete Institute (PCI) sponsored a recent study at Michigan State University (MSU) via the Daniel P. Jenny Research Fellowship. The primary focus of the study was to investigate and characterize the behavior of the bond between SCC and pretensioned strands. Two objectives were identified for the research. The first objective was to study the material properties of three SCC mixes, while the second was to evaluate the transfer and development lengths of prestressing strands in girders cast with the SCC mixes. The study also tested normally-consolidated concrete (NCC) for comparison.

### 3.3.2 Material Properties

Table 3-10 presents information about the constituents of the six NCC and SCC batches used in the MSU study. The poor quality of the first batches of NCC and SCC2 mixes required testing of additional batches. The first and second batches are denoted in Table 3-10 as A and B, respectively. The range of the w/c ratio used in this study was 0.35~0.45. The study considered the variability in SCC mix composition by varying the amount of VMA and HRWR admixtures used in the mixes. For instance, SCC1 was designed with a relatively low w/c ratio and only HRWR, while SSC3 was designed with a relatively high w/c ratio and both VMA and HRWR. The SSC2 mix design properties fell somewhere between those of the SCC1 and SCC3 mixes. Degussa Admixtures Incorporated of Cleveland, OH helped create the concrete mix designs. Type-III Portland cement was used in all mixes. Set retardants were used to account for the relatively long time between casting and delivery to the MSU laboratory. A viscosity modifying admixture was used in the SCC2A, SCC2B and SCC3 mixes, but not the SCC1 or NCC mixes.

Table 3-11 summarizes the test results for the slump spread, VSI, J-ring, and T-box tests conducted on the SCC mixes. Based on the values in the table, the proposed mixes were qualified as SCC. The lowest slump value was 24.5 inches (SCC2B), and the range of VSI values was 0 to 1. Physical tests for NCC mixes were not conducted. The target 28-day compressive strength for all mixes was 7000 psi. Figure 3-10 illustrates that all concrete mixes showed compressive strengths above the target strength and that the SCC compressive strength was greater than that of the NCC. Sixty days later, three SCC mixes (SCC2A, SCC2B, and SCC1) had strengths of approximately 9000 psi, while one mix (SCC3) had strength of 8000 psi.

Table 3-10. Concrete Mix Constituents (Haq 2005)

| **Constituent** | **Weights ($lb/yd^3$)** | | | | | |
|---|---|---|---|---|---|---|
| | NCCA | NCCB | SCC1 | SCC2A | SCC2B | SCC3 |
| Portland Cement Type III | 700 | 700 | 750 | 700 | 700 | 700 |
| Fine Aggregate | 1216 | 1216 | 1627.5 | 1426 | 1426 | 1275 |
| Coarse Aggregate | 1580 | 1580 | 1478.57 | 1380 | 1380 | 1435 |
| Water | 280 | 280 | 262.5 | 280 | 280 | 315 |
| Air-Entrainment | 6% | 6% | 6% | 6% | 6% | 6% |
| W/C - Target | 0.4 | 0.4 | 0.35 | 0.42 | 0.4 | 0.45 |
| Admixtures | oz./cwt | | | | | |
| Air-Entraining Admixture | 3.5 | 3.5 | 1.75 | 0.75 | 1.75 | 3.18 |
| High-Range Water Reducer | 8.06 | 4.06 | 12.93 | 14.59 | 12.03 | 15.37 |
| Viscosity Modifying Admixture | 0 | 0 | 0 | 6.99 | 1.78 | 15.37 |
| Set Retardant | 0 | 52.5 | 70 | 0 | 58.57 | 46.67 |

Table 3-11. Physical Test Results of SCC Mixes (Haq 2005)

| **Mix** | **Avg. Slump Spread (in)** | **VSI** | **J-Ring (in)** | **H2/H1** |
|---|---|---|---|---|
| SCC1 | 27 | 0 | 0.25 | 0.8 |
| SCC2A | 25 | 0.5 | 0.38 | 0.86 |
| SCC2B | 24.5 | 0 | -- | 0.77 |
| SCC3 | 27 | 1 | 0 | 0.69 |

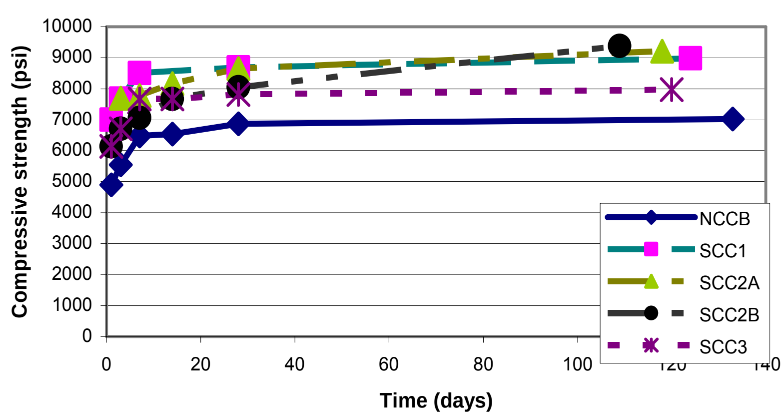


Figure 3-10. Compressive strength of concrete mixes (Haq 2005).

### 3.3.3 Pull-out Test

Performing simple pull-out tests allowed researchers to study the relative bond strength of the concrete mixes. The tests were performed using the same 0.5-inch diameter strands used in the study's large-scale specimens and were conducted 3 and 7 days after casting to evaluate bond strength over time. The peak load values recorded from the pull-out tests were used in Equation 3-1 to compute the relative bond strength of the concrete mixes. In this equation, $D_n$ is the nominal circumference of the strands ($D_n = 4/3\pi d_b$) and $L_b$ is the embedment length, which in this case was 18 inches. Table 3-12 presents a summary of the bond strength values determined from the pull-out tests. Six strands were tested for each concrete mix; three strands were tested at the day of release (3 days after casting) and three strands were tested 7 days after casting. The values in Table 3-12 are averages of these tests. The results of NCCA were not presented since the researcher considered the test for NCCA as a trial test. The last column in the table presents a normalized value for bond strength using the square root of the concrete compressive strength, where the bond strength $U$ is defined in Equation 3-1.

$$U = \frac{P_{\max}}{D_n L_b} \qquad (3\text{-}1)$$

Table 3-12. Average and Relative Bond Strength at Time of Release (Haq 2005)

| | Peak Load (kip) | Bond Strength, U (psi) | Compressive Strength, $f_c'$ (psi) | $U' = U / \sqrt{f_c'}$ |
|---|---|---|---|---|
| NCCB | 30.4 | 806.39 | 5545.12 | 10.83 |
| SCC1 | 16.61 | 440.51 | 7685.02 | 5.02 |
| SCC2A | 26.01 | 689.8 | 7693.25 | 7.86 |
| SCC2B | 19.39 | 514.42 | 6703.8 | 6.28 |
| SCC3 | 20.12 | 533.61 | 6703.8 | 6.52 |

Table 3-12 shows that the bond strength of the SCC mixes was relatively less than that of the NCC mix. When compared to the values recommended by Logan (1997), the maximum pull-out loads recorded during the MSU tests were smaller. Researchers attributed this to the difference in the types of concrete and strands used in both studies. To avoid such inconsistency, another series of pull-out tests were conducted using concrete mix and strands identical to those suggested by Logan (1997). Table 3-13 shows a comparison between the average results of the original MSU tests and the new tests conducted using Logan's recommendations. As shown in Table 3-13, the first slip and peak pull-out loads recorded using the Logan test specifications were satisfactory. However, strands used in the NCCB mix and SCC mixes did not reach the required first slip load. Additionally, strands in the SCC mixes did not reach the required peak load.

Table 3-13. Pull-out Test Results for Logan's (1997) Mix and MSU's SCC and NCC mixes

| Concrete mix | Average First slip load (kips) | Standard deviations (kips) | Average Peak pull-out load (kips) | Standard deviations (kips) |
|---|---|---|---|---|
| Logan | 22.8 | 3.37 | 40.5 | 2.31 |
| SCC | 7.9 | 0.83 | 31.3 | 2.91 |
| NCCB | 12.9 | 1.23 | 37.7 | 1.43 |

**3.3.4 Design of Test Girders**

Large-scale T-beams with the cross-section shown in Figure 3-12 were tested. The concrete specimens included two strands along the bottom line, two No. 4 reinforcing bars along the top line, and lateral stirrups placed every 12 inches throughout the entire span length. Two beams were cast from each concrete mix (NCCA, NCCB, SCC1, SCC2A, SCC2B, and SCC3) resulting in a total of twelve beams. Figure 3-12 presents a schematic of the casting bed layout. The transfer length was evaluated at both ends of each beam. The south end is referred to as Side #1 while the north end is referred to as Side #2. The beams were pretensioned with two 0.5-inch diameter, Grade 270 low-relaxation 7-wire strands. The total length of the girders used for the flexural tests was 38 feet.

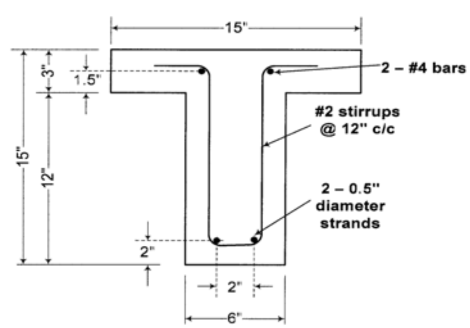


Figure 3-11. Girder cross-section (Haq 2005).

