BOND STRENGTH OF SELF-CONSOLIDATING CONCRETE FOR PRESTRESSED CONCRETE APPLICATIONS

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CHAPTER 1

INTRODUCTION

1.1 INTRODUCTION

Self-consolidating concrete (SCC) has recently gained significant popularity in the precast industry in the United States. It is "a highly flowable, yet stable concrete that can spread readily into place and fill the formwork without any consolidation and without undergoing significant separation."[1] SCC has been used in many precast concrete products, especially those with narrow forms and those requiring heavy reinforcement. Self-consolidating concrete has been defined largely by its three characteristics[1, 2]:

- Filling ability – ability to fill all spaces in formwork under its own weight;
- Passing ability – ability to fill spaces around reinforcing bars and other reinforcement under its own weight; and
- Resistance to segregation – composition remains uniform throughout transportation and placement.

1.2 PROBLEM STATEMENT

Although SCC has become very popular in the United States, there have been concerns regarding the bond strength, transfer length, and development length of prestressing strands and steel reinforcement with SCC. A self-consolidating concrete mix contains admixtures that act as lubricants to enhance its flowability. There are concerns that the admixtures would also weaken the bond between the concrete and the reinforcement. Very few studies have been conducted to evaluate the bond strength of SCC in the United States. Some of the studies reported that SCC had higher bond strength than that of the conventional concrete, and yet data in the very same studies indicated inadequate early-age bond strength of the SCC, which greatly affects the transfer length. Furthermore, there are no guidelines for determining the bond strength, transfer length or development length when using SCC. Therefore, there is an urgent need to investigate the bond strength of SCC with pre-tensioning strands and steel reinforcing bars, as compared to conventional concrete.

1.3 GOAL AND BENEFITS OF THE RESEARCH

The goal of this research was to experimentally measure the bond strength and transfer length of several pretensioned bridge girders cast with SCC. In addition, the bond strength of SCC with prestressing strands and common reinforcing bars were evaluated by Moustafa pull-out tests. It is expected that the experimental data would confirm that the use of SCC would not have adverse effects on the transfer length and development length of pretensioned girders.

1.4 LITERATURE REVIEW

SCC was first developed in the late 1980's[2] by researchers led by Okamura and Ozawa at the University of Tokyo, Japan. This highly workable concrete virtually places itself and therefore does not require as many workers in the field as pouring regular concrete. Labor savings are the main advantage of using SCC. SCC mixes may be categorized into three types: (1) the powder type, which contains a high powder content; (2) the VMA type, which utilizes <u>v</u>iscosity <u>m</u>odifying <u>a</u>dmixtures (VMA); and (3) the combined type, which contains both powder and VMA.[3] SCC requires a higher content of fine particles than the conventional concrete to increase flowability and to avoid segregation and bleeding. For example, conventional concrete typically contains about 38% of fine particles, while SCC mixtures require about 46% of fine particles. The additional fine particle content is accomplished by replacing cement with materials that have lower specific gravity. These materials include ground granulated blast-furnace slag and pozzolans such as fly ash, silica fume and calcined shale.[4]

Pull-out tests on steel reinforcing bars of 12-mm (0.5 in.) and 20-mm (0.8 in.) diameter were conducted at the University of Paisley.[5] Results showed that the bond strength of SCC was about 18 to 38% higher than that of regular concrete mixes. Chan et al.[6] at the National Taiwan University also found that the SCC members had significantly higher bond strength with reinforcing bars than did ordinary concrete members. They also reported that the reduction in bond strength due to bleeding and inhomogeneity in the ordinary concrete was prevented with the use of SCC.

Investigations conducted in the United States consisted of pull-out tests as well, with the top-bar factor calculated. This factor is defined as the bond strength of the bottom layer of reinforcing bars divided by the bond strength of the top layer. In the tests conducted by Attiogbe et al.[7], self-consolidating concrete yielded similar top-bar factors to those of normal concrete with 102 to 152 mm (4 to 6 in.) of slump. In a test using air-cured SCC and a VMA admixture, the top-bar factor was actually lower than that of conventional concrete. Attiogbe et al.[8] , in testing using both reinforcing bars and prestressing strands, concluded that the highly stable nature of SCC mixes enhanced the top-bar factor. However, the test results showed that, in half of the cases, the bond strength of the conventional concrete with prestressing strands was higher than that of the SCC. Khayat[9] reported top-bar factor improvement with the use of SCC which he accredited to the reduction in bleeding and segregation. Based on extensive experimentation, Carrasquillio[10] at the University of Texas at Austin also stated that "in no case was the pullout capacity of straight deformed bars embedded in superplasticizered concrete significantly less than that of the bars embedded in the concrete containing no superplasticizer."

From the literature review we can conclude that, test results from the previous studies suggest that the bond strength of SCC with deformed reinforcing bars is adequate. However, there has been no definitive test data to prove that the bond strength of SCC with prestressing strands is adequate.

1.5 SCOPE AND LAYOUT

Several tests were performed to determine the bond strength of both the 270 ksi, 0.6 in. diameter seven wire, low relaxation strands as and the grade 60 reinforcing bars. These tests included Moustafa pull-out test, transfer length tests and small specimen pull-out tests.

This report is divided into five chapters. Chapter One provides background information and summary of literature review. Chapter Two describes the pull out test specimens and summarizes the test results. Chapter Three describes the pretensioned bridge girders instrumented for transfer length measurement, the measuring procedure and results. Chapter Four provides a description of the test specimens for the small specimen pull-out tests, the test setup and testing procedure, and a brief discussion on the test results. Chapter Five provides conclusions and recommendations. Appendix I describes the material properties of the SCC that were used in testing. Appendix II provides Moustafa pull-out test results.

CHAPTER 2

STRAND PULL-OUT TESTING

2.1 BACKGROUND

There is no accepted standard test method for bond quality of prestressing steel, consequently a Precast/ Prestressed Concrete Institute Committee decided to use a simple pull-out test procedure developed by Moustafa in 1974 for testing lifting loops at the Concrete Technology Corporation (CTC) in Tacoma, Washington. The test method consisted of measuring the maximum pull-out force resisted by an untensioned, 0.5 in strand embedded 18 in (457 mm) within a concrete test block (Moustafa 1974). The 1992 test program included strand from seven different manufacturers, despite the fact that the Moustafa test does not accurately represent the bond performance of pretensioned strand. University of Oklahoma developed a research program to evaluate the effectiveness of various test methods for strand bond quality from three manufacturers (Rose and Russell 1997). Later Logan (1997) initiated a test program at Stresscon Corporation in Colorado to correlate the results of Moustafa pull-out tests with the results of development length tests of both simply-supported and cantilever beam specimens.

In this study, Moustafa pull-out test procedure was followed to measure the bond capacity of the 0.6 in. diameter smooth strands. Three concrete mixtures were tested in this study. The first two were SCC mixtures, while the third was a conventional concrete mixture. Appendix I provides the details about the concrete mixes.

2.2 PULL-OUT SPECIMENS AND TEST SETUP

Moustafa pull-out tests were conducted to determine the bond capacity of 0.6-in. low-relaxation, un-tensioned strands. The embedment length of all the strands tested was 18 in. (457 mm). The dimensions, reinforcement details, and the strand layout of the test specimens are shown in Figures 2.2.1 and 2.2.2.

