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Amgad F. Morgan Girgis
University of Nebraska-Lincoln

Christopher Y. Tuan
University of Nebraska Omaha, ctuan@unomaha.edu

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Bond Strength and Transfer Length of Pretensioned Bridge Girders Cast With Self-Consolidating Concrete



Amgad F. Morgan Girgis, Ph.D.

Research Assistant Professor
Department of Civil Engineering
University of Nebraska-Lincoln
Lincoln, Nebraska



Christopher Y. Tuan, Ph.D., S.E.

Associate Professor
Department of Civil Engineering
University of Nebraska-Lincoln
Lincoln, Nebraska

Self-consolidating concrete (SCC) is becoming increasingly popular in the precast/prestressed concrete industry in the United States. However, there have been concerns regarding the bond strength, transfer length, and development length of prestressing strands and mild steel reinforcement with SCC. Further, there are no design guidelines for using SCC. In this study, a literature survey on the bond strength of SCC was conducted. Moustafa pullout tests were performed to determine the bond strength of 0.6 in. (15.2 mm) pretensioning strands with SCC. The transfer lengths of three pretensioned concrete bridge girders were measured using Demec points. Pullout tests were also performed on 41 specimens using No. 4, No. 6, and No. 8 mild steel reinforcing bars and 0.6 in. prestressing strands. All the tests were performed using specimens cast with both SCC and conventional concrete. Test data have shown that the bond strength of SCC with deformed reinforcing bars is adequate. However, the use of viscosity-modifying admixtures in SCC may adversely affect its early compressive strength and its bond strength with pretensioning strands.

Over the last several years, self-consolidating concrete (SCC) has become increasingly more popular in the precast/prestressed industry in the United States. SCC may be defined as “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 The material has been used in many precast/prestressed concrete products, especially those with narrow forms and those requiring heavy reinforcement. SCC has been defined by its three principal characteristics:^{1,2}

- Flow 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.

Although SCC has become popular in the United States, there have been concerns regarding the bond strength, transfer length, and development length of prestressing strands and mild steel reinforcement contained in SCC. SCC contains admixtures that act as lubricants to enhance flowability, but the admixtures could also weaken the bond between the concrete and the reinforcement.

Very few studies have been conducted to evaluate the bond strength of reinforcement in SCC in the United States. As summarized in the literature review, some studies have reported SCC to have higher bond strengths than those of conventional concrete, while some data in the very same studies have also indicated inadequate early-age bond strength of SCC, which greatly affects the transfer length of reinforcement.

Furthermore, there are no guidelines for estimating 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 pretensioning strands and mild steel reinforcing bars, as compared to conventional concrete.

LITERATURE REVIEW

SCC was first developed in the late 1980s² by several 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 to place it in the field as regular concrete; labor savings are the main advantage of using SCC. SCC may be categorized into three types: (1) the powder type, which contains a high powder (fines) content; (2) the VMA type, which utilizes viscosity modifying admixtures (VMA); and (3) the combined type, which contains both powder and VMA.³

SCC requires a higher content of fine particles than conventional concrete to increase flowability and decrease segregation and bleeding. For example, conventional concrete typically is proportioned to contain about 38 percent fine particles, while SCC requires about 46 percent of its materials to be fine particles. The additional fine particle content is accomplished by replacing cement with materials that have

a lower specific gravity such as ground granulated blast-furnace slag and pozzolans (fly ash, silica fume, and calcined shale).⁴

Pullout tests on 0.5 and 0.8 in. (12 and 20 mm) diameter steel reinforcing bars were conducted at the University of Paisley in the United Kingdom.⁵ Results showed that the bond strength of SCC was about 18 to 38 percent higher than that of regular concrete. Chan et al.⁶ at 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 non-homogeneity in ordinary concrete was prevented with the use of SCC.

Investigations conducted in the United States consisted of pullout 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 Attigbe et al.,⁷ SCC yielded similar top-bar factors to those of normal concrete with 4 to 6 in. (102 to 152 mm) of slump. In a test using air-cured SCC and a VMA, the top-bar factor was actually lower than that of conventional concrete.

Attigbe et al.⁸ also concluded in another study, using both reinforcing bars and prestressing strands, that the highly stable characteristics of SCC 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.

Khayat⁹ reported top-bar factor improvement with the use of SCC, which was accredited to the reduction in bleeding and segregation. However, Fig. 4 in Khayat's paper shows that the top-bar bond strength of the SCC with reinforcing bars was actually lower than that of conventional concrete.

Based on extensive experimentation, Carrasquillo¹⁰ at the University of Texas at Austin also stated that "in no case was the pullout capacity of straight deformed bars embedded in superplasticized concrete significantly less than that of the bars embedded in the concrete containing no superplasticizer."

From the literature review, test results from the previous studies suggest that the bond strength of SCC with deformed

Table 1. Concrete mixtures used for the bridge girders.