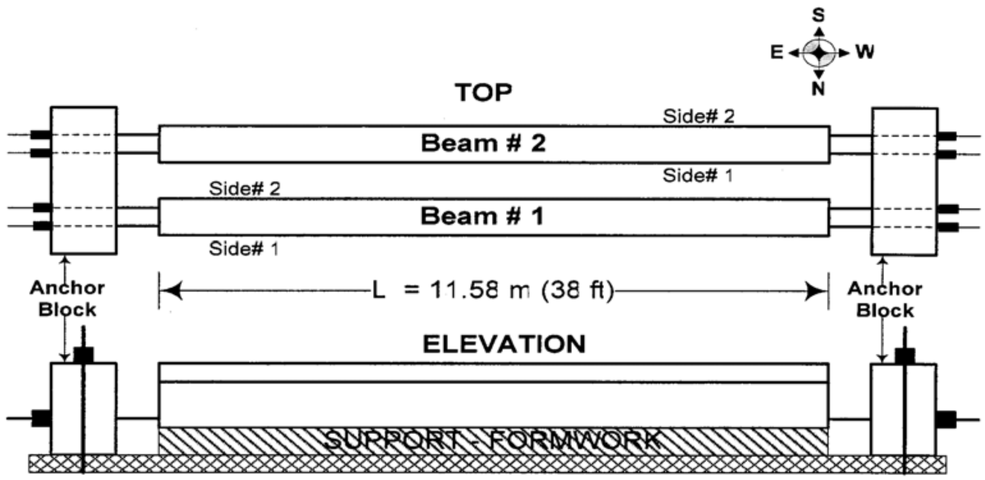


Figure 3-12. Casting bed layout (Haq 2005).

### 3.3.5 Results

*3.3.5.1 Transfer Length*

Researchers used the 95% AMS and end slip methods to evaluate transfer lengths. Table 3-14 summarizes the results of the transfer length tests. The results show that the transfer length values measured in SCC specimens were greater than those measured in NCC specimens. The majority of the transfer length values satisfied the AASHTO code requirement ($60d_b$ = 30 inch) and the ACI flexure design requirement ($d_b f_{se}/3$). However, the transfer length recommended by the ACI code for shear design ($50d_b$ = 25 inch) was violated. Researchers could not conclude that all provisions were satisfied.

Table 3-14. Transfer Length Results (Haq 2005)

| | Test Transfer length(in) | | For Shear Design | | Flexure Design |
|---|---|---|---|---|---|
| Concrete Mix | 95% AMS | Draw-in | ACI (in) | AASHTO (in) | $(f_{se}/3)d_b$ ACI/AASHTO (in) |
| NCCB | 19.65 | 24 | 25 | 30 | 28.74 |
| SCC1 | 29.81 | 31 | 25 | 30 | 28.6 |
| SCC2A | 27 | 20 | 25 | 30 | 30.67 |
| SCC2B | 29.81 | 27.4 | 25 | 30 | 30.13 |
| SCC3 | 30.13 | 30.1 | 25 | 30 | 34.09 |

*3.3.5.2 Development Length*

Iterative flexural tests at each beam end, where the embedment length was varied (see Figure 3-13), were conducted to determine the development length of the girders. Since two beams were cast from each batch of concrete (6 batches), the total number of tests conducted was 24.

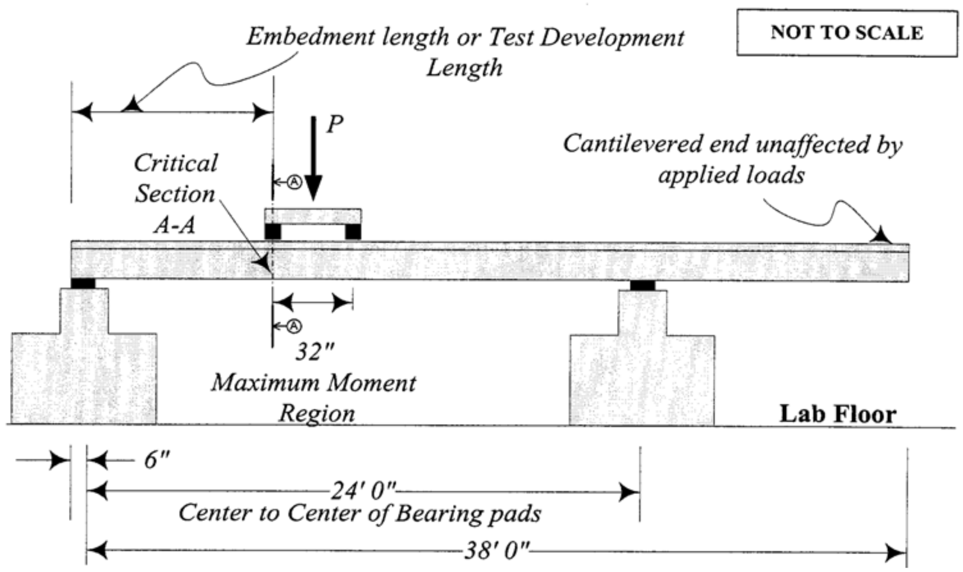


Figure 3-13. Flexural test schematic (Haq 2005).

Table 3-15 summarizes the development length test results, showing that the development length values were generally greater than the AASHTO/ACI code requirement before applying the $K$ (1.6) factor. In the table, $L_{d,\ test}$ represents the actual embedment length of strands in the flexural girders, $L_{d,\ code}$ is the development length recommended by the ACI/AASHTO code provisions, and $M_{n,\ test}$ and $M_{n,\ code}$ are the experimental and theoretical moment capacity values of the tested girders, respectively. An $L_{d,\ test}$ / $L_{d,\ code}$ value less than 1.0 indicates that the ACI/AASHTO code provisions are satisfied. The following types of failures were observed during the tests: flexural failure (F), shear-slip (S), and flexural slip (FS). The specimen ID is denoted using the following format: concrete mix-beam number-beam end. The “A” beam end represents the west end of the beam while the “B” beam end represents the east end.

Table 3-15. Development Length Test Results (Haq 2005)

| Specimen ID | Development Length (in) | $L_{d,\ test}$ / $L_{d,\ code}$ | $M_{n,\ test}$ / $M_{n,\ code}$ | Failure Type |
|---|---|---|---|---|
| SCC1-1-A | 72.38 | 1.07 | 1.057 | FS |
| SCC1-1-B | 133.75 | 2.03 | 1.137 | F |
| SCC1-2-A | 122 | 1.79 | 1.121 | F |
| SCC1-2-B | 118.5 | 1.74 | 1.217 | F |
| SCC2A-1-A | 70.5 | 1.09 | 1.105 | FS |
| SCC2A-1-B | 64.5 | 1 | 0.966 | S |
| SCC2A-2-A | 80 | 1.27 | 1.127 | FS |
| SCC2A-2-B | 86.75 | 1.37 | 1.137 | FS |
| SCC2B-1-A | 70.5 | 1.21 | 1.007 | FS |
| SCC2B-1-B | 102.75 | 1.76 | 1.14 | FS |
| SCC2B-2-A | 126.75 | 1.78 | 1.101 | F |
| SCC2B-2-B | 124.5 | 1.75 | 1.187 | F |
| SCC3-1-A | 58 | 1.06 | 0.953 | S |
| SCC3-1-B | 97.75 | 1.79 | 1.09 | FS |
| SCC3-2-A | 106.5 | 1.8 | 1.1 | FS |
| SCC3-2-B | 103 | 1.74 | 1.132 | FS |
| NCCA-1-A | 76 | 1.06 | 0.952 | FS |
| NCCA-1-B | 122.7 | 1.71 | 1.158 | FS |
| NCCA-2-A | 111 | 1.55 | 1.201 | FS |
| NCCA-2-B | 60 | 0.84 | 0.907 | S |
| NCCB-1-A | 63.75 | 1.06 | 1.036 | F |
| NCCB-1-B | 64 | 1.07 | 1.049 | F |
| NCCB-2-A | 103.5 | 1.31 | 1.145 | F |
| NCCB-2-B | 93.5 | 1.22 | 1.137 | F |

**3.3.6 Conclusions**

1) Pull-out tests revealed that the bond strength of SCC was less than that of NCC.
2) The transfer length values of strands in SCC girders were greater than in NCC girders. The values were near those recommended by the ACI and AASHTO codes.
3) The development length values for SCC specimens did not meet code provisions.

## 3.4 GIRGIS AND TUAN (2004)

### 3.4.1 Objectives

The Nebraska Department of Roads sponsored a research project to investigate the bond strength of SCC and the transfer length of prestressing strands in SCC girders. The study was conducted at the University of Nebraska, Lincoln.

### 3.4.2 Material Properties

The study used two SCC mixes (Mix #1 and Mix #2) and one conventional concrete mix (Mix #3). One important advantage of this project was that the three concrete mixes were used in three Nebraska bridge projects. Mix #1 was used for the Oak Creek Bridge in Lancaster, NE, Mix #2 was used in the Clarks South Bridge in Merrick, NE, and Mix #3 was used in the North Broadway Bridge in Sedgwick, KS (Girgis and Tuan, 2004). Table 3-16 shows the constituents of the three concrete mixes, all of which contained Type III Portland cement. The w/c ratios of Mix #1, Mix #2 and Mix #3 were 0.31, 0.40 and 0.40, respectively. The proportions of the three admixtures varied. Neither fly ash nor any VMA was added to the conventional concrete mix (Mix #3).

Table 3-16. SCC and conventional concrete mixtures (Girgis and Tuan, 2004)

| Constituents | Mix #1 | Mix #2 | Mix #3 |
|---|---|---|---|
| Portland Cement Type III | 800 lbs | 632 lbs | 732 lbs |
| Fly Ash, Class C | 150 lbs | 100 lbs | ------ |
| Water | 35 gal | 35 gal | 35 gal |
| W/C | 0.31 | 0.40 | 0.40 |
| ½" limestone (SSD) | 1282 lbs | 1311 lbs | 1350 lbs |
| C33 sand (SSD) | 1417 lbs | 1449 lbs | 1460 lbs |
| Air Entraining Admixture | 2~5% | 6~8% | 6~8% |
| Retarding and water reducing | 0~5 oz/yard | 0~5 oz/yard | 0~5 oz/yard |
| HRWR, 5~40% of water reducing | 2~14 oz/yard | 2~14 oz/yard | 4~9 oz/yard |
| HRWR, 5~15% of water reducing | 4~8 oz/yard | ------- | 4~8 oz/yard |
| Viscosity Modifier Admixture | 2~10 oz/yard | 2~10 oz/yard | -------- |

Figure 3-14 shows the compressive strengths of the concretes at various ages. At early ages, Mix #3 had the greatest compressive strength, but at 28 days Mix #1 had greater compressive strength compared to the other two mixes. The flowability test results are given in Table 3-17. Flow cone tests were conducted for the SCC mixes, while the standard slump flow test was used for the conventional concrete mix.