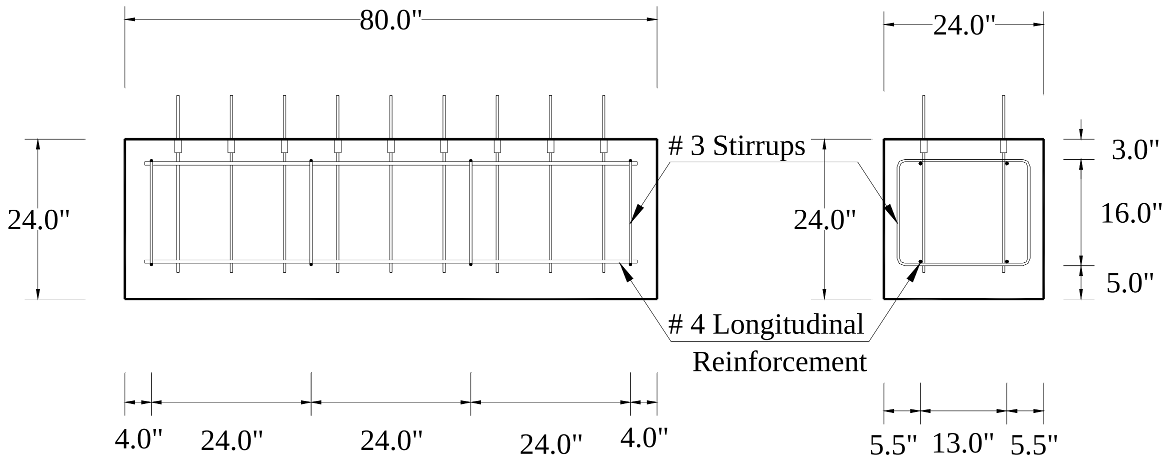


Figure 2.2.1 Moustafa Pull-out Test Dimensions and Reinforcement Details

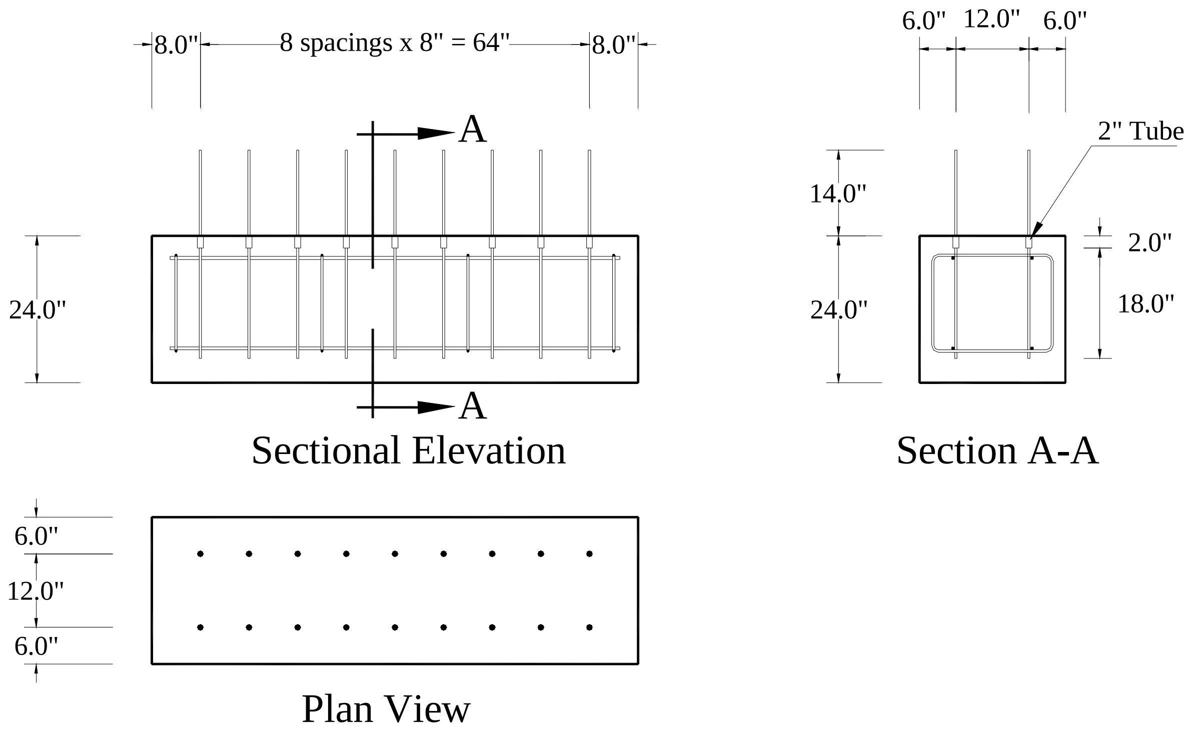


Figure 2.2.2 Moustafa Pull-Out Test Strand Layout

Six specimens were prepared for the pull-out tests. Two specimens were prepared for each concrete mix. The first specimen for each mix had standard dimensions with 18 in. (457 mm) embedment. The second specimen for each concrete mix had variable embedment length (16 in.,18 in. and 20 in.).

2.3 TEST PROCEDURE

The strands were saw-cut to the required lengths. The strand samples were visually examined to verify that there is no rust or dirt. Test block forms were setup for 2x2x6.67 ft (610x610x169mm) dimensions as shown in Figure 2.2.1. A reinforcement cage was made from # 4 longitudinal bars and #3 stirrups as shown in Figure 2.2.2. The cage was secured inside the form by chairs. The strand samples were tied securely in place with the required embedment length. Each strand was taped by duct tape at the end of the embedment length then a 2 in. plastic tube was used to cover the duct tape. The concrete was cast from the same mix of the bridge girders. 17 cylinders per concrete mix (4 in.x 8 in.)(102 mm x 203 mm) were kept to monitor the concrete strength development with time.

A central-hole hydraulic jack with a 110 kips (55 ton) capacity was used to pull-out the strands. A load cell was located at the top of the jack to record the pull-out force. A steel plate and a chuck were placed at the top of the load cell. A steel frame was positioned between the jack and the specimen. This test setup is shown in Figure 2.3.1.

Figure 2.3.1 Moustafa Pull-out Test Setup

2.4 RESULTS AND DISCUSSION

Results of the 0.6 in (15.2 mm) strand pull-out tests are given in Appendix 2. The average pull-out capacity after two days using mix # 1 was 43.4 kips (21.7 tons) with a standard deviation of 2.7 kips. Figure 2.4.1 shows a comparison between the test results using mix # 1 and results from the literature. The pull-out data from the previous literature were interpolated to match the same compressive stress before comparing to the results from this study. Moustafa's benchmark was developed for 0.5 in. (12.7 mm) strands. For the purpose of this study, a multiplier was applied based on the ratio of the circumference of the 0.6 in. (15.2 mm) strand to the 0.5 in. (12.7 mm) strand.

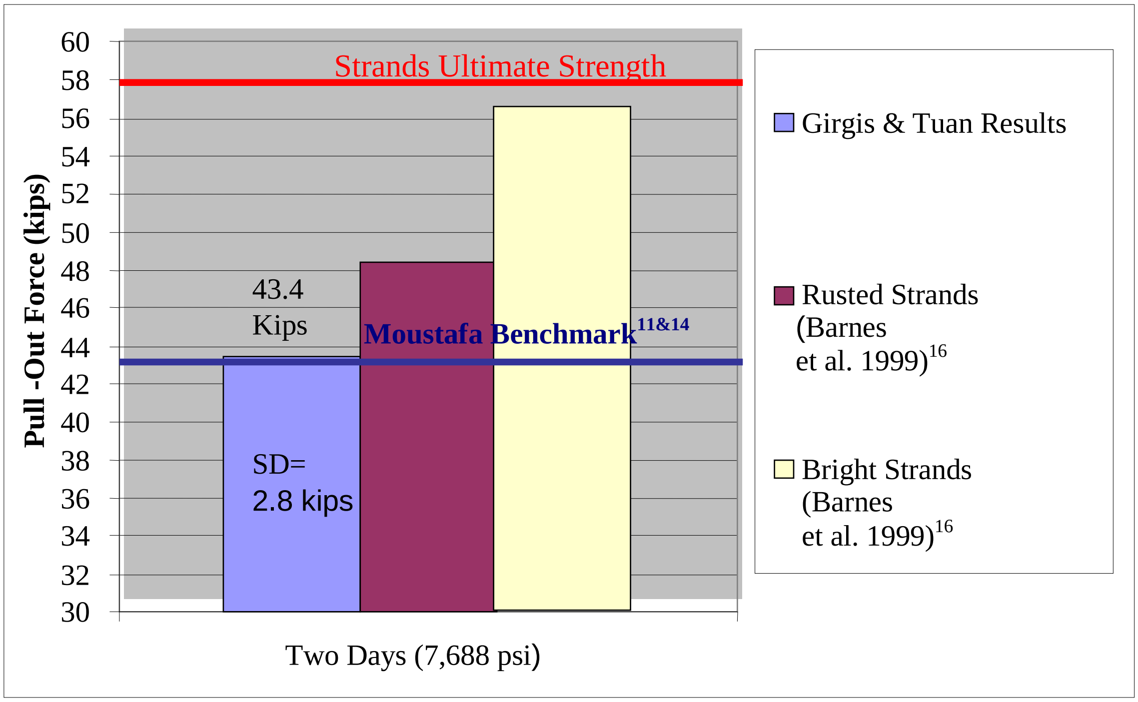


Figure 2.4.1 Mix #1 Pull-Out Capacity vs. Data from Literature

The average pull-out capacity using mix # 2 was 54.15 kips (27.08 tons) at one day from casting and 65.68 kips (32.84 tons) at 28 days with a standard deviation of 5 kips (2.5 tons) at one day from casting and 2.68 kips (1.34 tons) at 28 days. Figure 2.4.2 shows a comparison between the test results using mix # 2 and results from the literature. The pull-out data from the literature were also interpolated to match the same compressive stress before comparing to the results from this study.

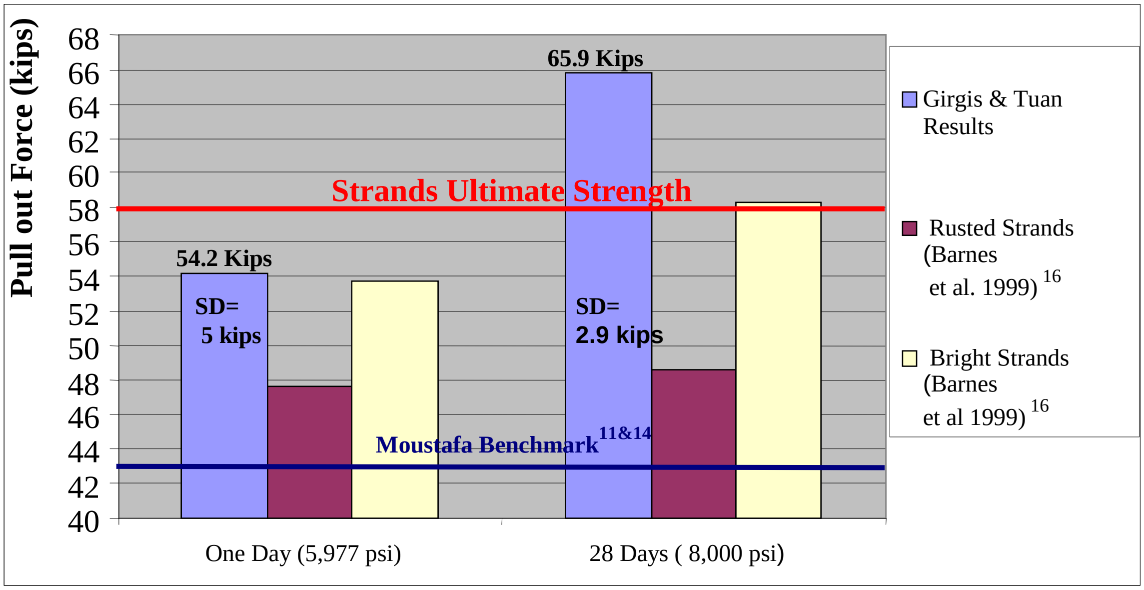


Figure 2.4.2 Mix #2 Pull-Out Capacity vs. Data from Literature

Figure 2.4.3 shows a comparison between of the average pull-out capacity between the SCC concrete mix # 2 and the conventional concrete mix # 3. The average pull-out capacity of the conventional concrete was 48.0 kips (24.0 tons) with a standard deviation of 8.6 kips (4.3 tons) as shown in Figure 2.4.3.