Constituent Materials		Mix 1	Mix 2	Mix 3
Portland cement Type III		800 lb	632 lb	732 lb
Fly ash, Class C		150 lb	100 lb	—
Water		292 lb	292 lb	292 lb
<i>w/c</i>		0.31	0.40	0.40
½ in. limestone (SSD)		1282 lb	1311 lb	1350 lb
C33 Sand (SSD)		1417 lb	1449 lb	1460 lb
Air-entraining admixture		3 to 6 oz	3 to 6 oz	3 to 6 oz
Water reducing admixture	Retarding and water reducing	0 to 5 oz	0 to 5 oz	0 to 5 oz
	HRWR, 5-40 percent of water reducing	2 to 14 oz	2 to 14 oz	4 to 9 oz
	HRWR, 5-15 percent of water reducing	4 to 8 oz	—	4 to 8 oz
Viscosity modifying admixture		2 to 10 oz	2 to 10 oz	—

Notes: SSD = saturated surface dry condition; material quantities were specified per cubic yard of concrete; HRWR = high-range water reducer; 1 in. = 25.4 mm; 1 lb = 0.454 kg; 1 oz = 29.57 mL.

Table 2. Compressive strength of concrete mixtures at various ages (psi).

Time (days)	1	3	7	14	28
Mix 1 (self consolidating)	6492	7688	7978	8573	10887
Mix 2 (self consolidating)	5977	6738	7199	7506	8033
Mix 3 (conventional concrete)	6970	7527	8388	8755	9523

Note: 1 psi = 6.895 kPa.

Table 3. Flowability of the mixtures.

Concrete Mixture	Flowability Test	Results
Mix 1	Flow cone test	30 in. diameter
Mix 2	Flow cone test	26 in. diameter
Mix 3	Slump test	10 in. slump

Note: 1 in. = 25.4 mm.

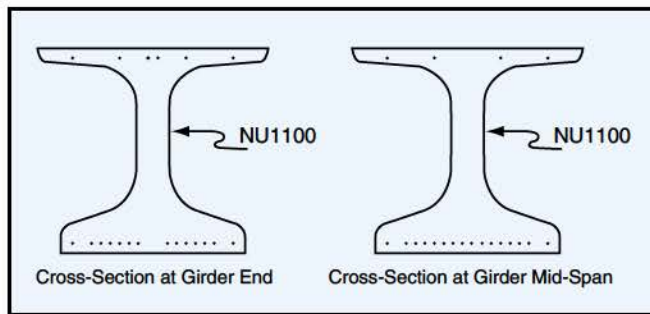


Fig. 1. Project I girder pretensioned strands scheme.

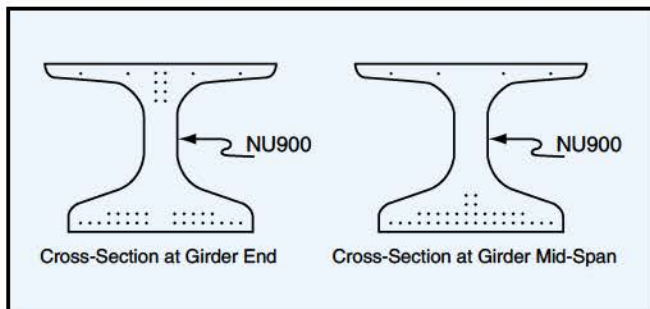


Fig. 2. Project II girder pretensioned strands scheme.

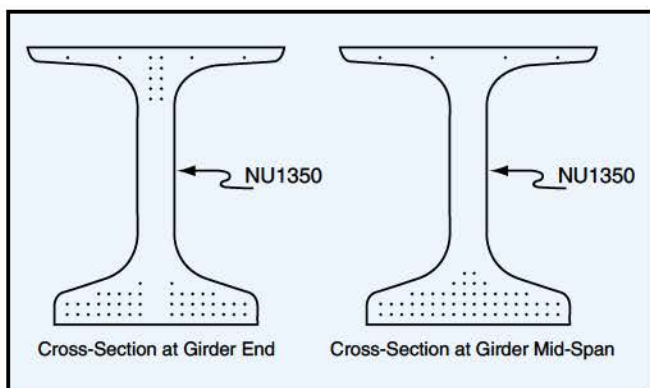


Fig. 3. Project III girder pretensioned strands scheme.

reinforcing bars is adequate. However, there have been no definitive test data to prove that the bond strength of SCC with prestressing strands is adequate.

DESCRIPTION OF BRIDGE PROJECTS

Three concrete mixtures were tested in this study. The first two (Mixes 1 and 2) were SCC mixtures, while the third (Mix 3) was a conventional concrete mixture. Table 1 gives the proportions of the three mixtures. Mix 3 was identical to Mix 2, except that Mix 3 contained no VMA and had a reduced amount of superplasticizer. Table 2 lists the compressive strengths of these mixtures with time, and Table 3 gives the flowabilities of these mixtures.

The prestressing strands used for the three bridge projects were from the same supplier. The strands had been pre-qualified by Moustafa pullout tests using the standard concrete mix prescribed by Logan.¹¹ For instance, the 0.6 in. (15.2 mm) diameter strand achieved an initial slip force of 32 kip (142 kN) and an ultimate pullout strength of 57 kip (254 kN), well exceeding the recommended minimum values of 21 and 48 kip (93 and 214 kN), respectively.