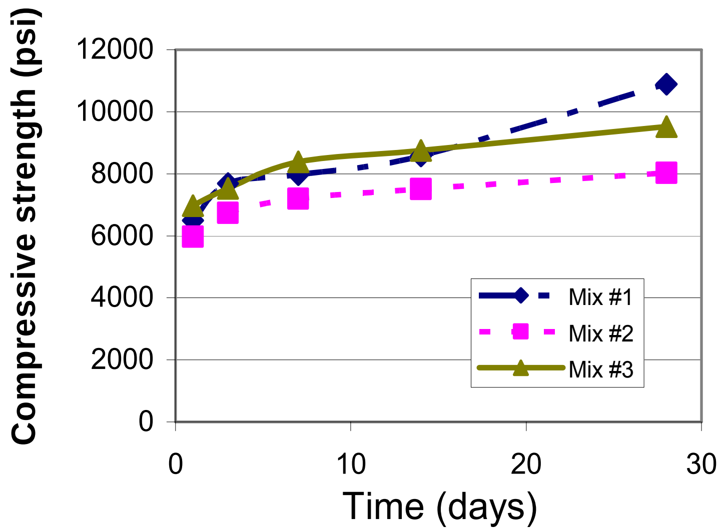


Figure 3-14. Concrete compressive strength over time for the 2004 Girgis & Tuan study.

Table 3-17. Flowability test results (Girgis and Tuan, 2004)

| Concrete Mix | Flowablity Test | Results |
|---|---|---|
| Mix #1 | Flow cone test | 30 in. diameter |
| Mix #2 | Flow cone test | 30 in. diameter |
| Mix #3 | Slump Test | 10 in. slump |

### 3.4.3 Pull-out Test

A series of pull-out tests were utilized to investigate the bond capacity of the three concrete mixes, using 0.6-inch diameter smooth strands instead of the strands suggested by Logan (1997). A total of six specimens were cast for the tests, two for each mix. The first specimen had an 18 inch embedment length, the recommended standard (Moustafa 1974). The second specimen had several embedment lengths (16 inches, 18 inches, and 20 inches). Figure 3-15 shows the results of the pull-out tests. At an early age, Mix #1, Mix #2 and Mix #3 had maximum pull-out forces of 43.4 kips, 54.15 kips, and 48 kips, respectively.

At 28 days, Mix #2 and Mix #3 were tested again and their maximum pull-out forces were 65.68 kips and 63.14 kips, respectively. To compare the results to the recommended value of 36 kips, researchers adopted a multiplier based on the ratio of strand diameters, 0.6 inches and 0.5 inches, resulting in a benchmark value of 43.2 kips. All pull-out loads were above the benchmark value. Through testing the second set of specimens, researchers studied the effect embedment length had on bond strength; however, the results obtained were inconsistent.

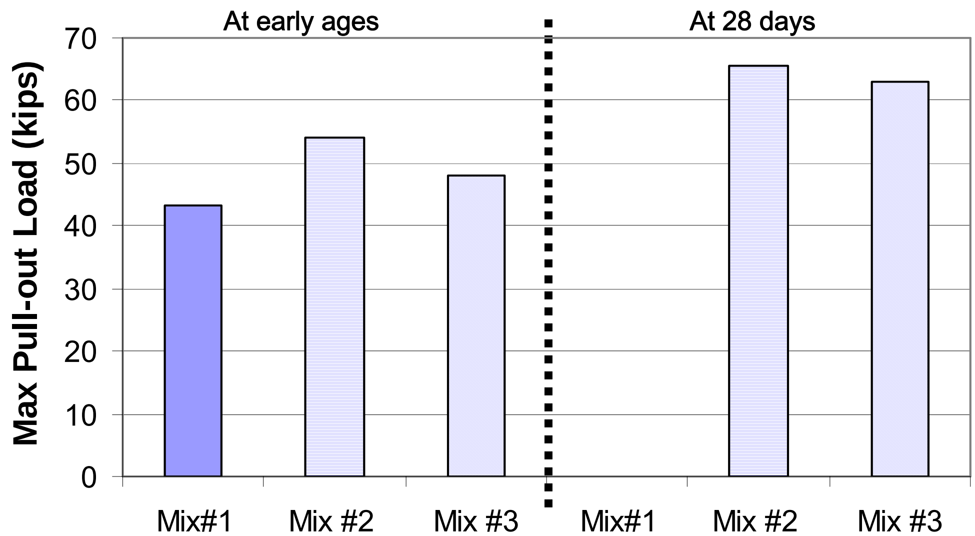


Figure 3-15. Maximum pull-out loads with 18 in. embedment length (Girgis and Tuan, 2004).

### 3.4.4 Design of Test Girders

The study used three types of girders to evaluate transfer length. For Mix #1, used in the Oak Creek Bridge, researchers tested the NU1100 I-section depicted in Figure 3-16. The total length of the girders was 72’-6”. For Mix #2, used in the Clarks South Bridge girders, researchers tested the NU900 I-section. The span length of this girder was 90’-2”. Finally, for Mix #3, used in the North Broadway Bridge girders, researchers tested the NU1350 I-section. The length of the girders in this study was 124’. The web width was 5.9” for all sections, but the heights of the sections were different. The NU1350 I-girder section, which was made of the conventional concrete, had the largest moment of inertia.

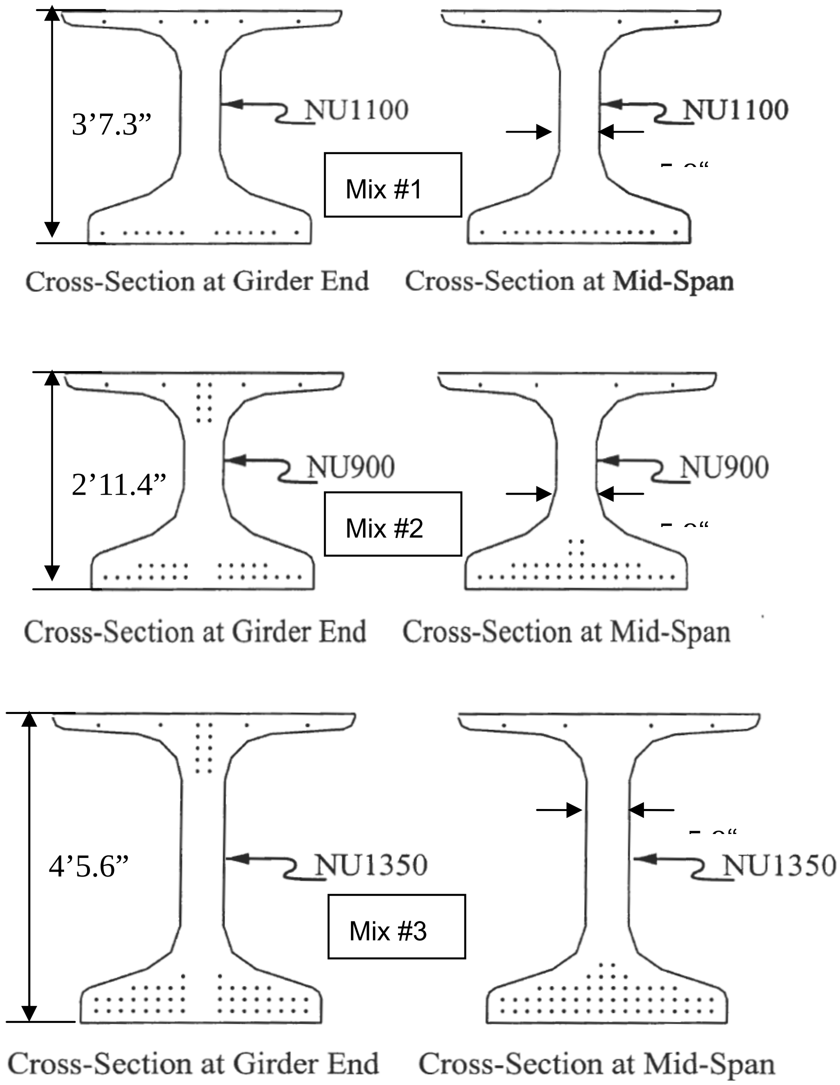


Figure 3-16. Cross-section of the girders used in the study (Girgis and Tuan, 2004).

### 3.4.5 Results

*3.4.5.1 Transfer Length*

The 95% AMS method was used to evaluate the transfer length of the girders cast with Mixes #1, #2 and #3. Data was acquired from both sides and both ends of the girders at the bottom flanges. Figure 3-17 shows an example of the graph used in evaluating the transfer length of the girder cast with Mix #1. The data points shown in the figure were recorded from the south side of the girder. The west side showed a 40 inch transfer length, while the east side had a 30 inch transfer length. Additionally, the graph from the north side showed a 44 inch and a 30 inch transfer length at the west and east sides, respectively.

Thus, the strands used in the girders cast with Mix #1 had a 36 inch average transfer length. Table 3-18 contains the transfer length results for all three girders. Based on shear design requirements, the maximum transfer length recommended by the ACI and AASHTO codes were 30 inches and 36 inches, respectively. A graphical comparison between the recommended values and the experimental values is presented in Figure 3-18, which shows that the transfer length values of the SCC mixes either matched or exceeded the values recommended by the codes. However, the transfer length of the strands used in the conventional concrete mix was less than the value recommended by the codes.