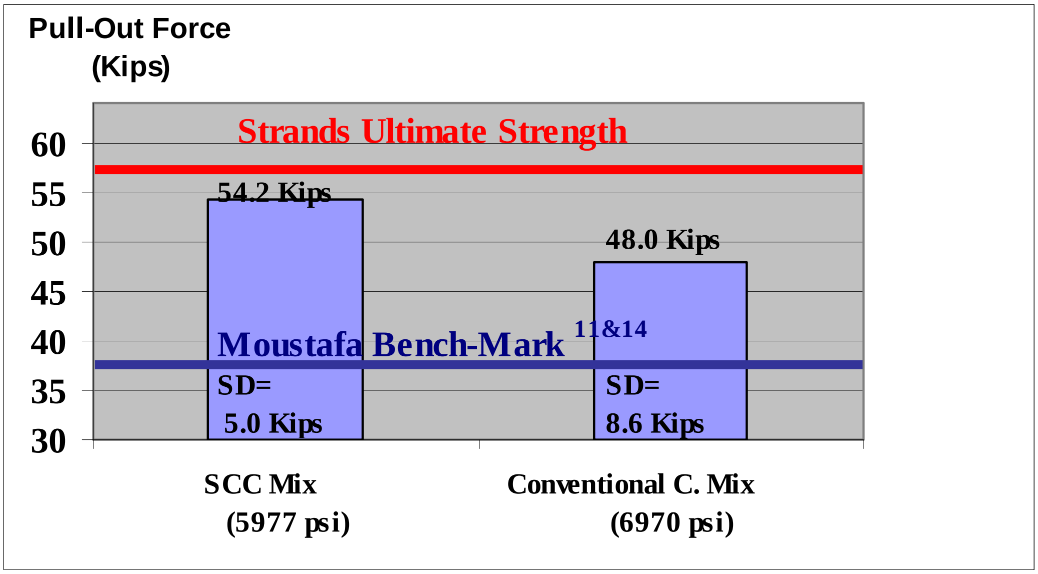


Figure 2.4.3 Comparison of Pull-Out Capacities of Mix #2 (SCC) vs. Mix #3 (Conventional Concrete)

Pull-out tests of different embedment lengths were conducted using mix # 1 as shown in Figure 2.4.4. Figure 2.4.4 shows that the pull-out capacity of 18 in. embedment was higher than that of the 20 in. The inconsistency was probably due to the higher standard deviation of the 18 in. strand embedment.

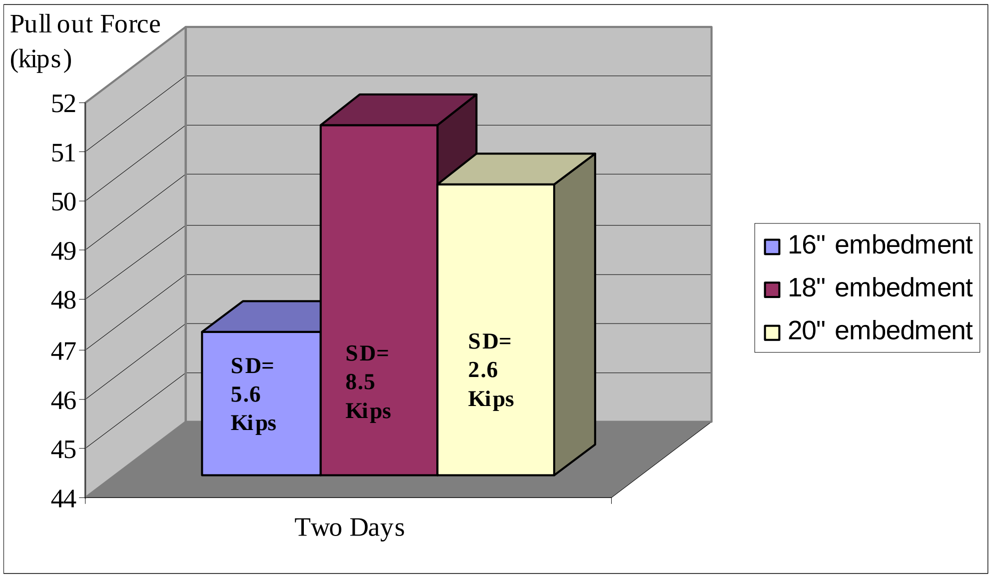


Figure 2.4.4 Mix #1 Pull-Out Capacity for various embedment lengths

Pull-out tests of different embedment lengths were also tried using mix # 2 as shown in Figure 2.4.5 at one day and 28 days. The data of the pull-out test at one day is consistent. However, there is no valuable information to be gained from the 28 day data since all of the data exceeds the ultimate strength of the strand.

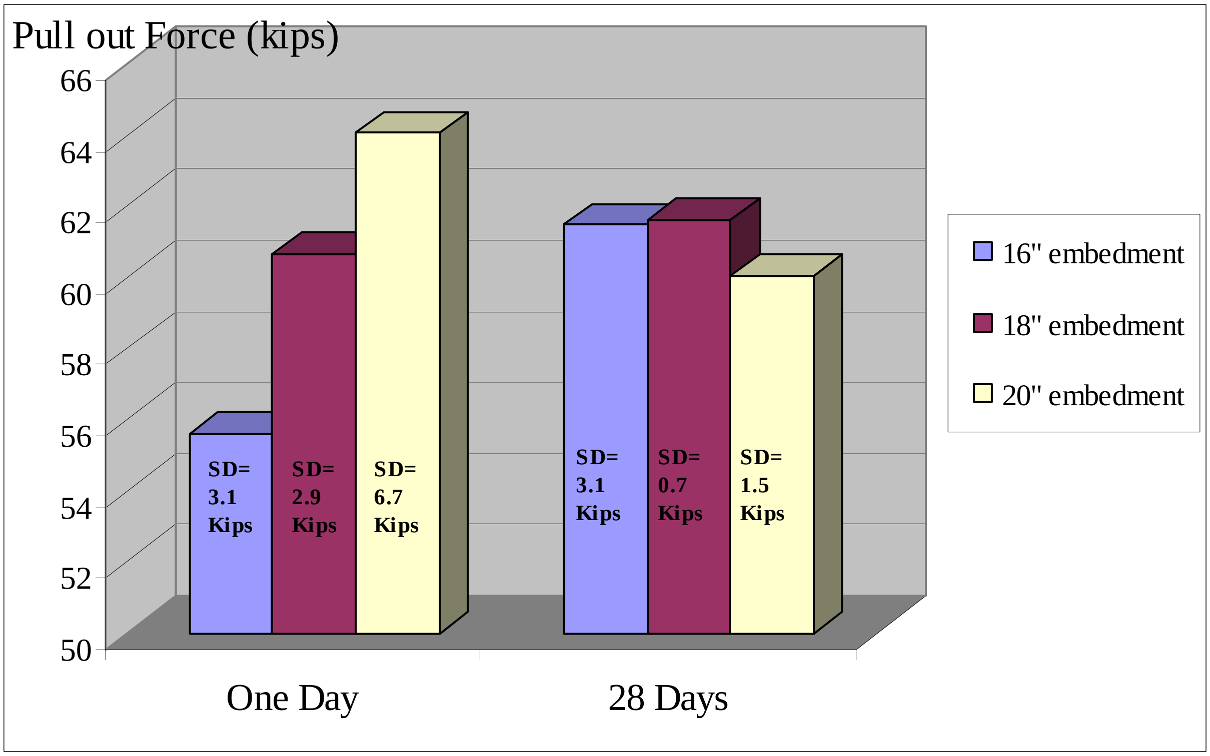


Figure 2.4.5 Mix # 2 Pull-Out Capacity for various embedment lengths

Figure 2.4.6 shows the pull-out tests results of different embedment lengths for mix three. The testing was conducted at one day and 28 days since casting.

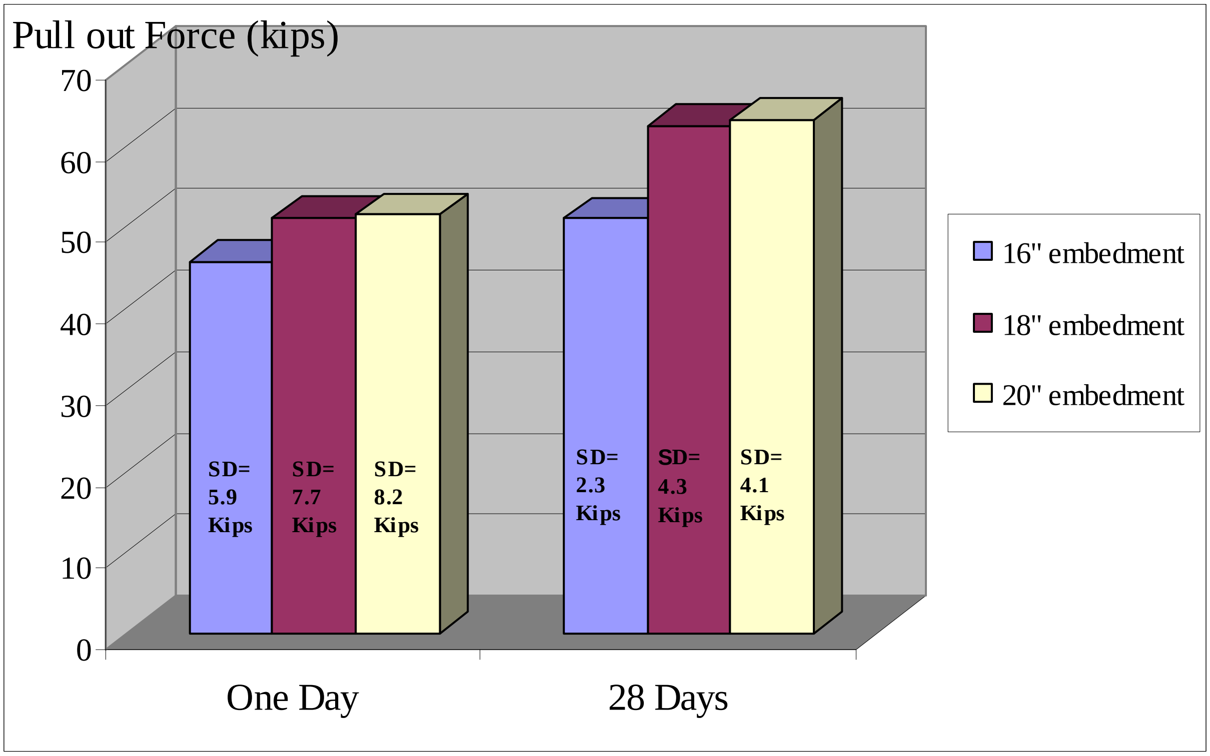


Figure 2.4.6 Mix # 3 Pull-Out Capacity for various embedment lengths

CHAPTER 3

TRANSFER LENGTH TESTING

3.1 INTRODUCTION

Transfer Length L_t is the length of strand over which the prestress force in pretensioned members is transferred to the concrete. The term has no meaning for nonprestressed reinforcement.