Project I

Project I is a three-span bridge, with span lengths of 72.5, 100, and 72.5 ft (22, 30, and 22 m), built with NU1100 I-girders. Girder depths are 43.3 in. (1100 mm) and web widths are 5.9 in. (150 mm). The bridge cross section consists of 14 girders spaced at 9.3 ft (2.8 m) on center, with an overall width of 126.7 ft (38.6 m). The deck consists of a composite, 7.5 in. (190 mm) thick cast-in-place concrete slab placed on the girders.

There are three girder segments per girder line. End segments are each 72.5 ft (22.1 m) long, and the field segment is 99.0 ft (30.2 m) long. These lengths allow for two splice joints. Mix 1 was used for the girders.

The girder tested in this study was a 72.5 ft (22.1 m) long end girder. As shown in Fig. 1, the girder has 14, 0.6 in. (15.2 mm) diameter straight strands at 2 in. (51 mm) spacing, two harped strands, and four top strands.

Project II

Project II is a 90.0 ft (27.4 m) long single-span bridge using NU900 I-girders. Girder depths are 35.4 in. (900 mm) and web widths are 5.9 in. (150 mm). The bridge cross section consists of six girders spaced at 8.0 ft (2.4 m) on center with an overall bridge width of 46.3 ft (14.1 m). The deck is a composite cast-in-place concrete slab, 7.5 in. (190 mm) thick.

Mix 2 was used for the girders. The girder tested in this study was a typical 90.2 ft (27.5 m) long girder. As shown in Fig. 2, the girder has 26, 0.6 in. (15.2 mm) diameter straight strands at 2 in. (51 mm) spacing, eight harped strands, and four top strands.

Project III

NU1350 I-girders were used in this project. Girder depths are 53.6 in. (1350 mm) and web widths are 5.9 in. (150 mm). Mix 3 was used for the girders of this bridge. The girder

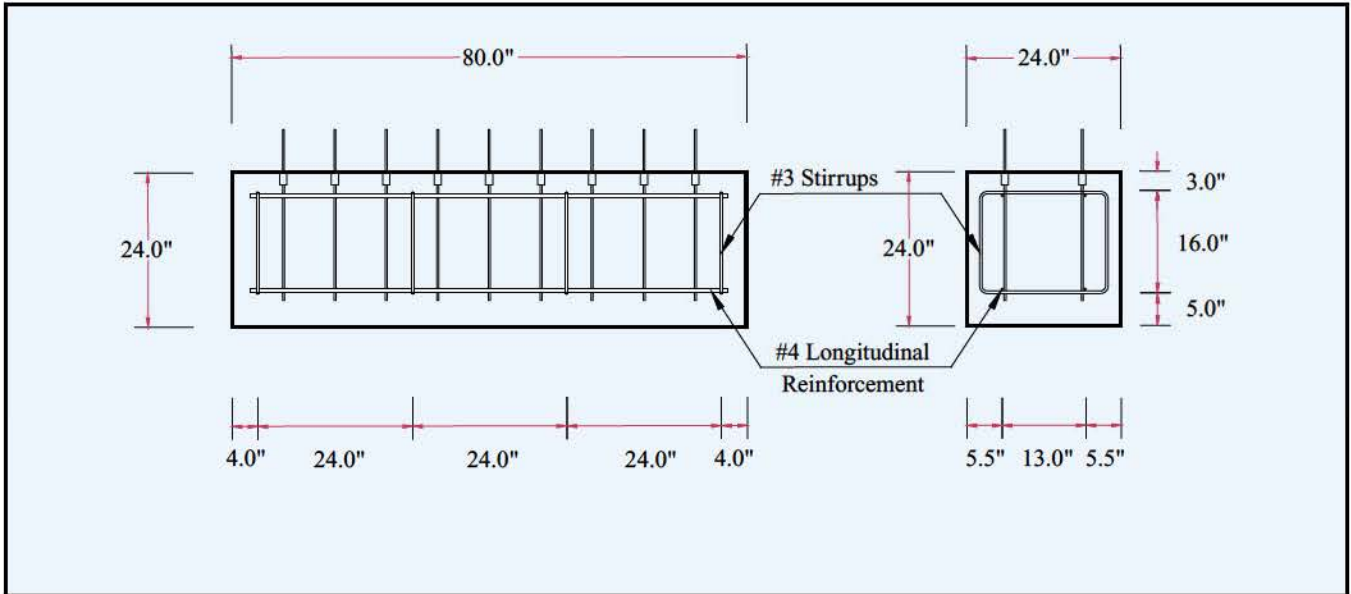


Fig. 4. Moustafa pullout test dimensions and reinforcement details. Note: 1 in. = 25.4 mm.

tested in this study was 124.0 ft (37.8 m) long. As shown in Fig. 3, the girder has 44, 0.5 in. (12.7 mm) diameter straight strands at 2 in. (51 mm) spacing, ten harped strands, and four top strands.

EXPERIMENTAL INVESTIGATIONS

To provide a basis for a comparison of bond strength with prestressing strands, Demec point readings were taken from the three bridge girders described previously to determine the transfer lengths. Moustafa pullout tests¹² were also conducted with the intent to confirm the transfer length measurements. Also, pullout tests on small specimens were conducted to de-

termine the bond strengths at 28 days and compared against the results from using the Moustafa pullout tests.