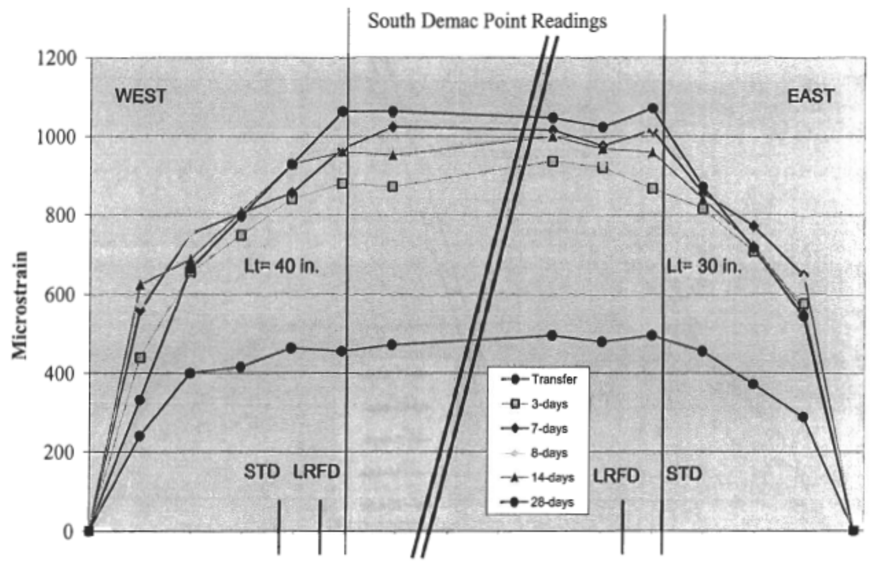


Figure 3-17. South side concrete strain along the length of the girder cast with Mix #1 (Girgis and Tuan, 2004).

Table 3-18. Transfer Length Results (Girgis and Tuan, 2004).

| | **Mix #1** | | **Mix #2** | | **Mix #3** | |
|---|---|---|---|---|---|---|
| | **North Side** | **South Side** | **North Side** | **South Side** | **North Side** | **South Side** |
| West Side | 44 | 40 | 40 | 48 | | 20 |
| East Side | 30 | 30 | 42 | 42 | 20 | 20 |
| Avg. Transfer Length (in.) | 36 | | 43 | | 20 | |

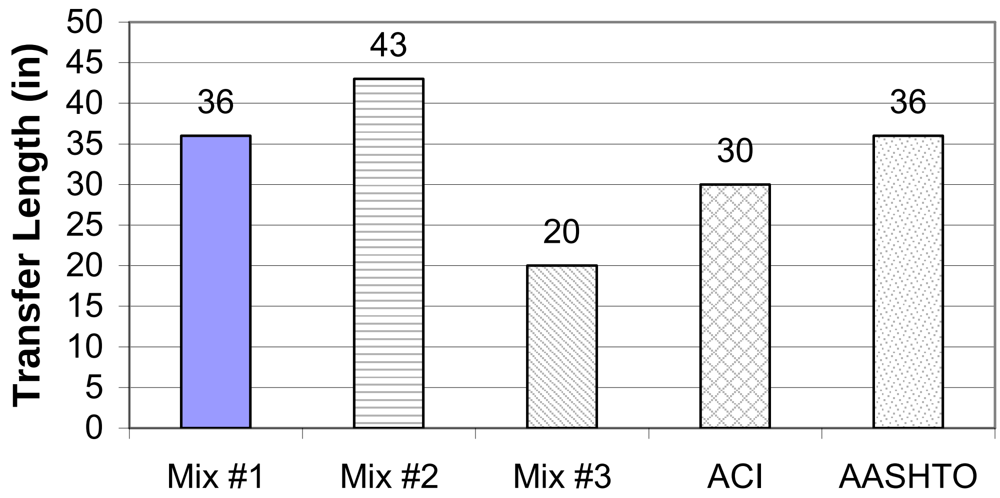


Figure 3-18. Average experimental and recommended transfer lengths.

### 3.4.6 Conclusions

1) Pull-out tests conducted with the proposed SCC mixes and the 0.6-inch diameter strands showed reasonable bond strengths. Thus, the 0.6-inch diameter strands were considered suitable for this study.
2) The transfer lengths of the strands used in the SCC girders were greater than those of the strands used in conventional concrete girders by approximately 98%.
3) The transfer lengths of the strands used in SCC girders exceeded the ACI and AASHTO code recommended values. However, the transfer length of strands used in the conventional concrete girder satisfied the code provisions.

## 3.5 HAMILTON AND LABONTE (2005)

### 3.5.1 Objectives

The Florida Department of Transportation (FDOT) sponsored this research project to compare the structural performance of AASHTO Type-II bridge girders cast with SCC to those cast with conventional concrete. The study was conducted at the University of Florida at Gainesville. One of the primary tasks of the study was to determine the transfer length of AASHTO Type II bridge girders cast with the proposed SCC mixes.

### 3.5.2 Material Properties

Two SCC mixes and two conventional or standard concrete mixes were proposed as trial mixes for this project. Only one SCC mix and one standard (STD) concrete mix were selected for the beam tests. Table 3-19 shows the constituents of the beam test mixes and Table 3-20 shows the mixes' plastic properties. The target compressive strength chosen for the mixes, all of which were approved by FDOT, was 8,500 psi. The FDOT State Materials Office conducted the compressive strength tests for the concrete various ages. A summary of the test results is shown in Figure 3-19. As shown, the SCC had a slightly lower strength at early ages than the standard concrete. After 50 days, however, both concretes had compressive strengths above 8,500 psi.

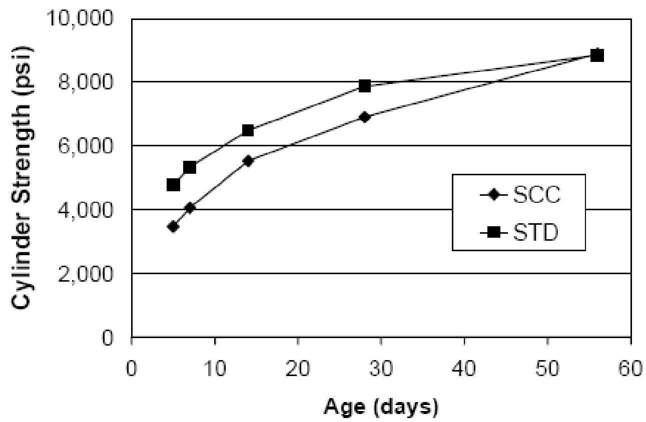


Figure 3-19. Average concrete compressive strength over time (Larson et al. 2006).

Table 3-19. Concrete Mix Designs (Hamilton and Labonte, 2005)

| Constituents | Pair B | |
|---|---|---|
| | STD (oz/cy) | SCC (oz/cy) |
| Cement : Lehigh Type I/II | 752 | 752 |
| Fly ash : ISG Class F | 168 | 168 |
| Coarse Aggregate: Tarmac #67 | 1307 | 1307 |
| Fine Aggregate: Florida Rock silica sand | 1414 | 1414 |
| Water | 258 | 258 |
| Admixtures | (oz/cy) | |
| Air entraining agent : MBVR –S | 1.8 | 1.8 |
| Set retarding water reducer : Pozzolith 100 XR | 13.8 | 13.8 |
| High range water reducer : Glenium 3200 HES | 27.6 | 64.4 |

Table 3-20. Summary of Tests Results (Hamilton and Labonte, 2005)

| Test | Standard (STD) | SCC |
|---|---|---|
| Slump | 7.2 in | N/A |
| Slump flow | N/A | 27.2 in |
| Slump flow T-20 | N/A | 1.3 sec |
| J-ring spread | N/A | 28.0 in |
| J-ring T-20 | N/A | 1.3 sec |
| J-ring H1/H2 | N/A | 5.75 in/ 5.5 in |
| L-box H1/H2 | N/A | 4.0 in/ 4.0 in |
| L-box T-200 | N/A | 0.5 sec |
| L-box T-400 | N/A | 1.0 sec |
| U-box H1/H2 | N/A | 13.75 in/ 14.0 in |
| V-funnel flow | N/A | 2.0sec |

### 3.5.3 Design of Test Girders

Six 42-foot long girders were cast in a single bed. A schematic plan of the six girders in the casting bed is depicted in Figure 3-20. The cross section of the AASHTO Type-II girders used in the study is shown in Figure 3-21. To err on the conservative side, researchers used the cutting method to release the strands instead of releasing the strands gradually. The resulting transfer length would expectedly be slightly longer than if the strands were released gradually (Russell and Burns, 1997). Figure 3-20 shows the points at which the strands were cut, transferring the stress rapidly at one end of the beams and more gradually at the other. Girders STDF2 and SCCF1 were used to evaluate transfer lengths.

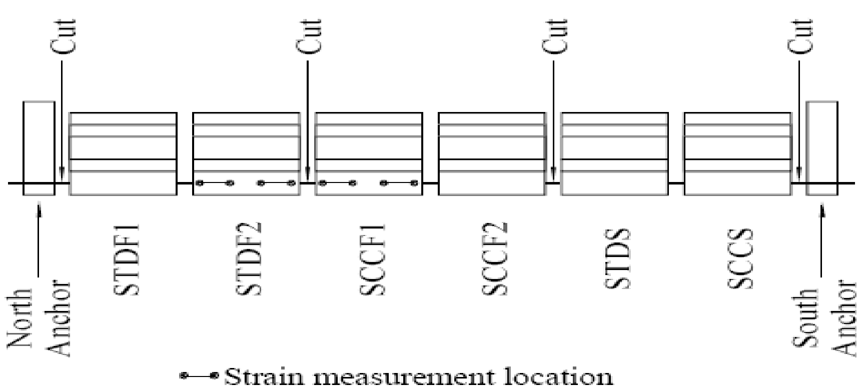


Figure 3-20. Casting bed layout and cut locations (Hamilton and Labonte, 2005).