Due to losses, the prestress level varies with time. Prestress is transferred to concrete through adhesion, Hoyer effect, and mechanical interlock. Once strands slip, adhesion is lost. Hoyer effect is the most important effect, followed by mechanical interlock. Short transfer length may cause excessive concrete stress at transfer and may result in splitting or bursting cracks in the end zone. Long transfer length may reduce girder shear resistance and imply long development length, which may adversely affect the flexural strength of the girder.

3.2 TESTED GIRDERS AND PROJECT DESCRIPTIONS

Three girders were tested in this study. Transfer length was measured at the four corners of the bottom flange of each girder. The same three concrete mixtures that were tested in the pull-out tests were also used in the three test girders. Table 3.2.1 shows three bridge projects and the concrete mix used in each project.

Table 3.2.1 Bridge Projects Investigated

Project	Project I Oak Creek Bridge	Project II Clarks South Bridge	Project III North Broadway Bridge
County	Lancaster, Nebraska	Merrick, Nebraska	Sedgwick, Kansas
Mix	Mix # 1 (SCC)	Mix # 2 (SCC)	Mix # 3 (Conventional Concrete)

3.2.1 Project I

Project I is a three-span bridge, 22-30-22 m (72.5-100-72.5 ft) long, built with the NU1100 I-girders. The girder depth is 1,100 mm (3 ft 7.3 in.) and the web width is 150 mm (5.9 in). The bridge cross-section consists of 14 girders spaced at 2,819 mm (9 ft 3 in.), with an overall width of 38,608 mm (126 ft 8 in.). The cast-in-place concrete slab is a 190 mm (7.5 in.) thick composite deck on the girder. There are three girder segments per girder line. The end segments are 22,098 mm (72 ft 6 in.) long each, and the field segment is 30,175 mm (99 ft 0 in.) long. These lengths allow for two splice joints. Mix # 1 was used for the girders. The girder tested in this study was a 72 ft 6 in. long, exterior girder. As shown in Figure 3.2.1.1, the girder has 14 – 0.6 in. straight strands at 2 in. spacing, 2 harped strands, and 4 top strands. Figure 3.2.1.1 shows the girder cross section and the reinforcement in project I.

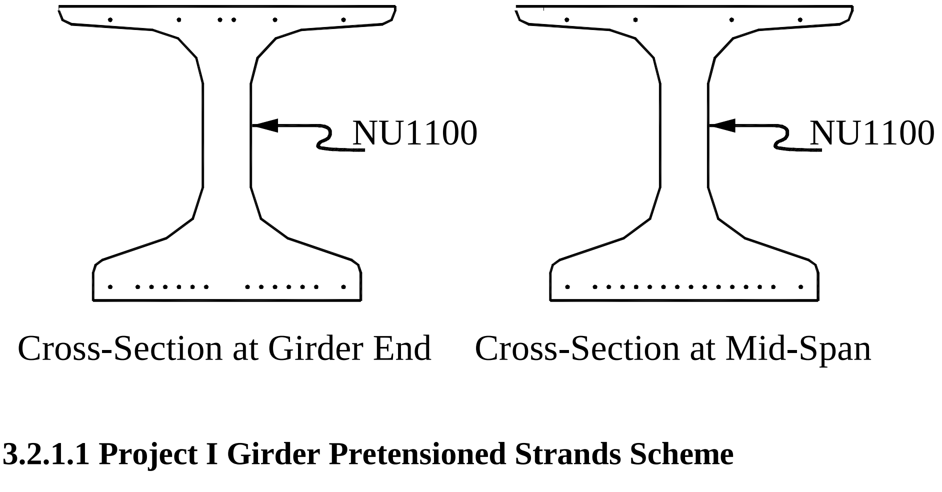
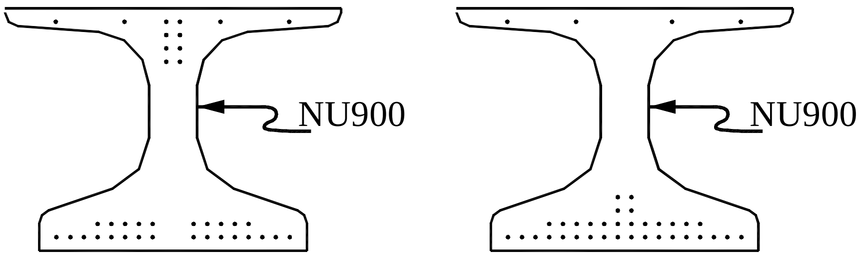


Figure 3.2.1.1 Project I Girder Pretensioned Strands Scheme

3.2.2 Project II

Project II is a 27,432 mm (90 ft 0 in.) single-span bridge using the NU900 I-girders. The girder depth is 900 mm (2 ft 11.4 in.) and the web width is 150 mm (5.9 in.). The bridge cross-section consists of 6 girders spaced at 2,438 mm (8 ft 0 in.). The bridge width is 14,122 mm (46 ft 4 in.). The cast-in-place concrete slab is 190 mm (7.5 in.) thick composite deck on the girder. Mix # 2 was used for the girders. The girder tested in this study was a typical 90 ft 2 in.-long girder. As shown in Figure 3.2.2.1, the girder has 26 – 0.6 in. straight strands at 2 in. spacing, 8 harped strands, and 4 top strands. Figure 3.2.2.1 shows the girder cross section and the reinforcement in project II.



Cross-Section at Girder End   Cross-Section at Mid-Span

Figure 3.2.2.1 Project II Girder Pretensioned Strands Scheme

3.2.3 Project III

The NU1350 I-girders were used in project III. The girder depth is 1,350 mm (4 ft 5.6 in.) and the web width is 150 mm (5.9 in.). Mix # 3 was used for the girders of this bridge. The girder tested in this study was 37,795 mm (124 ft) long. As shown in Figure 3.2.3.1, the girder has 44 –

0.5 in. straight strands at 2 in. spacing, 10 – 0.5 in. harped strands, and 4 top strands. Figure 3.2.3.1 shows the girder cross section and the reinforcement in project III.

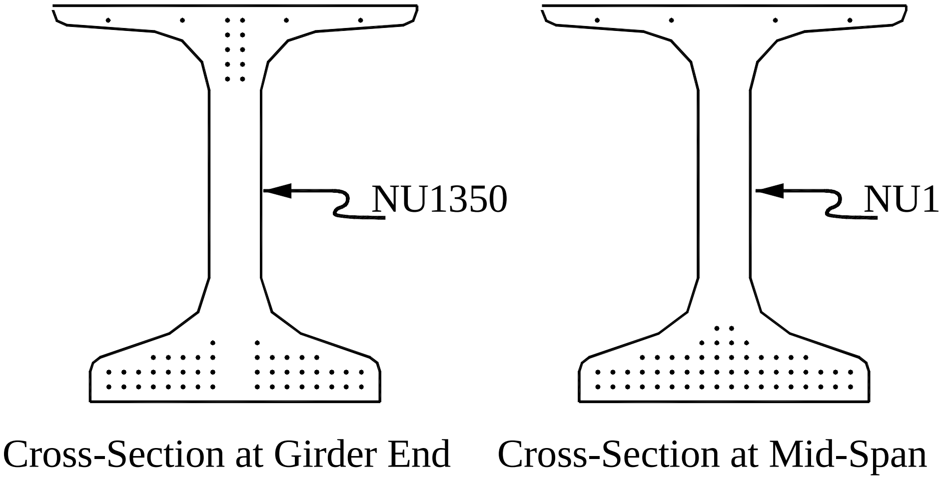


Figure 3.2.3.1 Project III Girder Pretensioned Strands Scheme

3.3 TRANSFER LENGTH MEASUREMENTS

A line of Demec points were mounted on the bottom flanges at the bottom flange strands centroid. Demec points are small stainless steel circular discs with a 1 mm pinhole at the center for precise distance measurements with a caliper refer to Figures 3.3.1 and 3.3.2. A fast-setting, two-part epoxy was used to bond the Demec points to the concrete surface.