Moustafa Pullout Tests

Moustafa pullout tests were conducted to determine the bond capacity of 0.6 in. (15.2 mm) diameter low-relaxation, untensioned strands. The embedment length of all the strands tested was 18 in. (457 mm). Figs. 4 and 5 show the dimensions, reinforcement details, and strand layout of the test specimens. Fig. 6 shows the test setup. A central-hole hydraulic jack with a 110 kip (55 ton) capacity was used to pull out the strands. A load cell was placed at the top of the jack to record the pullout force. A steel plate and a chuck were

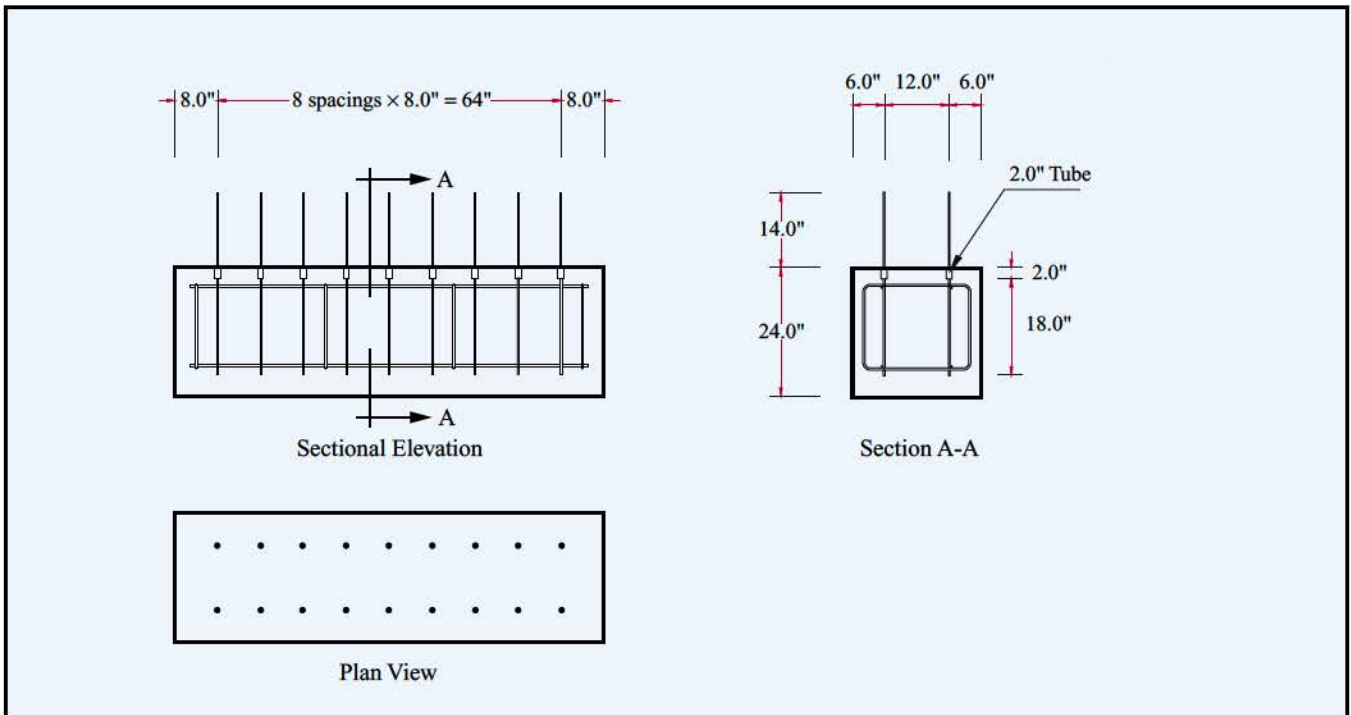


Fig. 5. Moustafa pullout test strands layout. Note: 1 in. = 25.4 mm.



Fig. 6. Moustafa pullout test setup.

placed at the top of the load cell. A steel frame was placed between the jack and the specimen.

Transfer Length Measurements

The transfer lengths of the prestressed bridge girders built with the SCC and the conventional concrete given in Table 1 were measured. A fast-setting, two-part epoxy was used to bond Demec points to the surface of the bottom flanges of the prestressed concrete girders to measure the concrete

Table 4. Small pullout specimens embedment lengths.

Embedment Length, L_e	No. 4 bars	No. 6 bars	No. 8 bars	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.	$7.00 d_b$	$4.67 d_b$	$3.50 d_b$	$5.83 d_b$

Note: 1 in. = 25.4 mm.

strains at release of the prestressing strands (see Fig. 7). The same detensioning sequence was followed for the three girders. Strands were jacked down first, then cut using a torch, starting from the outside inwards. Prestress symmetry was maintained by cutting strands from both sides of the girders.

Demec points are small stainless steel circular discs with a 0.039 in. (1 mm) pinhole at the center for precise distance measurements with a caliper (see Figs. 8 and 9). Concrete strains can be calculated from the changes in distance between Demec points.