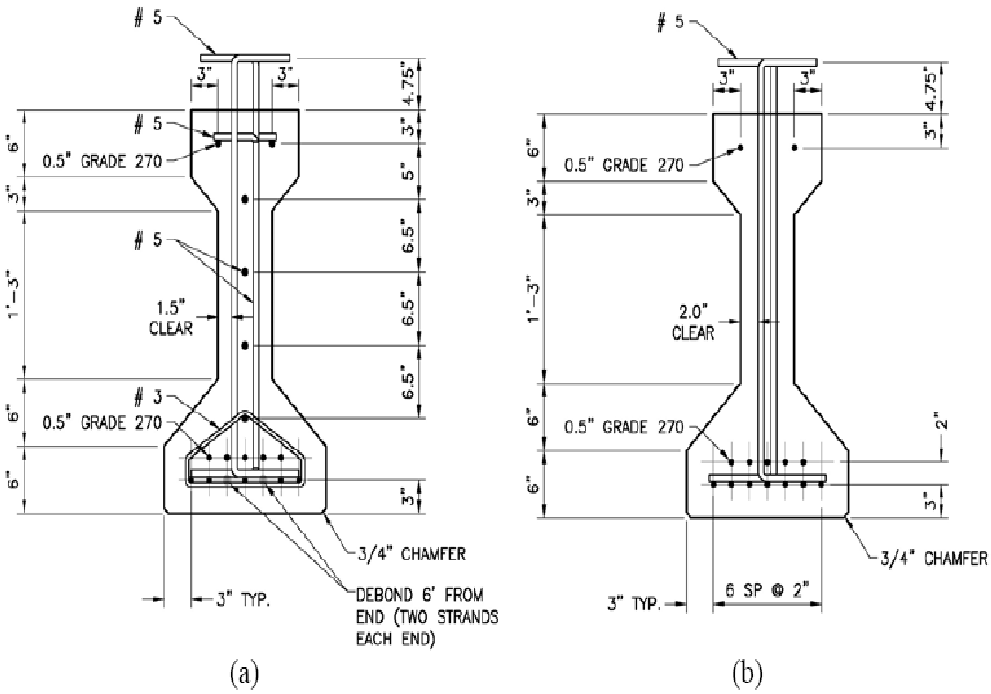


Figure 3-21. AASHTO Type II cross-section at the (a) end and (b) middle of a girder (Hamilton and Labonte, 2005).

### 3.5.4 Results

Strain gauges were placed along the bottom edge of girders STDF2 and SCCF1. The 95% AMS method was used to evaluate the transfer lengths, calculated for each side of the girders. A summary of the transfer length results is shown in Table 3-21. As expected, the transfer lengths measured at the cutting ends were longer than at the free ends. Both the AASHTO and ACI code provisions, where the recommended transfer lengths were 30 inches and 25 inches, respectively, were satisfied. The results revealed no significant difference between the transfer lengths of strands in the SCC girders and strands in the standard concrete girders. However, the relatively small number of strain gauges used to capture the varying strains could have yielded misleading data.

Table 3-21. Transfer Length Test Results (Hamilton and Labonte, 2005)

| Girder | Location | End Type | Transfer Length (in.) |
|---|---|---|---|
| STDF2 | North | Free | 12.1 |
| STDF2 | South | Cutting | 15.5 |
| SCCF1 | North | Cutting | 15 |
| SCCF1 | South | Free | 13 |

### 3.5.5 Conclusions

1) SCC compressive strength was comparable to that of conventional concrete.
2) No significant differences were observed between the transfer length of strands used in SCC and conventional concrete girders.
3) The transfer length values measured were affected by the method used in releasing the strands. The transfer length at the cut end was 28% longer than at the free end for standard concrete, and 15% longer than at the free end for SCC.
4) The AASHTO and ACI transfer length values recommended for shear design were conservative for SCC and standard concrete by approximately 100% (i.e. twice the length obtained from the experiments).

## 3.6 NAITO et al. (2005 & 2006)

### 3.6.1 Objectives

Conducted at Lehigh University and sponsored by the Pennsylvania Department of Transportation (PennDOT), this project had three main objectives: (1) Investigate the material characteristics of SCC and conventional high early strength concrete (HESC), (2) Evaluate the transfer length, maximum moment, and maximum shear force of full-scale bulb tee girders cast with SCC and HESC, and (3) Investigate the characteristics of bond between concrete and prestressing strands. Section 3.6 presents only the results of the pull-out and transfer length tests.

### 3.6.2 Material Properties

The properties of the study's SCC and HESC mixes are presented in Table 3-22. The PennDOT high early strength specification was adopted in proportioning the HESC mixes (CPDOT 2004). The coarse aggregate size in the HESC mix ranged from 0.75 inches to 0.375 inches (AASHTO No. 67 to No. 8). The SCC mix used only 0.375-inch coarse aggregate. Both mixes used natural silica sand as a fine aggregate. The mixes contained several admixtures including an HRWR (ASTM C 494 Type FHRWR: Glenium 3030NS), an air-entraining admixture (ASTM C 260 neutralized vinsol resin: MB-VR standard), a retarding admixture (ASTM C 494 Type B: Pozzolith 100 XR), and a commercially available VMA (Rheomac VMA 358). The w/c ratio of the HESC and the SCC was 0.34 and 0.32, respectively. The level of the entrained air was 5.4% for the HESC and 5% for the SCC.

Table 3-22. Concrete Mix Designs (Naito et al. 2005)

| Material Type | HESC | SCC |
|---|---|---|
| Total Cement, *lb/yd*$^3$ | 750 | 849 |
| Slag Cement, % | 34 | 25 |
| Fine Aggregate SSD, *lb/yd*$^3$ | 1172 | 1283 |
| Coarse Aggregate #67 SSD, *lb/yd*$^3$ | 1383 | 0 |
| Coarse Aggregate #8 SSD, *lb/yd*$^3$ | 552 | 1651 |
| Water-Cement Ratio | 0.34 | 0.32 |
| HWRW Admixture, *oz/yd*$^3$ | 60.0 | 136.2 |
| Retarding Admixture, *oz/yd*$^3$ | 4.0 | 0 |
| Air-Entraining Admixture, *oz/yd*$^3$ | 2.4 | 2.0 |
| VMA, *oz/yd*$^3$ | 0 | 16.0 |
| Coarse Aggregate Volume, % | 39 | 34 |
| Density, *lb/yd*$^3$ | 149.8 | 148.8 |
| Air Content, % | 5.4 | 5 |
| Slump/Spread, in. (mm) | 6.3 | 21.3 |

A slump flow test was performed on the HESC mix and an inverted slump cone, VSI, and J-ring tests were performed on the SCC mix. The test results are shown in Table 3-23. The inverted cone spread of the SCC was 21.3 in., slightly below the 22 in. value typically recommended for SCC. However, the researchers decided that the difference was negligible.

The target compressive strengths at 24 hours and 28 days were 6800 psi and 8000 psi, respectively. Table 3-24 shows the actual compressive strengths at various ages. The compressive strengths of both mixes at 24 hours met the target strength. The concretes did not meet, but came near, the 28-day target strength. After 28 days, the compressive strength of the SCC was slightly higher than that of the HESC.

Table 3-23. Physical Test Results (Naito et al. 2005)

| Test | HESC | SCC |
|---|---|---|
| Slump (in.) | 6.3 | N/A |
| Inverted Cone Spread (in.) | N/A | 21.3 |
| Spread through J-Ring (in.) | N/A | 19.9 |
| VSI | N/A | 0~0.5 |

Table 3-24. Compressive Strength over Time (Naito et al. 2005)

| Days | HESC (psi) | SCC (psi) |
|---|---|---|
| 1 | 6809 | 8232 |
| 3 | 6802 | 7809 |
| 7 | 7568 | 8724 |
| 14 | 7520 | 7980 |
| 28 | 7366 | 8276 |
| 38 | - | 9166 |
| 56 | 7155 | 8634 |
| 61 | 7136 | - |
| 79 | 7580 | - |
| 89 | - | 9842 |
| 101 | 8950 | 10427 |

### 3.6.3 Pull-out Test

Pull-out tests were conducted using 0.5-inch diameter strands and a concrete mix similar to that which was suggested by Logan (1997). A total of 36 strands were tested. At the test date, the concrete compressive strength was 4,000 psi. Although the average maximum pull-out force was 31.5 kips, below the recommended value (36 kips), researchers decided it was acceptable for the study based on past engineering experience.

### 3.6.4 Design of Test Girders

Naito et al. tested bulb tee section girders with the cross-sectional geometry depicted in Figure 3-22 and 30'-1/8" length. The Mid Atlantic States Prestressed Concrete Committee for Economic Fabrication (PCEF), part of the PennDOT standard, defined the girder geometry.

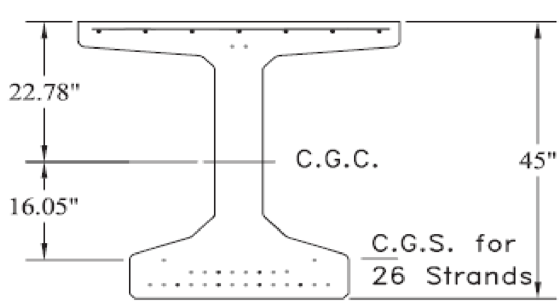


Figure 3-22. Cross-section of the bulb tee (Naito et al. 2005).

### 3.6.5 Results

Researchers used the 95% AMS method to evaluate the transfer length of strands in prestressed girders cast with SCC and HESC. A total of four beams were cast, two for each mix type. Strain gauges were placed at one end of each beam. Figure 3-23 shows the strain values recorded at various distances from the girder end. The transfer length values for strands in the HESC and SCC girders were 15.8 inches and 15.7 inches, respectively, less than the values recommended by the ACI (25 inches) and AASHTO (30 inches).