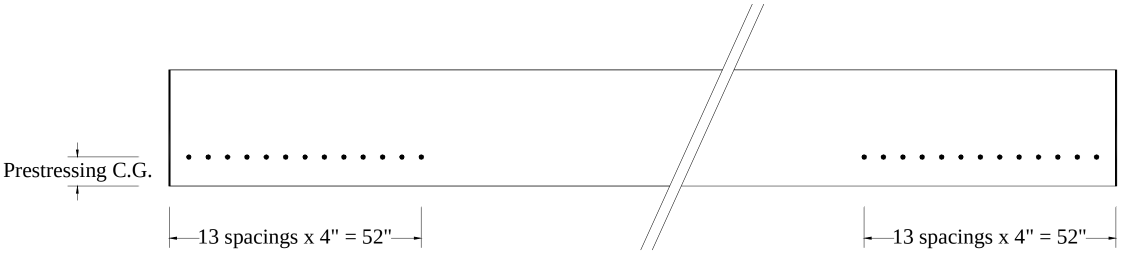


Figure 3.3.1 Girder Elevation Showing Locations of Demec Points

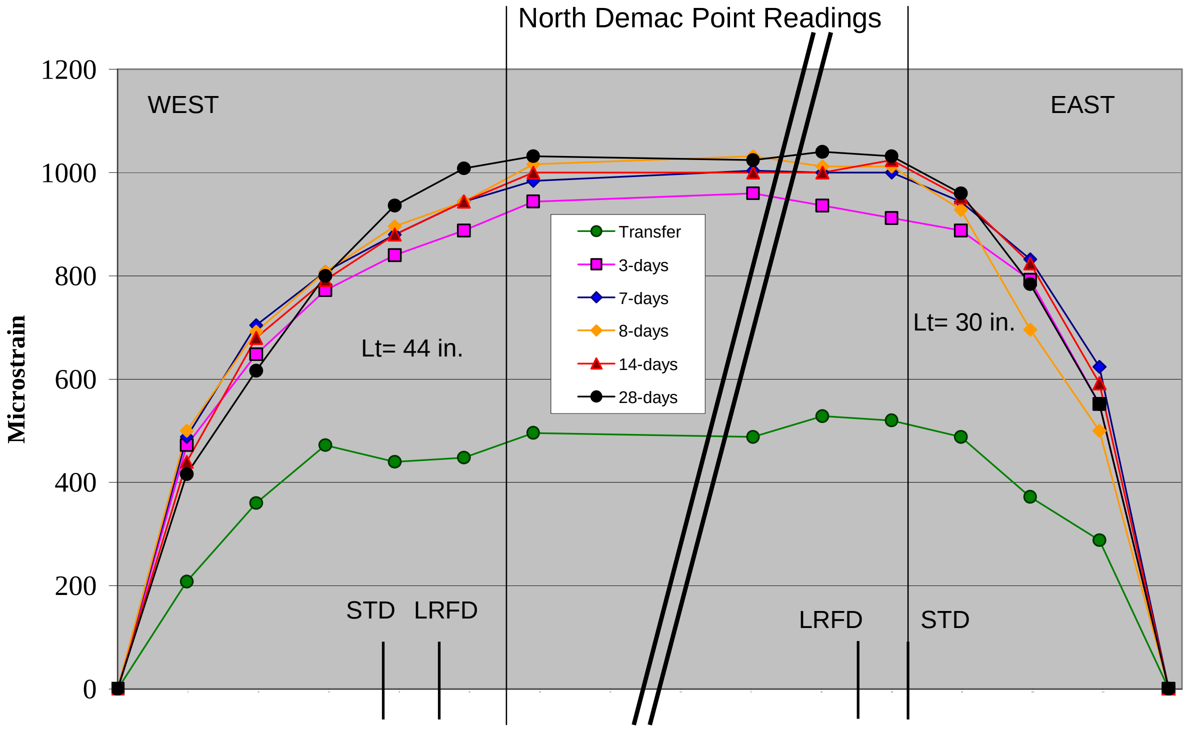
Figure 3.3.2 Demec Point Locations

Readings of the distances between Demec points were taken before and after releasing the prestressing force, and at 3, 7, 14 and 28 days since casting the concrete. The reading of the distances before prestress release was taken as the baseline data. The distances between Demec points were measured using a caliper gage as shown in Figure 3.3.3. The change in this distance was used to calculate the strain in the concrete at the center of gravity (C.G.) of the strands of the bottom flange. The concrete strains along the strands' C.G. were then plotted along the length of the girder. The concrete strains are zero at girder ends and increase from the girder end until they become constant, at which point all prestressing forces are transferred to the concrete. As recommended by Lane[17], the transfer length can be determined by measuring the distance from the end of the girder to the point where 95% of the maximum concrete strain was measured.

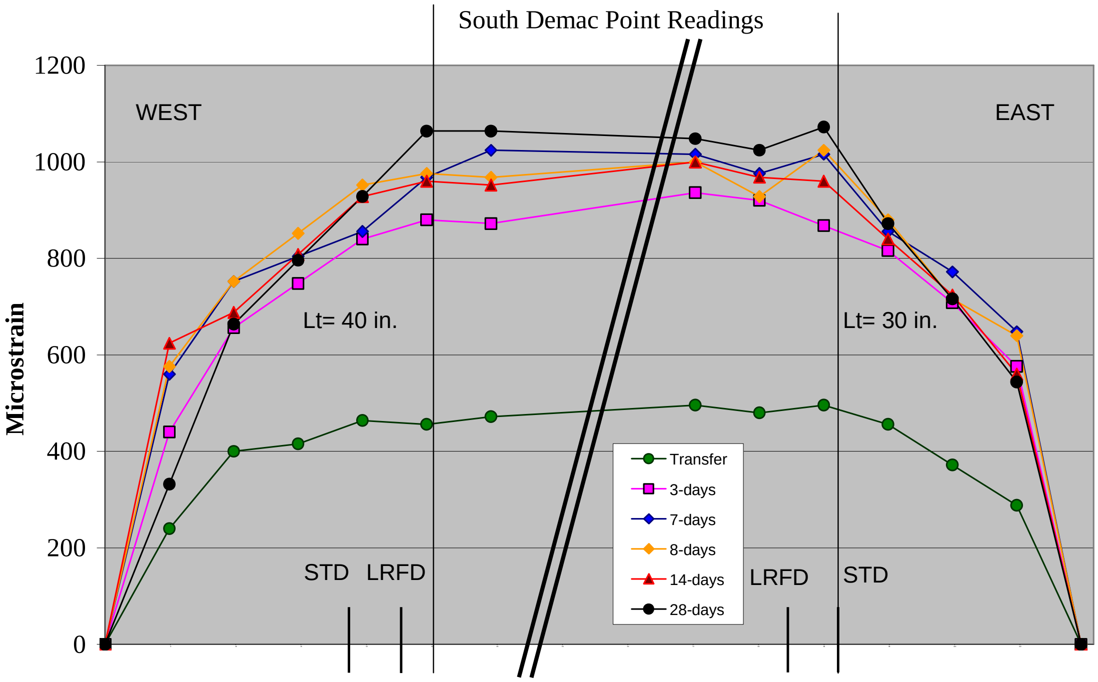
Figure 3.3.3 Transfer Length Measurement

3.4 TRANSFER LENGTH RESULTS

Demec points were mounted on both sides and both ends of the bottom flanges of the three bridge girders. Figures 3.4.1 through 3.5.3 present the concrete strain variations along the girder bottom flange for project I, II, and III, respectively. By averaging data obtained from the four corners of each of the girders, the average transfer lengths of the SCC mixes #1 and #2 were determined to be 914 mm (36 in.) and 1092 mm (43 in.), respectively. These values are longer than the 50 strand diameters 762 mm (30 in.) specified by the ACI 318[18] and the AASHTO Standard Bridge Specifications[19] and the 60 strand diameters 914 mm (36 in.) specified by the AASHTO LRFD Specifications[20]. The average transfer length of the conventional concrete mix #3 was determined to be 508 mm (20 in.), which is less than required by both bridge specifications.