Small Specimen Pullout Tests: Deformed Bars and Strands

A total of 41 small concrete specimens were subjected to pullout tests. Eleven specimens contained No. 4, nine specimens contained No. 6, and ten specimens contained No. 8, Grade 60 deformed reinforcing bars. The remainder of the

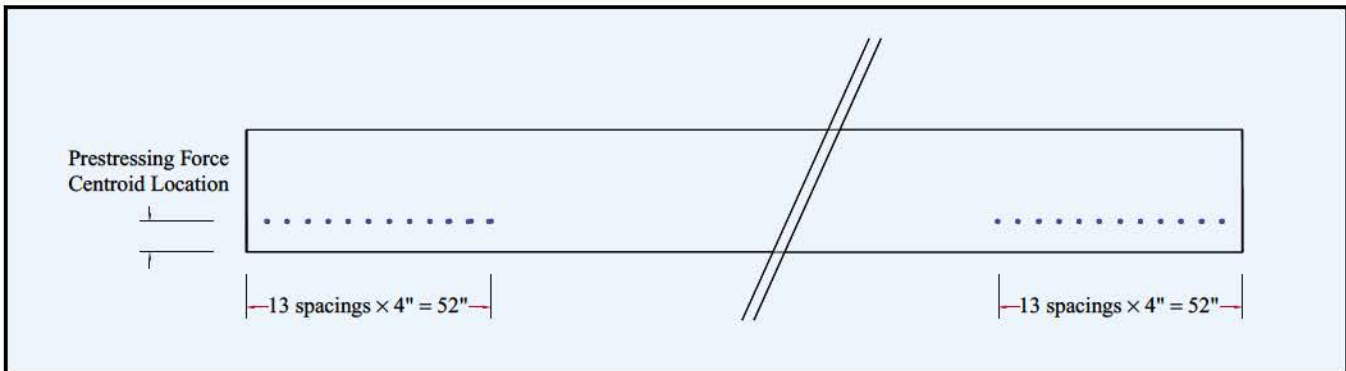


Fig. 7. Girder elevation showing locations of Demec points. Note: 1 in. = 25.4 mm.



Fig. 8. Demec point locations and beam end.



Fig. 9. Transfer length measurement.

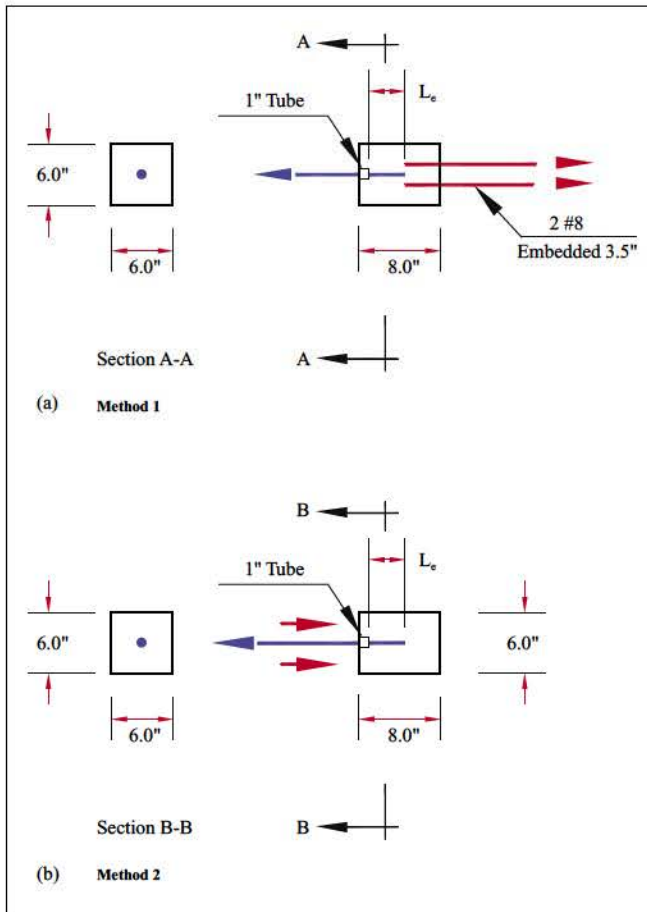


Fig. 10. Small pullout tests. Note: 1 in. = 25.4 mm.

specimens contained 0.6 in. (15.2 mm) diameter low-relaxation strands. Mix 2 was used to cast the first 32 specimens and Mix 3 was used for the remaining nine specimens. These specimens were tested 28 days after casting.

Test Specimen Details

Researchers^{7, 8, 9} typically conduct pullout 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 strengths. Chapman and Shah¹³ have developed a test procedure that may be considered to be a modified version of the Danish Standard.¹⁴

The small specimens tested in this study were intended for comparison purposes, as shown in Fig. 10. The embedment lengths of the bars tested were varied as given in Table 4.

Test Setup

Two methods of applying the pullout force were conducted. The first method was to apply a pullout force on a bar, while supporting the specimen from two embedded No. 8 bars protruding from the other side of the specimen, as shown in the Figs. 10(a) and 11. The second method was a standard pullout test by applying a pullout force on a bar, while supporting the specimen from the same side of the tested bar by bearing on the concrete, as shown in the Fig. 10(b). These pullout tests were carried out to compare the ultimate bond strengths among the different concrete mixtures.