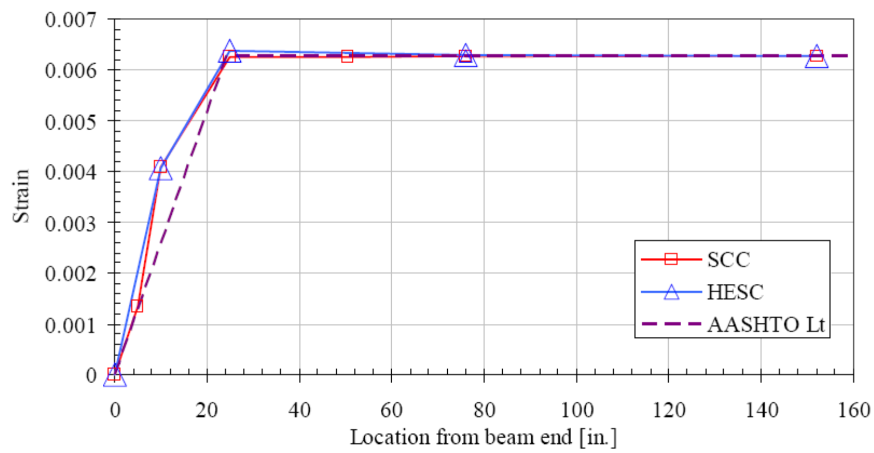


Figure 3-23. Strain at distances away from the girder end (Naito et al. 2005).

### 3.6.6 Conclusions

1) The maximum average pull-out load recorded for the strands was 31.5 kips, less than the 36-kip value recommended by Logan. However, the strands were still accepted and used in the study.
2) The transfer length values recorded from both the SCC and HESC girders were satisfactory according to the ACI and AASHTO code provisions.
3) No significant differences were observed between the transfer length of the strands used in the SCC and HESC girders.

## 3.7 RUIZ et al. (2006)

### 3.7.1 Objectives

Conducted at the Engineering Research Center of the University of Arkansas at Fayetteville, this project evaluated the transfer lengths of prestressing strands in SCC beams and compared them to those of strands in conventional concrete. The transfer lengths were then compared to ACI and AASHTO recommended values.

### 3.7.2 Material Properties

A relatively high strength SCC was used in this project. The target strength of the SCC mix was 7 ksi at the time of prestress release and 12 ksi at 28 days. The maximum size of the coarse aggregate was 0.5 inches. Both HRWR and VMA admixtures were included in the concrete mix. Table 3-25 shows the constituents of the SCC mix.

Table 3-25 SCC Mix Design (Ruiz et al. 2006)

| Materials | SCCI |
|---|---|
| Cement (*lb/yd*$^3$) | 950 |
| Coarse Aggregate (*lb/yd*$^3$) | 1350 |
| Fine Aggregate (*lb/yd*$^3$) | 1474 |
| Water (*lb/yd*$^3$) | 285 |
| W/C | 0.3 |
| ADVA 170 (*oz/cwt*) | 11 |
| ADVA 555 (*oz/cwt*) | 2 |

### 3.7.3 Design of Test Beams

Twelve 18'-long prestressed beams were cast; half used SCC and half used conventional concrete. The beams had a 6.5" x 12" rectangular section with two 0.6-inch diameter strands (Figure 3-24). Two No. 6 Grade 60 reinforcing bars were placed in the compression zone of the beams. The beams were reinforced laterally by a 0.25" stirrups placed every 6 inches. Before testing the beams, a series of tests were conducted to identify the plasticity and mechanical properties of the SCC mix. Table 3-26 summarizes the results of 14 tests on the SCC mix.

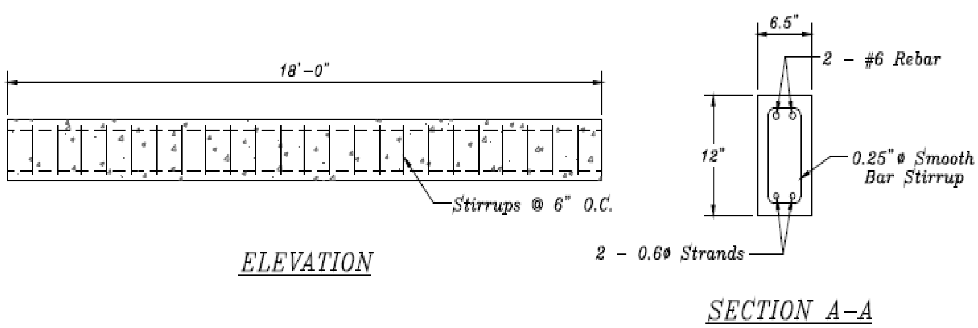


Figure 3-24. Elevation and cross-section of the tested beams (Ruiz et al. 2006).

Table 3-26. Plasticity and Mechanical Test Results for SCC Mixes (Ruiz et al. 2006)

| Specimens | Test Type | | | Concrete Strength (ksi) | |
|---|---|---|---|---|---|
| | Slump Flow (in.) | T20 (sec) | VSI | Initial Day | 7 Day |
| SCCI-1 | 26 | 4 | 1 | 8.52 | 12.48 |
| SCCI-1 | 26 | 3 | 1 | | |
| SCCI-1 | 27 | 4 | 0.5 | | |
| SCCI-2 | 30 | 3 | 0.5 | 8.7 | 13.29 |
| SCCI-2 | 25 | 4 | 1 | | |
| SCCI-2 | 24 | 3.4 | 1 | | |
| SCCI-3 | 27.5 | 2.75 | 1.5 | 7.22 | 10.68 |
| SCCI-3 | 30 | 2 | 1.5 | | |
| SCCI-4 | 30 | 2 | 1.5 | 5.9 | 9.59 |
| SCCI-4 | 28 | 2.75 | 0.5 | | |
| SCCI-5 | 30 | 2.63 | -- | 7.43 | 9.71 |
| SCCI-5 | 31 | 2.15 | -- | | |
| SCCI-6 | 28.5 | 3.28 | -- | 7.33 | 10.29 |
| SCCI-6 | 27 | 2.75 | -- | | |
| Average | 27.9 | 2.98 | 1 | 7.52 | 11.00 |

### 3.7.4 Results

Detachable Mechanical (Demec) strain gauges were placed along the beams to evaluate transfer length using the 95% AMS method. To confirm the results from the strain gauges, vibrating wires (VW) were placed between the strands at each end of the beam. Researchers compared the recorded strains of specimen Number's 3, 5, and 6 using the Demec point strain gauges and vibrating wires. Table 3-27 presents the strains recorded 24 inches from each beam end. The values showed that a 2~10% difference between Demec and VW results for specimens 5 and 6. A 20% difference, which researchers attributed to poorly equipped vibrating wires, was observed in specimen No. 3.

Table 3-27. Strain Comparison from Demec Gauges and Vibrating Wires (Ruiz et al. 2006)

| Specimen | Live End | | | Dead End | | |
|---|---|---|---|---|---|---|
| | Demec ( $\mu\varepsilon$ ) | VW( $\mu\varepsilon$ ) | Demec/VW | Demec( $\mu\varepsilon$ ) | VW( $\mu\varepsilon$ ) | Demec/VW |
| SCCI-3 | 592 | 733 | 0.81 | 543 | 691 | 0.79 |
| SCCI-5 | 619 | 608 | 1.02 | 592 | 536 | 1.10 |
| SCCI-6 | 576 | 598 | 0.96 | 640 | 605 | 1.06 |

The transfer lengths were evaluated using the strain profiles at the time of release, as well as at 3 days and 7 days after releasing the strands. The transfer lengths did not vary significantly after three 3 days of prestress release. Figure 3-25 shows the strain profile for specimen No. 1 at the live end. The figure shows a transfer length of approximately 21 inches at release, 3 days and 7 days.

A summary of the transfer length results at 7 days is shown in Table 3-28, comparing the ACI and AASHTO code requirements to experimental results. For 0.6-inch diameter strands, the ACI and AASHTO code provisions recommend a maximum transfer length of 30 inches and 36 inches, respectively. On average, Equation 2-1 would result in a maximum recommended transfer length of 36.7 inches. The Table 3-28 indicates that the measured average transfer lengths at both ends of the beams were significantly smaller than the values recommended by the codes.

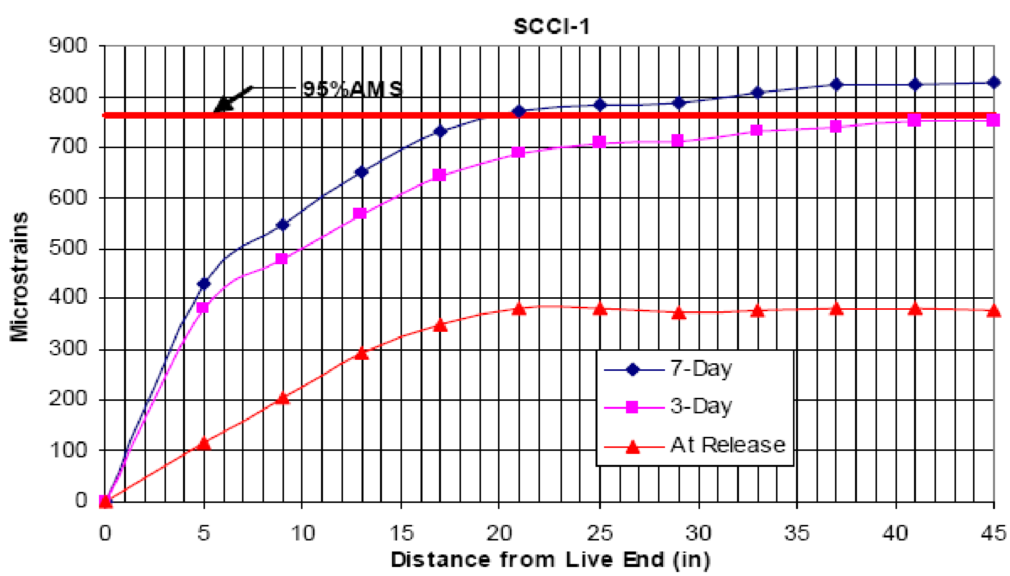


Figure 3-25. Strain profiles along the end zone of specimen No. 1 (Ruiz et al. 2006).