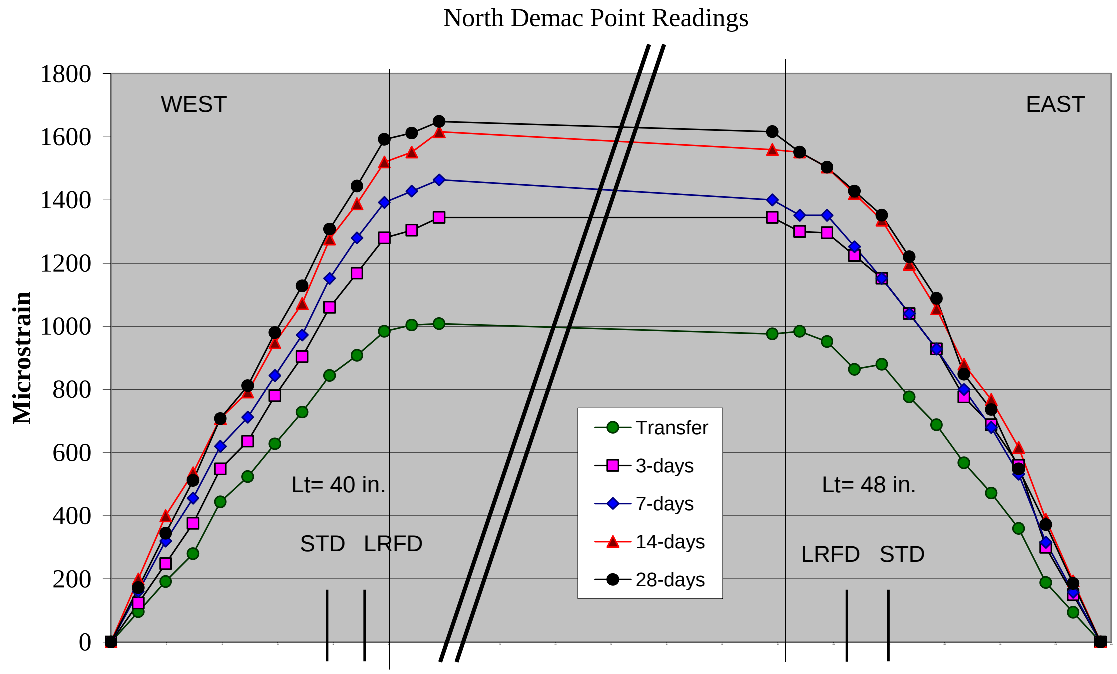


A) North Side of the Girder

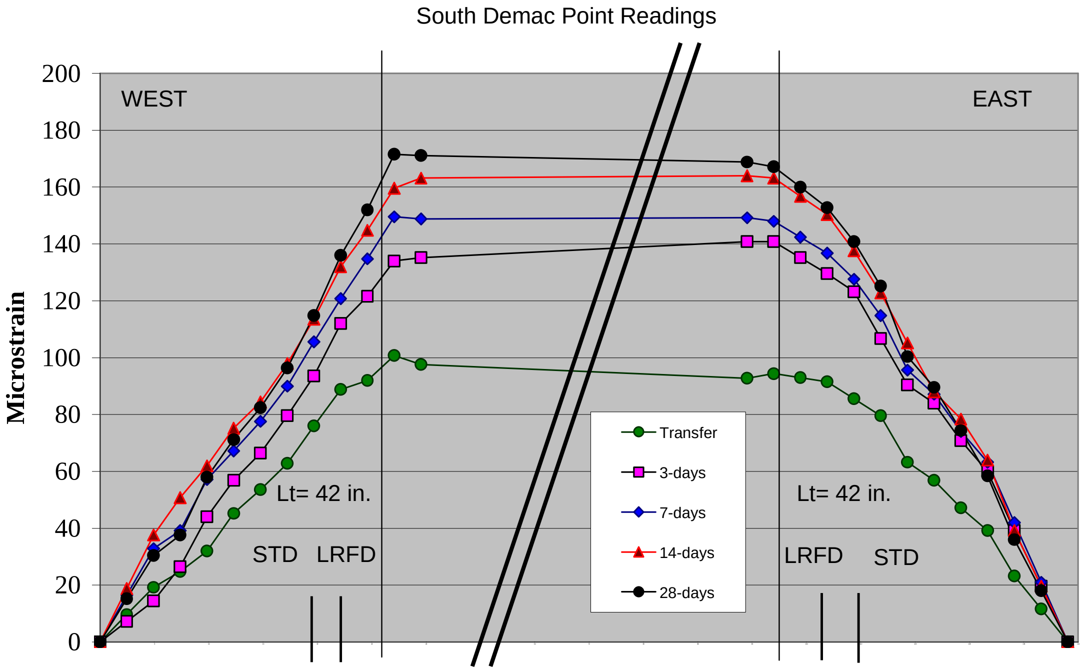


B) South Side of the Girder

Figure 3.4.1 Concrete Strain of Project I along the Girder Length

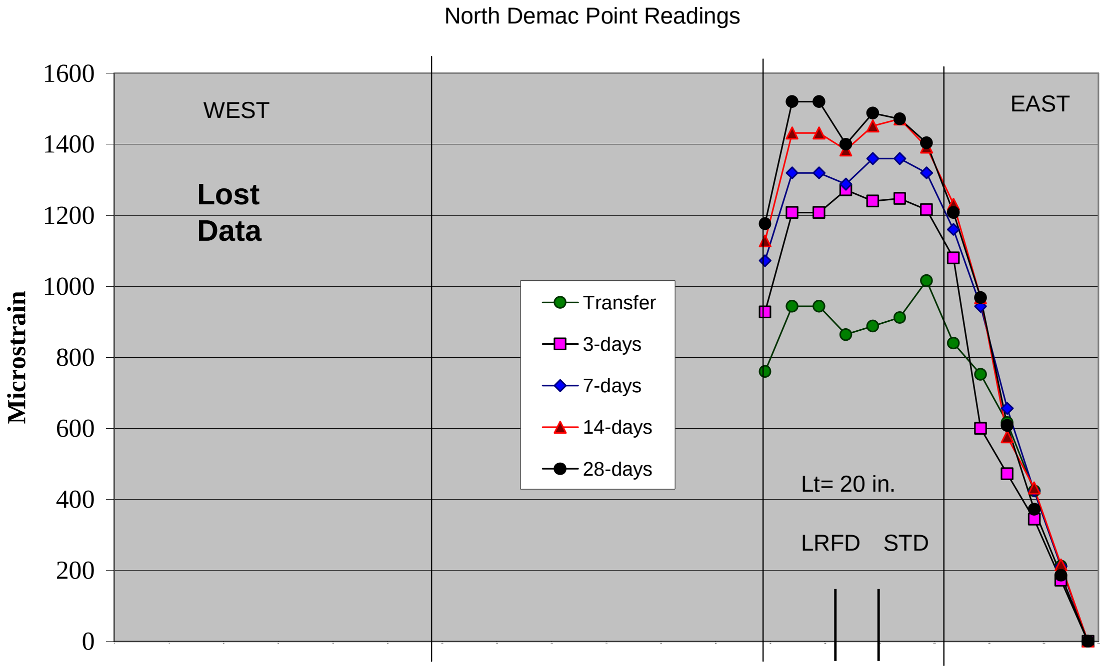


A) North Side of the Girder

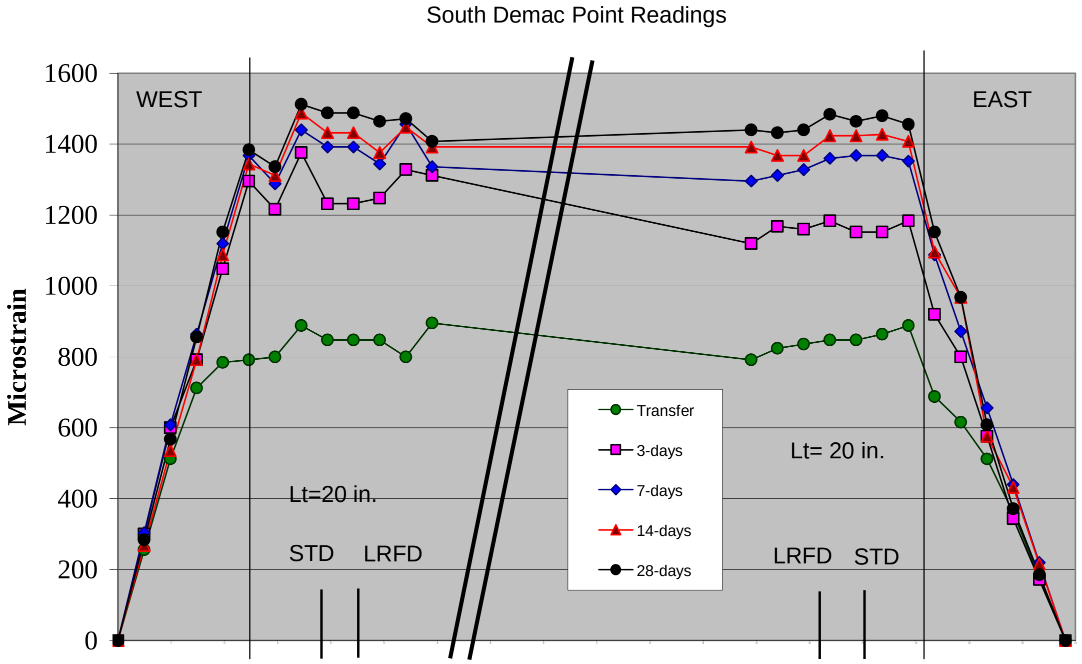


B) South Side of the Girder

Figure 3.4.2 Concrete Strain of Project II along the Girder Length



A) North Side of the Girder



B) South Side of the Girder

Figure 3.4.3 Concrete Strain of Project III along the Girder Length

The amount of force that is transferred to the concrete along the girder can also be estimated from the concrete strain plots.

3.5 MAXIMUM INITIAL CONCRETE STRAIN CALCULATIONS

Measured concrete strains were verified using elastic analysis of the section at the transfer length. Strain calculations were based on Eq. (3.5.1):

Predicted Concrete Strain[16] =
$$f_{pi}A_{ps}\left[\frac{1}{A_{tr-rel}}+\frac{e_{tr-rel}\cdot y_{tr-rel-C.G}}{I_{tr-rel}}\right]+\frac{M_d\, y_{tr-rel-C.G}}{I_{tr-rel}} \qquad \textbf{(3.5.1)}$$

The predicted and measured concrete strains at the strands' CG are compared in Table 3.5.1. It should be noted that errors are introduced when a uniaxial stress state in the concrete was assumed. The difference between measured and calculated concrete strains might be improved by conducting a three-dimensional analysis using finite element modeling.

Table 3.5.1 Maximum Concrete Strains due to Prestressing Release (Calculated vs. Measured)

Project	**Bridge Girder**	**Measured L_t (in.)**	**f_{pi} x A_{ps} (kips)**	**e_{tr_rel} (in.)**	**$y_{tr\_rel\_C.G}$ (in.)**	**A_{tr_rel} (in.^{2})**	**I_{tr_rel} (in.^{4})**	**M_g (kip.in)**
Project I	NU1100	38	703	6.34	2.00	711.21	**185,131**	**994**
Project II	NU900	46	1,494	8.50	2.75	686.50	**112,356**	**1459**
Project III	NU1350	21	1,673	4.29	3.90	791.91	**308,147**	**18132**

Project	**Predicted concrete strain* (Microstrain)**	**95% of the Measured Concrete Strain* (Microstrain)**	**% Differences**
Project I	354	494	28%
Project II	706	957	26%
Project III	448	845	47%

*At the prescreening strands' C.G. in the bottom flange at transfer length.