A pullout force was recorded at bond failure between the



Fig. 11. Pullout test setup – Method 1.

bar and the concrete. The loading rate of the pullout force was approximately 1 kip (4.45 kN) per minute. This method utilizes the same way of applying force as the Moustafa pullout test does; however, the embedded length is much shorter (which yields a better average bond strength). The specimen size is also much smaller in order to eliminate the confinement effect and to better represent the I-beam, open trapezoidal, or box girder web widths.

TEST RESULTS

Moustafa Pullout Tests

Fig. 12 shows the test results from Mix 1; the average pullout strength was 43.4 kip (193 kN) with a 2.8 kip (12.5 kN) standard deviation. When compared with data in the literature, the average pullout strength is almost equal to the Moustafa pullout test benchmark,¹¹ but lower than the results from Barnes' 1999 report.^{15, 16} The Moustafa pullout test benchmark is a pullout force of 36.0 kip (160 kN) for 0.5 in. (12.7 mm) diameter strands, and this value was scaled up with respect to the strand diameter for 0.6 in. (15.2 mm) diameter strands, thus the calculated value of 43.2 kip (192 kN). The pullout forces obtained from Barnes' report have been interpolated to obtain values corresponding to a concrete strength of 7688 psi (53 MPa).

Fig. 13 shows the test results from Mix 2; the average pullout strength after one day was 54.2 kip (241 kN) with a 5 kip

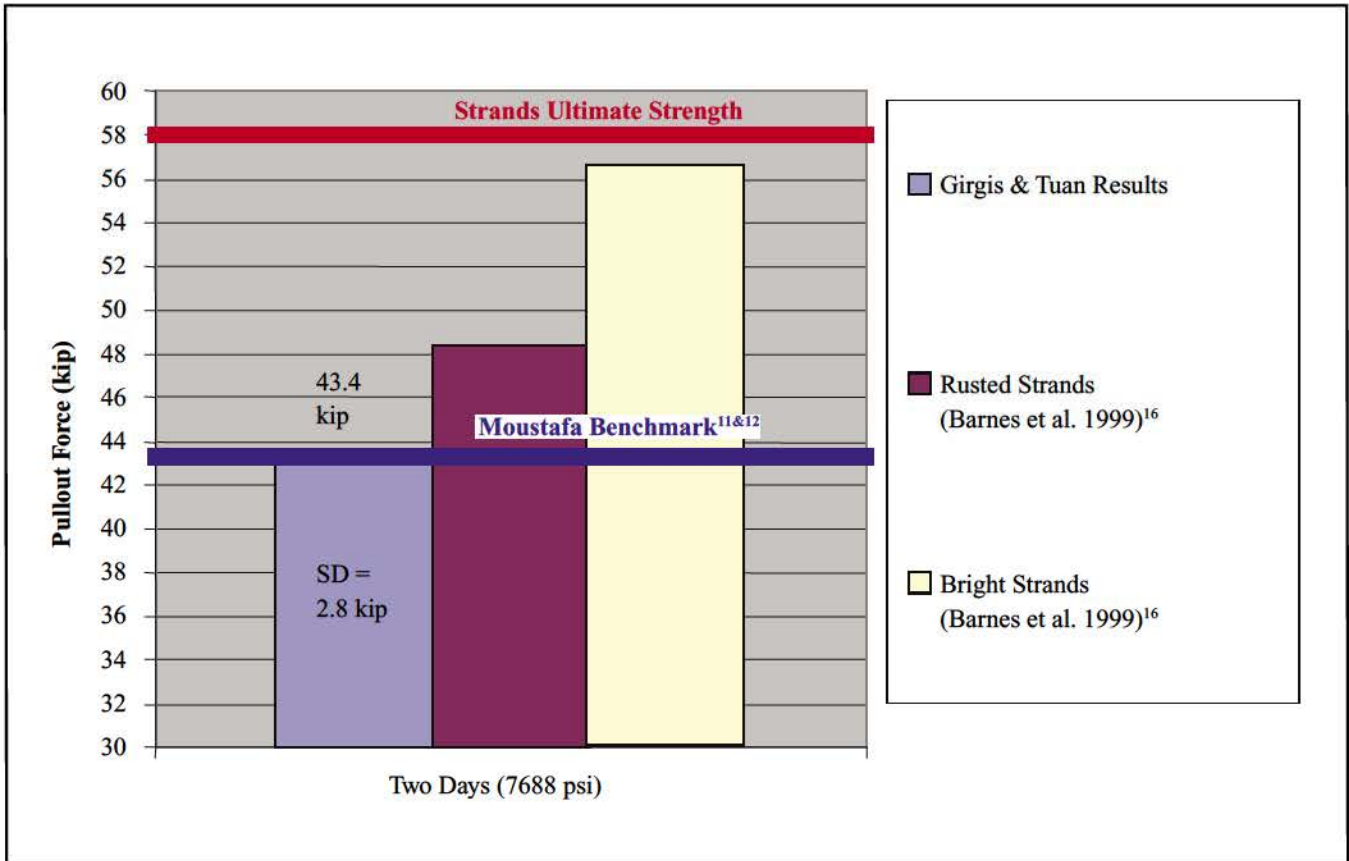


Fig. 12. Mix 1 pullout capacity versus data from literature. Note: 1 psi = 6.895 kPa; 1 kip = 4.448 kN.