Table 3-28. Transfer Lengths of the Beams at 7 Days after Releasing Strands

| Specimen | Measured Transfer Length (in.) | | Code Provisions (in.) | | |
|---|---|---|---|---|---|
| | Live End | Dead End | ACI 11.3.4 | AASHTO 5.11.4.1 | ACI-318 12.9.1 Equation 2.1 |
| SCC-1 | 21 | 26 | 30 | 36 | 35.9 |
| SCC-2 | 20 | 14 | | | 35.8 |
| SCC-3 | 17.5 | 19 | | | 37.5 |
| SCC-4 | 25.5 | 20.5 | | | 36.8 |
| SCC-5 | 21 | 21 | | | 37.1 |
| SCC-6 | 17.5 | 17.7 | | | 37.2 |
| Average | 20.3 | 19.7 | 30 | 36 | 36.7 |

### 3.7.5 Conclusions

1) The compressive strength seemed to have minor impact on the transfer length of prestressing strands, based on the comparable transfer length values observed at all three measurement ages. Although the compressive strengths at these times were different, the transfer lengths measured were almost identical.
2) The measured transfer length in all six beams cast with SCC satisfied the ACI code requirement by a margin of 33% and the AASHTO code requirement b a margin of 44%.

# CHAPTER 4 CURRENT PRACTICE IN ILLINOIS

To provide IDOT with an accurate assessment of the applicability of the conclusions derived from previous studies to IDOT's prestressed girders, the SCC mix designs used in these studies should first be compared to those typically produced in the State of Illinois. As noted in Section 2.1, the engineering properties of SCC vary significantly among fabricators due to variations in the material constituents each includes in its mix design. SCC mixes produced in Illinois must adhere to provisions developed by IDOT, resulting in mixes which differ from those used in previous studies in terms of materials and performance. This chapter discusses current Illinois SCC mix requirements, presents two Illinois mixes which meet these requirements, and compares these mixes to those used in previous studies.

## 4.1 IDOT PROVISION FOR SELF-CONSOLIDATING CONCRETE

According to *Self-Consolidating Concrete for Precast Products*, an IDOT Bureau of Design & Environment provision revised in 2007 to fit with IDOT's Standard Specifications for Road and Bridge Construction, the following mix design criteria shall be met:

(a) The minimum cement factor shall be according to Article 1020.04 of the Standard Specifications. If the maximum cement factor is not specified, it shall not exceed 7.05 cwt/cu yd.

(b) The maximum allowable water/cement ratio shall be according to Article 1020.04 of the Standard Specifications or 0.44, whichever is lower.

(c) The slump requirements of Article 1020.04 of the Standard Specifications shall not apply.

(d) The coarse aggregate gradations shall be CA 13, CA 14, CA 16, or a blend of these gradations. CA 11 may be used when the Contractor provides satisfactory evidence to the Engineer that the mix will not segregate. The fine aggregate proportion shall be a maximum 50 percent by weight (mass) of the total aggregate used.

(Notes (e) through (j) of the provision are irrelevant to the discussion and are thus excluded)

Table A-1 of Appendix A highlights the constituents of previous SCC mixes that do not comply with the standards mentioned above. Nine of the SCC mixes in Table A-1 fail to meet either one or both of the criteria in Notes (a) and (b), which set the ranges for cement factors and w/c ratios for precast, prestressed members at 565 – 705 lbs/yd$^3$ and 0.32 – 0.44, respectively. Note (d) limits the fine aggregate proportion of an SCC mix in such a member from being greater than 50% by weight. Eight of the previous SCC mixes do not comply with this criterion.

### 4.2 ILLINOIS SCC MIX DESIGNS

Tables 4-1 and 4-2 detail two SCC mixes representative of those typically used in the State of Illinois. Table 4-1 describes the first design, developed by the Prestress Engineering Corporation (PEC) of Prairie Grove, IL, while Table 4-2 shows the second design, developed by the Egyptian Concrete Company (ECC) of Salem, IL. The constituents of both mixes adhere to current Illinois provisions, as evidenced in Table A-1.

Table 4-1. SCC Mix Design from Prestress Engineering Corporation (PEC)

| **Materials** | **SCC** (Quantity per $yd^3$) |
|---|---|
| Type III Portland Cement | 660 lbs |
| Coarse Aggregate | 1547 lbs |
| Fine Aggregate | 1447 lbs |
| Air-Entraining Agent | 19 oz |
| High-Range Water Reducer | 80 oz |
| Water | 30 gal |
| W/C | 0.379 |
| Entrained Air | 6.5 % |

Table 4-2. SCC Mix Design from Egyptian Concrete Company (ECC)

| **Materials** | **SCC** (Quantity per $yd^3$) |
|---|---|
| Type III Portland Cement | 650 lbs |
| Coarse Aggregate | 1367 lbs |
| Fine Aggregate | 1367 lbs |
| Fly Ash | 150 lbs |
| Air-Entraining Agent | 6 oz |
| High-Range Water Reducer | 54.75 oz |
| Normal-Range Water Reducer | 30 oz |
| Retarder | 7.5 oz |
| Water | 33 gal |
| W/C | 0.34 |
| Entrained Air | 6.5 % |

Table A-1 reveals several design parameters that are similar throughout the Illinois SCC mixes and those used in previous studies, such as the amount of high-range water reducer, water-to-cement ratio, and ratio of fine aggregate to total aggregate. However, the Illinois mixes are otherwise different than those detailed in Chapter 3. Mix #2 of the Girgis & Tuan study notwithstanding, both Illinois SCC mixes have lower cement contents than those used in previous studies. Additionally, viscosity modifying admixtures are absent from the Illinois mixes. The amount of air-entraining agent per cubic yard of the PEC mix design is higher than the amount used in nearly all previous mixes (Table A-1). The ECC mix, meanwhile, is the only to utilize normal-range water reducers and is one of few (see Tables 3-16 and 3-19) to incorporate the material additive fly ash (Tables 4-2 and A-1). Given the differences between the Illinois SCC mixes and those tested in previous studies, and recognizing the dependency of performance on mix composition, the need for continued research is evident.

Based on the comparison study presented in this chapter, it was concluded that many of the SCC mixes presented in Chapter 3 would not meet the current practice and provisions in the State of Illinois. Therefore, to facilitate IDOT's use of SCC in prestressed girders, research must be conducted on SCC mixes approved by IDOT and used in today's Illinois precast industry. Research utilizing these mixes would complement prior research and afford IDOT with new knowledge pertaining to the use of SCC in prestressed AASHTO bridge girders.

# CHAPTER 5 CONCLUSIONS AND RECOMMENDATIONS

## 5.1 CONCLUSIONS

This report summarizes the findings of a synthesis study conducted at the University of Illinois at Urbana-Champaign to investigate the effect of using SCC on the transfer and development length of prestressing tendons used in AASHTO bridge girders. The Illinois Department of Transportation (IDOT) sponsored this study to determine the feasibility of using SCC in the construction of future bridges in the State of Illinois using the current design specifications (AASHTO-LRFD and ACI), which is used for conventional concrete bridges. A comprehensive literature review was conducted using the University of Illinois Library Databases and the Transportation Research Information Services (TRIS, TRB). The outcome of the literature review showed that in the last few years seven major research studies were conducted on the subject at various States including Kansas, Virginia, Nebraska, Florida, Arkansas, Pennsylvania, and Michigan.

The work conducted under these studies consisted primarily of four stages: 1) Concrete mix design and testing, 2) Pull-out testing to evaluate the strands bond strength, 3) Transfer length testing, and 4) Development length testing, which was only conducted in three studies. The following is a summary of the major findings documented in the final reports of these seven studies:

- Concrete compressive strength seemed to have minor effect on the transfer and development lengths of prestressing strands regardless of the type of concrete.
- Strands location (i.e. top or bottom) and spacing had major impact on the transfer length values. Specimens with top strands and/or small spacing had on average a greater transfer length compared to those with bottom strands and/or larger spacing, respectively. On the other hand, the development length was less affected by the location and spacing of the strands compared to the transfer length.
- Of the seven studies, three showed that the bond strength and transfer length values measured for strands used in SCC were less than those measured for strands used in conventional concrete.
- Three studies illustrated that using SCC caused the transfer length values to exceed the limit recommended by the AASHTO and/or ACI design provisions.
- One of the three studies conduced on the development length evaluation showed that using SCC resulted in a development length value which exceeded the limit recommended by the AASHTO and ACI design provisions.

Based on the information obtained from the literature and presented in this report the authors were able to draw the following three main conclusions:

**(1)** SCC is a relatively complex material with properties that highly depend on the constituents and proportions of the concrete mix. Therefore, the transfer and development length results could be highly affected by the concrete properties which could differ significantly from one concrete producer to the other. This was evident in the inconsistent results and conclusions found in the literature.

**(2)** The geometry of the girders section, which directly influences the strands spacing and location could have a significant effect on the transfer and development length results. The information listed in this report were only based on the testing of either rectangular or I-shape sections. These results could vary with using other types of sections such as the box section, which is commonly used in the State of Illinois.

**(3)** The SCC mixes used in previous research do not necessarily comply with current Illinois standards and specifications for SCC used in precast prestressed members. Further tests utilizing IDOT-approved SCC mixes are necessary to provide data pertinent to IDOT's prestressed applications.

## 5.2 RECOMMENDATIONS

Before SCC is fully adapted by IDOT, further research is needed in order to complement the information and results obtained from the literature. Based on the final conclusions drawn by the authors in the previous section, it is recommended that large-scale girders typically used in Illinois bridges would be cast with IDOT-approved SCC and tested to determine the transfer and development lengths of their prestressing tendons. The two main parameters that should be taken into account during these tests are: (1) The SCC mix design, and (2) The geometry of the tested girders. These two parameters should reflect the current practice followed by IDOT and Illinois precast companies.