CHAPTER 4

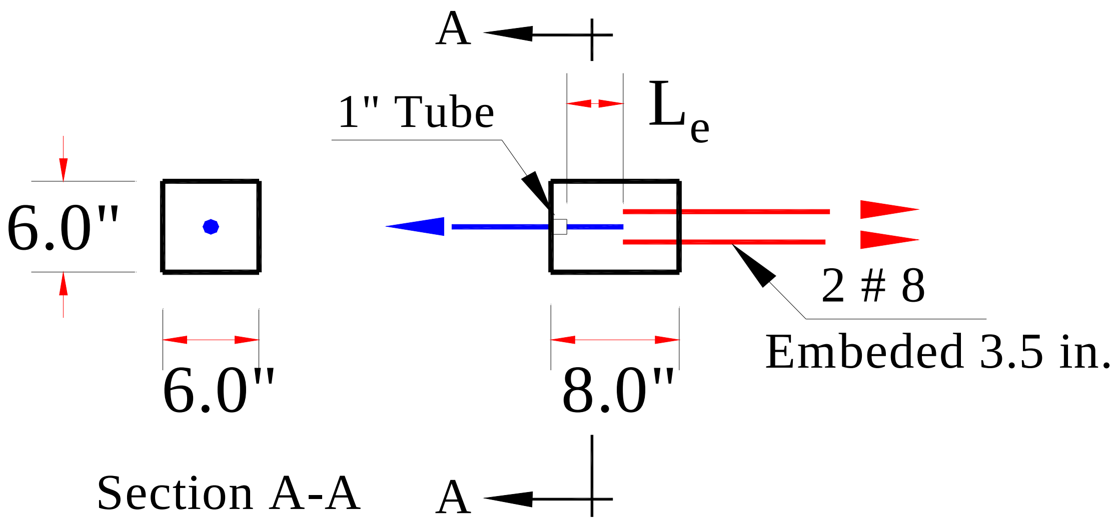
REINFORCING BARS PULL-OUT CAPACITY

4.1 OBJECTIVE

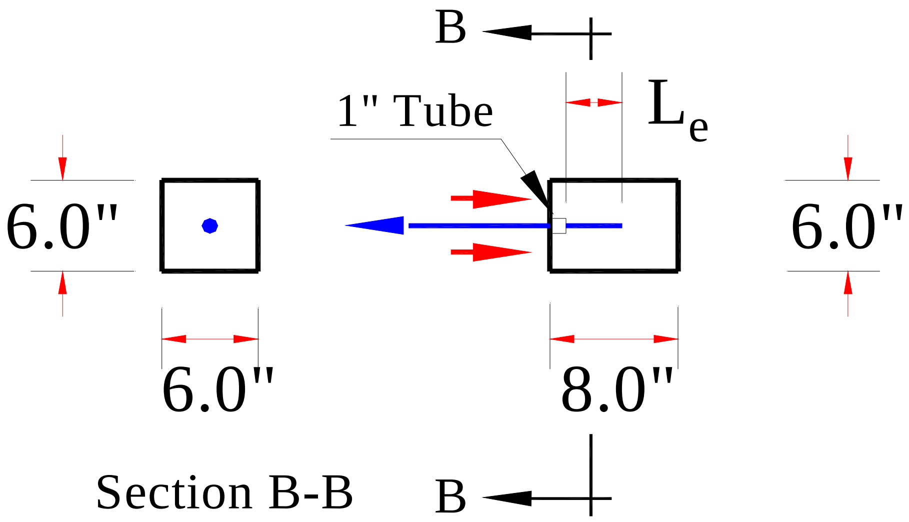
Small specimens were tested to compare the bond strength between SCC and conventional concrete using reinforcing bars and untensioned 0.6 in. prestressing strands.

4.2 PREVIOUS REINFORCING BAR PULL-OUT TESTS

Currently most researchers[7, 8, 9] conduct pull-out tests with short embedment lengths to closely simulate uniform bond stress. Concerns have been expressed, however, that these short embedment lengths would result in very high bond strength. Chapman and Shah[12] have developed a testing procedure that may be considered as a modified version of the Danish Standard[13]. Testing performed in References 7, 8, and 9 by applying a pull-out force on a bar, while supporting the specimen from the same side of the tested bar by bearing on the concrete. On the other hands testing performed in reference 12 by applying a pull-out force on a bar, while supporting the specimen from one embedded bar protruded from the other side of the specimen.



Method 1



Method 2

Figure 4.2.1 Small Pull-Out Tests

4.3 SMALL SPECIMEN PULL-OUT TESTS – DEFORMED BARS AND 0.6-IN. STRANDS

The small specimens tested in this study were intended for comparison purposes, both methods 1 and 2 were used in this study as previously shown in Figure 4.2.1. Pull-out tests were performed on 41 specimens using No. 4, No. 6, No. 8 bars and 0.6-in. strands. Eleven specimens contained standard No. 4, nine specimens contained standard No. 6, and ten specimens contained standard No. 8, Grade 60 deformed reinforcing bars. The rest of the specimens contained 0.6-in. diameter low-relaxation strands. Mix # 2 was used to cast the first 32 specimens, and mix #3 was used for the remaining nine specimens. The specimens were tested at 28 days from casting. The embedment lengths of the bars varied in the pull-out tests and are given in Table 4.3.1

Table 4.3.1 Small Pull-Out Specimens Embedment Lengths

Le	#4	#6	#8	0.6-in. Strands
1.5 in.	3.00 d_b	2.00 d_b	1.50 d_b	2.50 d_b
2.5 in	5.00 d_b	3.33 d_b	2.50 d_b	4.17 d_b
3.5 in	7.00 d_b	4.67 d_b	3.50 d_b	5.83 d_b

4.3.1 TEST SETUP

Two methods of applying the pull-out force were conducted. The first method was to apply a pull-out force on a bar, while supporting the specimen from two embedded No. 8 bars protruded from the other side of the specimen, as shown in method 1 in Figure 4.2.1. The second method was a standard pull-out test by applying a pull-out force on a bar, while supporting the specimen from the same side of the tested bar by bearing on the concrete, as shown in method 2 in Figure

4.2.1. These pull-out tests were conducted the bond strength among the concrete mixes. A pull-out force was recorded at the bond failure between the bar and the concrete. The loading rate of the pull-out force was approximately 4.45 kN (1 kip)/minute. The test setup is shown in Figure 4.3.1.

Figure 4.3.1 Pull-Out Test Setup – Method 1

4.3.2 TEST RESULTS

Bond strength results from the first method were compared to those from the second method. No significant difference between methods 1 and 2 was observed when reinforcing bars were used.

The bond strength results from method 1 was 20 % higher that those of method 2 when 06. in. strands were used as shown in Figure 4.3.2.1.

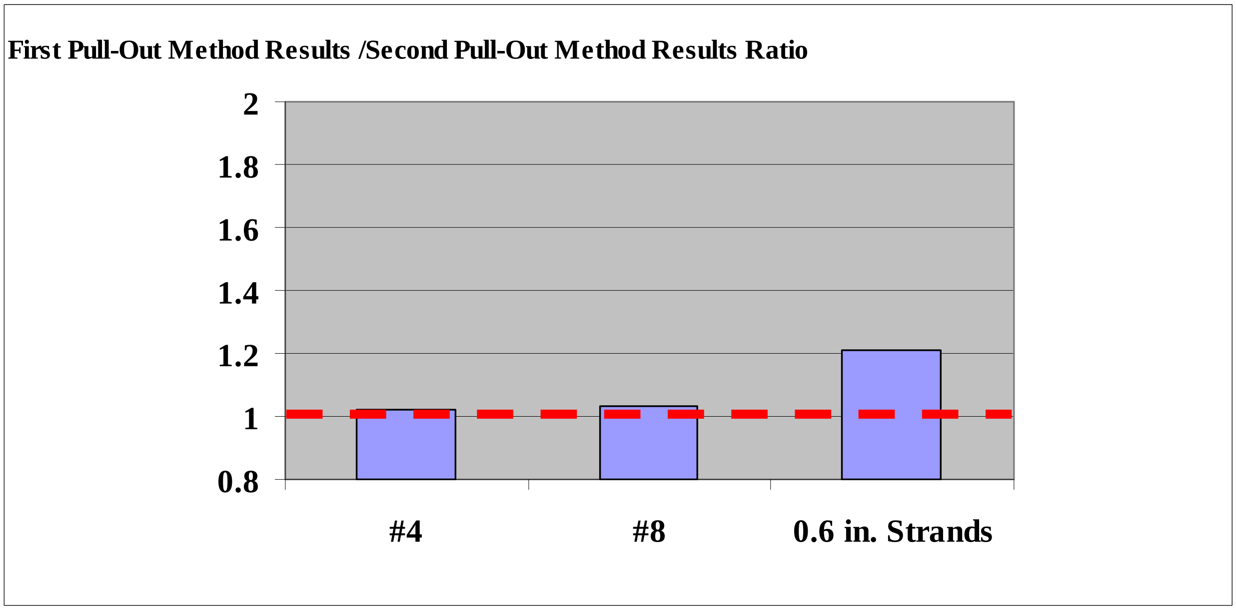


Figure 4.3.2.1. Method 1 to Method 2 Bond Strength Ratio at 28 days

Bond strength was computed by dividing the pull-out force by the product of the circumference of a reinforcing bar or a prestressing strand with the embedment length, as given in Eq.4.3.2.1. Bond strength results from the mix #2 were compared to the bond strength results from the mix #3, as shown in Figure 4.3.2.2. The comparisons were made for No. 4, No. 6, No. 8 and 0.6-in. strands. The bond strengths of the mix #2 and mix #3 are shown in Figures 4.3.2.3 and 4.3.2.4, respectively, for the various bar diameters. Figure 4.3.2.5 shows a typical specimen after bond failure in a pull-out test.

$$\textbf{Ultimate bond strength} = \frac{P_u}{(\Pi d_b)(L_e)} \qquad \textbf{(4.3.2.1)}$$