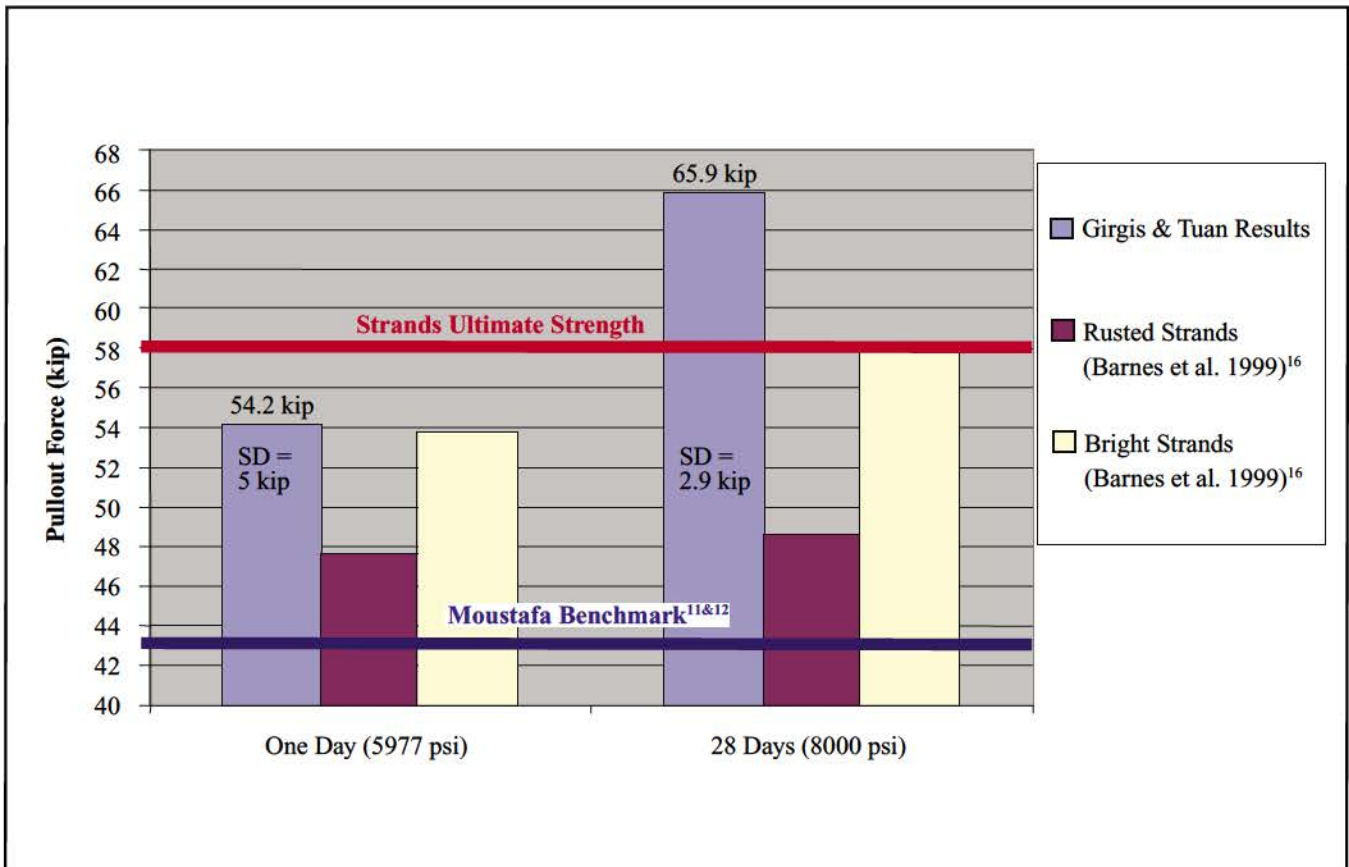


Fig. 13. Mix 2 pullout capacity versus data from literature. Note: 1 psi = 6.895 kPa; 1 kip = 4.448 kN.