## REFERENCES

*AASHTO LRFD Bridge Design Specifications*, Third Edition. American Association of State Highway and Transportation Officials, Washington, DC, 2004.

*ACI 318-08: Building Code and Commentary.* American Concrete Institute, Farmington Hills, MI, 2008.

Balazs, G. L. "Transfer Lengths of Prestressing Strands as a Function of Draw in and Initial Prestress." PCI Journal 38(2), 1993, pp. 86-93.

Collins M.P. and D. Michell. *Prestressed Concrete Structures.* Prentice Hall, Englewood Cliffs, NJ, 1991, pp. 98-99.

*Concrete Library 93, High-fluidity Concrete Construction Guideline*, Japan Society of Civil Engineers, 1999.

Cousins, T., D.W. Johnston, and P. Zia. *Bond of Epoxy Coated Prestressing Strand*. Publication No. FHWA/NC/87-005, Federal Highway Administration, Washington, DC, Dec. 1986.

Gaimster R and J. Gibbs. "Self-compacting concrete", CPS no. 123, the Materials and its Properties. Concrete, Vol.35, No.7, July/Aug. 2001.

Girgis, A.F. and C.Y. Tuan. *Bond Strength of Self-Consolidating Concrete for Prestressed Concrete App*lications. Report No. SPR-PL-1 (037) P528 & EACNH67114, Nebraska Department of Roads Bridge Division, Nebraska Department of Roads, Omaha, NE, Dec 2004, 36 p.

Hamilton, H.R., and T. Labonte. *Self-consolidating concrete structural investigation.* Final Report No. BD545 RPWO#21, Florida Department of Transportation Research Management Center, Florida Department of Transportation, Gainesville, FL, Mar 2005, 117 p.

Haq, M. Effect of Self-Consolidating Concrete (SCC) Mix Proportioning on Transfer and Development Length Prestressing Strands. MS Dissertation, Michigan State University, East Lansing, MI, 2005, 210 p.

*Interim Guidelines for the Use of Self-Consolidating Concrete in Precast/Prestressed Concrete Institute Member Plants,* First Edition. Precast/Prestressed Concrete Institute, Chicago, IL, 2003 (denoted in text as PCI 2003a).

Khayat, K.H., K. Manai and A. Trudel. “In Situ Mechanical Properties of Wall Elements cast using Self-Consolidating Concrete.” ACI Material Journal 94(6), Dec 1996, pp. 491-500.

Khayat, K.H., P. Paultre and S. Tremblay. “Structural Performance and In-Place Properties of Self-Consolidating Concrete used for Casting Highly Reinforced Column.” ACI Material Journal 98(5), Oct. 2001, pp. 371-378.

Larson, K.H. Evaluating the Time-Dependent Deformations and Bond Characteristics of a Self-Consolidating Concrete Mix and the Implication for Pretensioned Bridge Applications. PhD Dissertation, Kansas State University, Manhattan, Kansas, 2006, 229 p.

Larson, K.H., R. J. Peterman, and A. Esmaeily. *Evaluating the Time-Dependent Deformations and Bond Characteristics of a Self-Consolidating Concrete Mix and the Implication for Pretensioned Bridge Applications*. Final Report No. FHWA-KS-07-1, Kansas Department of Transportation Bureau of Materials and Research, Kansas Department of Transportation, Topeka, KS, April 2007, 252 p.

Larson, K.H., R. J. Peterman, and A. Esmaeily. “Bond Characteristics of Self-Consolidating Concrete for Prestressed Bridge Girders.” PCI Journal 52(4), 2007, pp. 44-57.

Logan, D.R. “Acceptance Criteria for Bond Quality of Strand for Pretensioned Prestressed Concrete Application.” PCI Journal 42(2), pp. 52-90, 1997.

Moustafa, S. *Pull-out Strength of Strand and Lifting Loops*. Concrete Technology Associates Technical Bulletin, 74-B5, available from Precast/Prestressed Concrete Institute, Chicago, IL, 1974.

Naito, C. J., G. Brunn, G. Parent and T. Tate. *Comparative Performance of High Early Strength and Self-consolidating Concrete for Use in Precast Bridge Beam Construction*. ATLSS Report #05-03, Advanced Technology for Large Structural Systems, Bethlehem, PA, May 2005, 102 p.

Naito, C.J., G. Parent, and G. Brunn. “Performance of Bulb-Tee Girders Made with Self-Consolidating Concrete.” PCI Journal 51(6), 2006, pp.72-85.

Okamura, H. “Self-Compacting High-Performance Concrete.” *ACI Material Journal* 96(3), 1999, pp.346-353.

Ouchi, M., S. Nakamura, T. Osterberg, S. Hallberg and M. Lwin. “Applications of Self-Compacting Concrete in Japan, Europe and the United States.” Bridge Technology Page, 2003, http://www.fhwa.dot.gov/BRIDGE/scc.htm, accessed Jun 11, 2008.

Persson, B. “A Comparison between Mechanical Properties of Self-Compacting Concrete and the Corresponding Properties of Normal Concrete.” Cement and Concrete Research, 2001, pp. 193-198.

*Precast/Prestressed Concrete Bridge Design Manual.* Precast/Prestressed Concrete Institute, Chicago, IL, 2003, Ch.8 (denoted in text as PCI 2003b).

*Publication 408/2003 – Specifications*. Commonwealth of Pennsylvania Department of Transportation, www.dot.state.pa.us, Oct 2004.

Ruiz, E.D., B.W. Staton, N.H. Do and W.M. Hale. “Estimating Transfer Lengths of Precast/Prestressed Beams Cast with Self-Consolidating Concrete.” TRB 2007 Annual Meeting, 2006.

Russell, B.W., and N. H. Burns. *Design Guidelines for Transfer, Development and Debonding of Large Diameter Seven Wire Strands in Pretensioned Concrete Girders.* Research Report 1210-5F, Center for Transportation Research, the University of Texas at Austin, Austin, TX, Jan. 1993, 300 p.

Russell, B. W., and N. H. Burns. “Measurement of Transfer Lengths on Pretensioned Concrete Elements.” Journal of Structural Eng, May 1997, pp. 541-549.

Trent, J. D. Transfer Length, Development Length, Flexural Strength, and Prestress Loss Evaluation in Pretensioned Self-Consolidating Concrete Members. MS Dissertation, Virginia Polytechnic Institute and State University, Blacksburg, VA, 2007, 152 p.

# APPENDIX A MIX DESIGN CONSTITUENTS AND STANDARDS

Table A-1. SCC Mix Designs from Previous and Proposed Research Compared to Current IDOT Standards

| | | LARSON | TRENT | | HAQ | | | | GIRGIS | | HAMILTON | NAITO | RUIZ | **PEC** | **ECC** |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Materials | Units | | S1CCM | S1CCM2 | SCC1 | SCC2A | SCC2B | SCC3 | Mix #1 | Mix #2 | | | | | |
| Cement (Any Type) | lbs | 750 | 750 | 745 | 750 | 700 | 700 | 700 | 800 | 632 | 752 | 849* | 950 | **660** | **650** |
| Fly Ash (Any Type) | lbs | - | - | - | - | - | - | - | 150 | 100 | 168 | - | - | **-** | **150** |
| Coarse Aggregate | lbs | 1360 | 1625 | 1650 | 1478.57 | 1380 | 1380 | 1435 | 1282 | 1311 | 1307 | 1651 | 1350 | **1547** | **1367** |
| Fine Aggregate | lbs | 1500 | 1340 | 1308 | 1627.5 | 1426 | 1426 | 1275 | 1417 | 1449 | 1414 | 1283 | 1474 | **1447** | **1367** |
| Fine Agg. / Total Agg. | | 0.52 | 0.45 | 0.44 | 0.52 | 0.51 | 0.51 | 0.47 | 0.53 | 0.53 | 0.52 | 0.44 | 0.52 | **0.48** | **0.50** |
| Air-Entraining Agent | oz. | 5 | - | - | - | - | - | - | - | - | 1.8 | 2 | - | **19** | **6** |
| | oz/cwt | - | - | - | 1.75 | 0.75 | 1.75 | 3.18 | - | - | - | - | - | **-** | **0.9** |
| High-Range Water Reducer | oz. | 70 | - | - | | - | - | - | 2~14 | 2~14 | 64.4 | 136.2 | - | **80** | **54.75** |
| | oz/cwt | - | - | - | 12.93 | 14.59 | 12.03 | 15.37 | - | - | - | - | 11 | **-** | **7.3** |
| Viscosity Modifying Admixture | oz. | - | - | - | | - | - | - | 2~10 | 2~10 | - | 16 | - | **-** | **-** |
| | oz/cwt | - | - | - | - | 6.99 | 1.78 | 15.37 | - | - | - | - | 2 | **-** | **-** |
| Water | gal | 27 | 34 | 34 | 31 | 33 | 33 | 37 | 35 | 35 | 31 | - | 34 | **30** | **33** |
| W/C Ratio | | 0.3 | 0.38 | 0.38 | 0.35 | 0.42 | 0.4 | 0.45 | 0.31 | 0.4 | - | 0.32 | 0.3 | **0.38** | **0.42** |
| Set Retardant | oz. | - | - | - | | - | - | - | 0~5 | 0~5 | 13.8 | - | - | **-** | **7.5** |
| | oz/cwt | - | - | - | 70 | - | 58.57 | 46.67 | - | - | - | - | - | **-** | **1** |

- Exceeds maximum cement factor of 705 lbs/yd$^3$

- Water/cement ratio falls outside range of 0.32 - 0.44

- Fine aggregate proportion exceeds maximum of 50% of total aggregate by weight

Note: *Mix also contains 25 lb slag

Illinois Department of Transportation

ILLINOIS

UNIVERSITY OF ILLINOIS AT URBANA-CHAMPAIGN