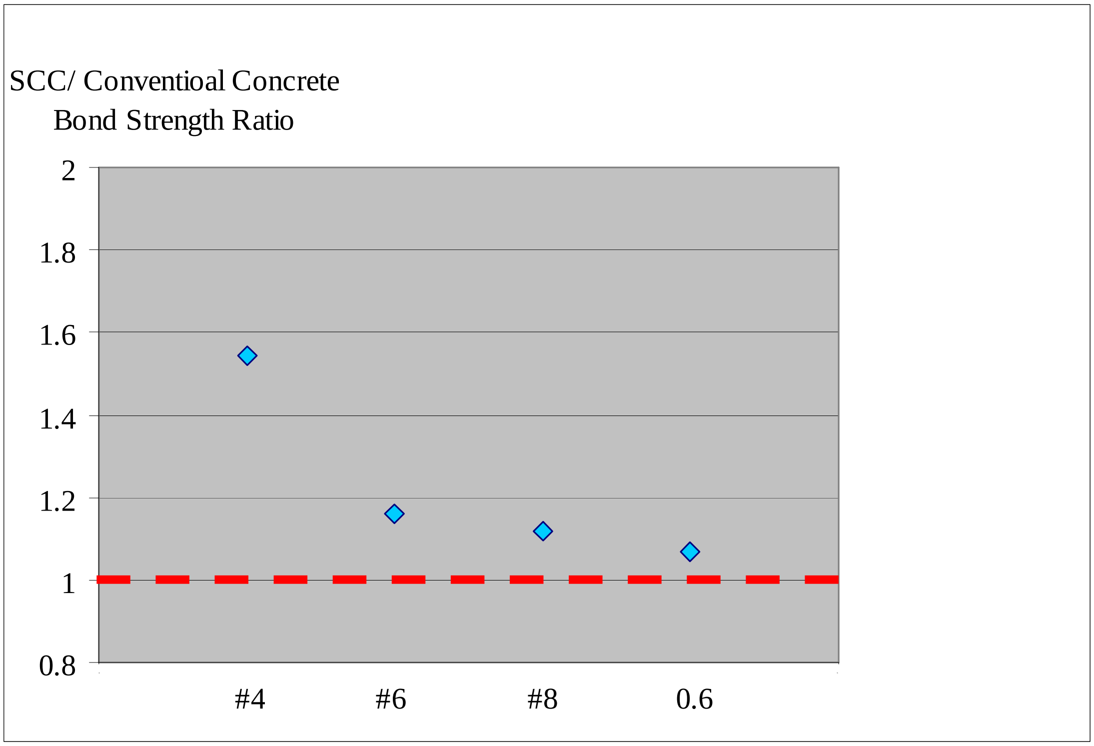


Figure 4.3.2.2 SCC Mix #2 to Conventional Concrete Mix #3 Bond Strength Ratio

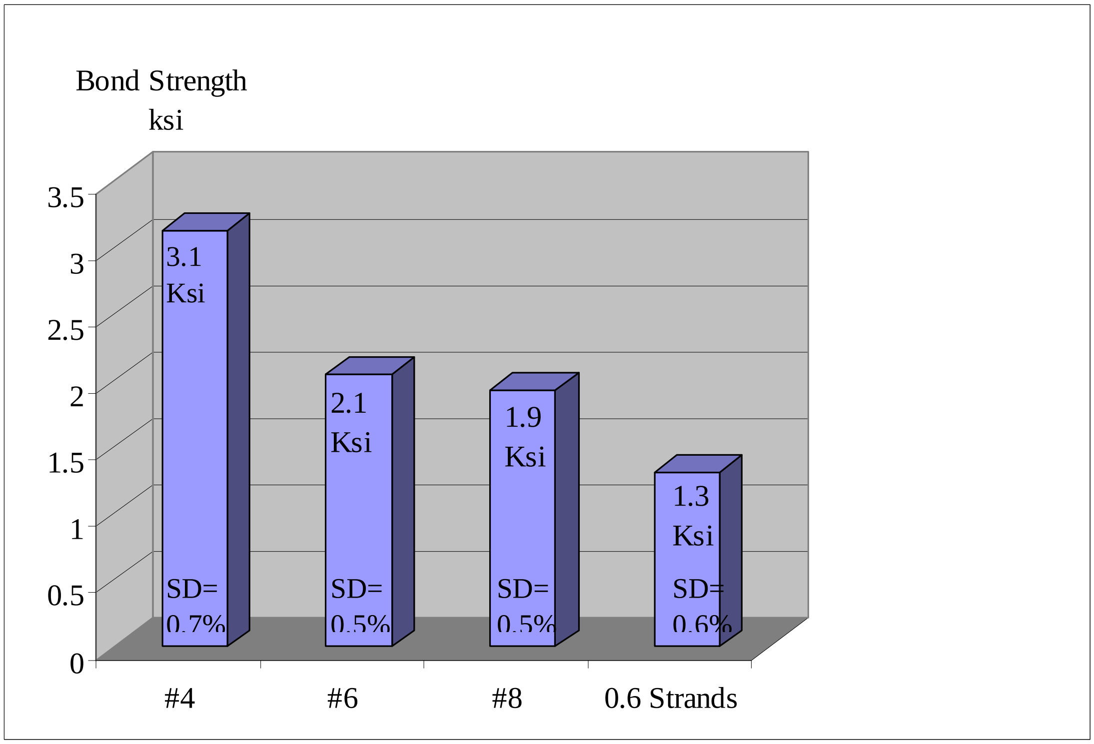


Figure 4.3.2.3 SCC Mix #2 28-day Bond Strengths

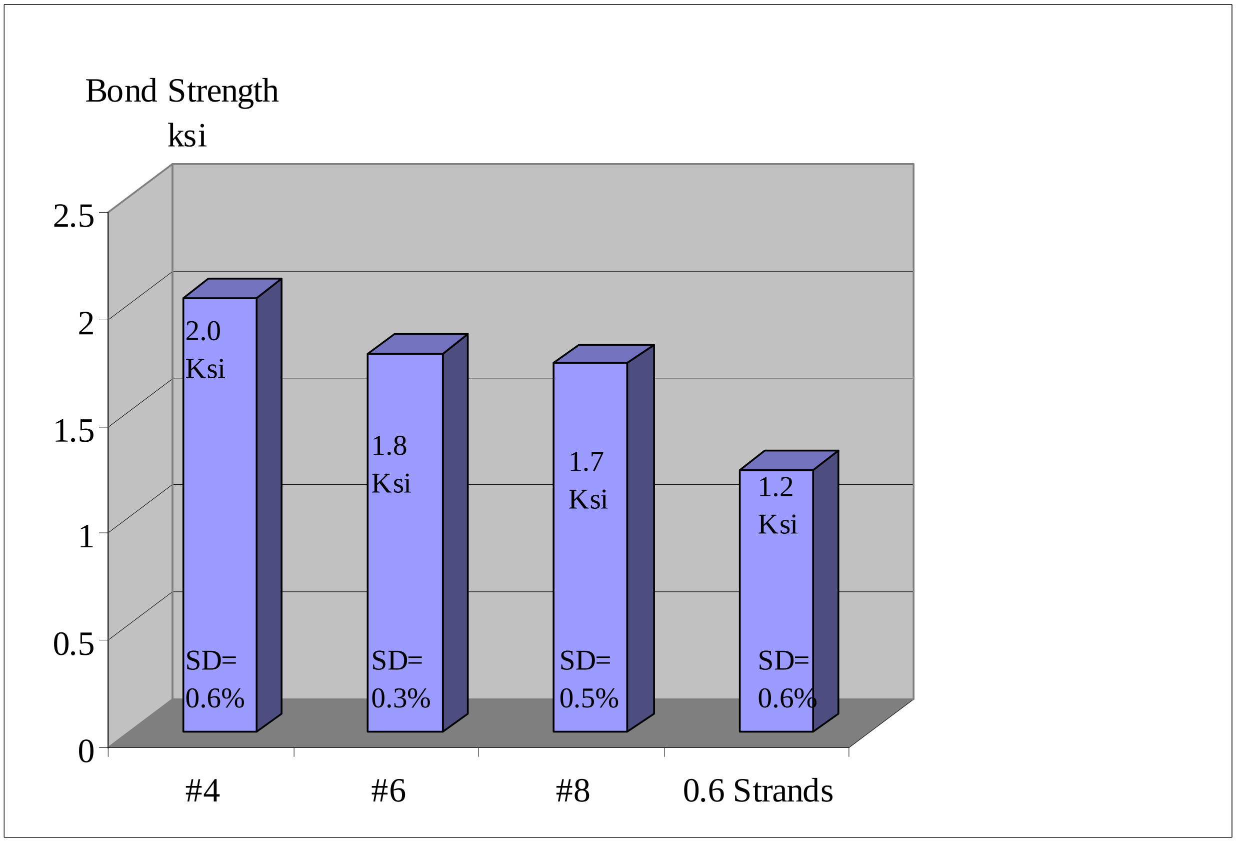


Figure 4.3.2.4 Conventional Concrete Mix #3 28-day Bond Strengths

Figure 4.3.2.5 Pull-Out Failure

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

Based on the experimental test results, the following conclusions can be made:

1. Limited test data has shown that the bond strength of SCC with deformed reinforcing bars was adequate. However, the use of VMA may adversely affect the early compressive strength and the bond strength of SCC with pre-tensioning strands. Further investigations into the SCC bond strength issues are warranted.
2. SCC mixtures may experience significantly longer transfer lengths than those of conventional concrete, more than 50% in some cases.
3. Moustafa pull-out tests failed to reveal any bond strength difference between the SCC and the conventional concrete with prestressing strands. A probable cause is that the three-dimensional stress state in a pretensioned concrete girder cannot be duplicated by the test.
4. Using Demec points for transfer length measurements has proved to be an efficient and accurate methodology.
5. Large-scale flexural tests using pretensioned concrete girders cast with SCC should be conducted to address development length issues.
6. SCC mixtures have lower early bond strength than conventional concrete, which causes greater transfer length in the SCC mixes than that of the conventional concrete.
7. SCC mixtures have higher bond strength than that of conventional concrete at 28 days, which causes less development length in the SCC mixes than the conventional concrete.
8. The smaller the deformed bar diameter, the higher the bond strength.

Based on the data that was presented in this report, it is recommended that when 0.6 in. stands and SCC are used, the following transfer length values are to be considered:

1. 80 times the diameter of the strand should be used in shear strength design.
2. 50 times the diameter of the strand should be used in calculating the concrete stresses at the end zone.

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NOTATION

L_t : Transfer length.

L_e : Embedment length.

P_u : Ultimate pull–out force.

d_b : Nominal diameter of reinforcing bar or prestressing strand’s circumference.

f_{pi} : Prestressing stress just before transfer.

A_{ps} : Area of prestressing strands.

e_{tr_rel} : Eccentricity of strands with respect to initial transformed section.

$y_{tr\_rel\_C.G}$: Distance from Neutral axis to the Prestressing strands C.G. of the I-girder bottom flange.

A_{tr_rel} : Area of transformed at transfer Length.

I_{tr_rel} : Moment of Inertia of transformed at transfer Length.

M_g : Moment due to girder self-weight at transfer Length.

SD : Standard Deviation.

LIST OF TABLES

LIST OF FIGURES