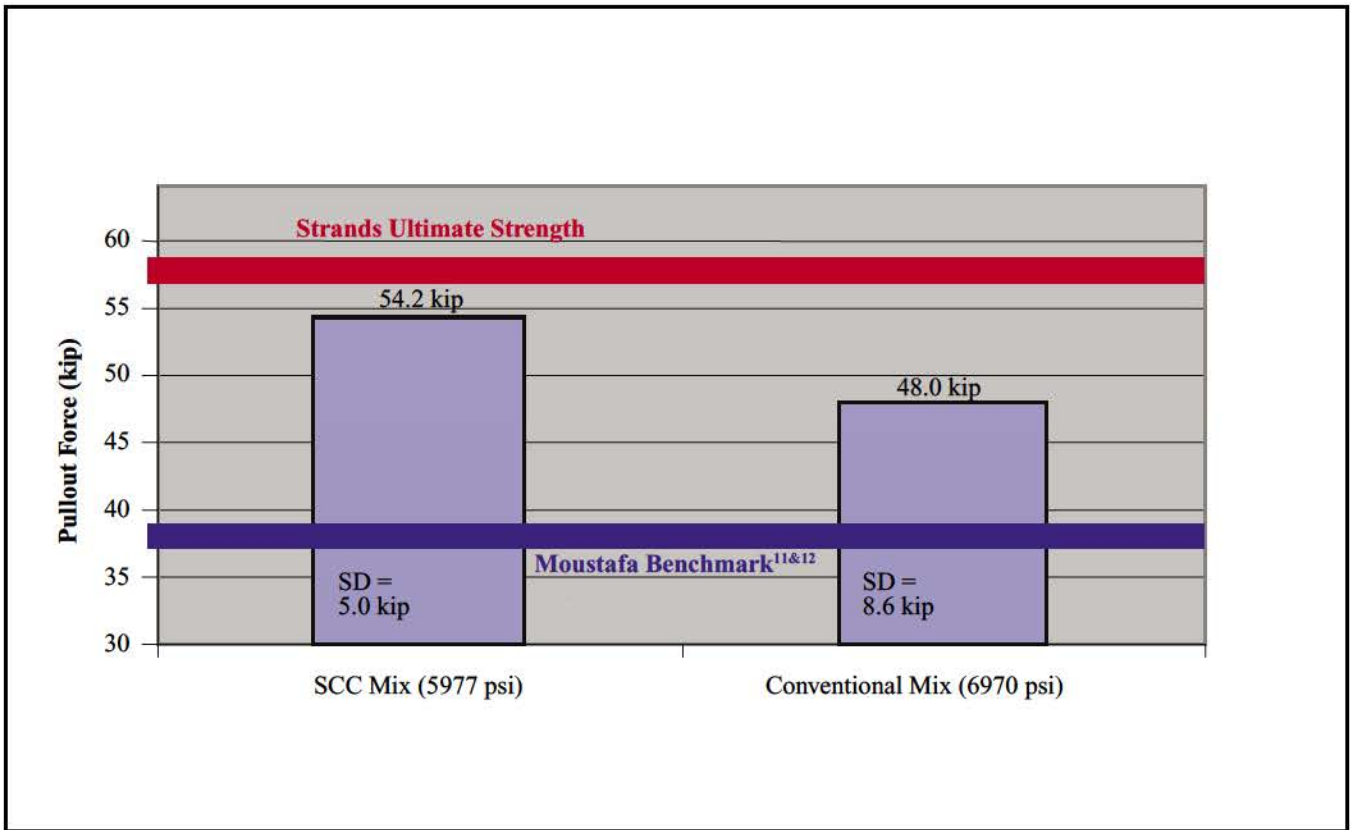


Fig. 14. Comparison of pullout capacities of Mix 2 (SCC) versus Mix 3 (conventional concrete) one day after casting. Note: 1 psi = 6.895 kPa; 1 kip = 4.448 kN.

(22.2 kN) standard deviation, and 65.9 kip (293 kN) at 28 days with a 2.9 kip (12.9 kN) standard deviation. When compared with data in the literature, the average pullout strength of the second SCC mix is greater than both the Moustafa pullout test benchmark¹¹ and the Barnes' 1999 results.^{15, 16} Test results from Mix 3 (conventional concrete) are compared against those from Mix 2 in Fig. 14.

Transfer Length Measurements

Demec point readings were taken before and after releasing the prestressing force, and at 3, 7, 14, and 28 days after casting the concrete, using those data taken before prestress release as a baseline. Distances between Demec points were measured using a caliper gauge; the change in this distance was used to calculate the strain in the concrete at the centroid of the bottom strands. Concrete strains along the centroid of the strands were then plotted along the length of the girder.

The concrete strains are zero at the girder ends and increase from the girder end until they become stable, at which point all prestressing forces are transferred to the concrete. As suggested by Russell,¹⁷ Buckner,¹⁸ and Lane,¹⁹ the transfer length can be determined by measuring the distance from the end of the girder to the point where 95 percent of the maximum concrete strain is measured.

Demec points were mounted on both sides and both ends of the bottom flanges of the three bridge girders. Figs. 15 through 17 present the concrete strain variations along the girder bottom flange for Projects I, II, and III, respectively. By averaging data obtained from the four corners of each

of the girders, the average transfer lengths of Mixes 1 and 2 were determined to be 36 and 43 in. (914 and 1092 mm), respectively. These values are longer than the transfer lengths specified by ACI 318²⁰ and the AASHTO Standard Bridge Specifications,²¹ about 50 strand diameters or 30 in. (762 mm).

Mix 2 has a transfer length greater than the 60 strand diameters, or 36 in. (914 mm), specified by the AASHTO LRFD Specifications.²² The average transfer length of the conventional concrete (Mix 3) was determined to be 20 in. (508 mm), which is less than that required by both the AASHTO Specifications and ACI 318. Mix 3 had a higher early compressive strength compared to that of Mix 2. As discussed by Barnes,¹⁶ Eq. (1) has been used to account for the effects of different compressive strengths at release and different strand diameters on the transfer length:

$$L_t = \alpha \frac{f_{pt}}{\sqrt{f'_{ci}}} d_b \quad (1)$$

where

- L_t = transfer length of prestressing strand
- α = proportionality constant
- f_{pt} = stress in prestressing strand immediately after release
- f'_{ci} = specified compressive strength of concrete at time of initial prestress
- d_b = diameter of prestressing strand

The calculated stresses in the prestressing strands immediately after release were virtually the same for both

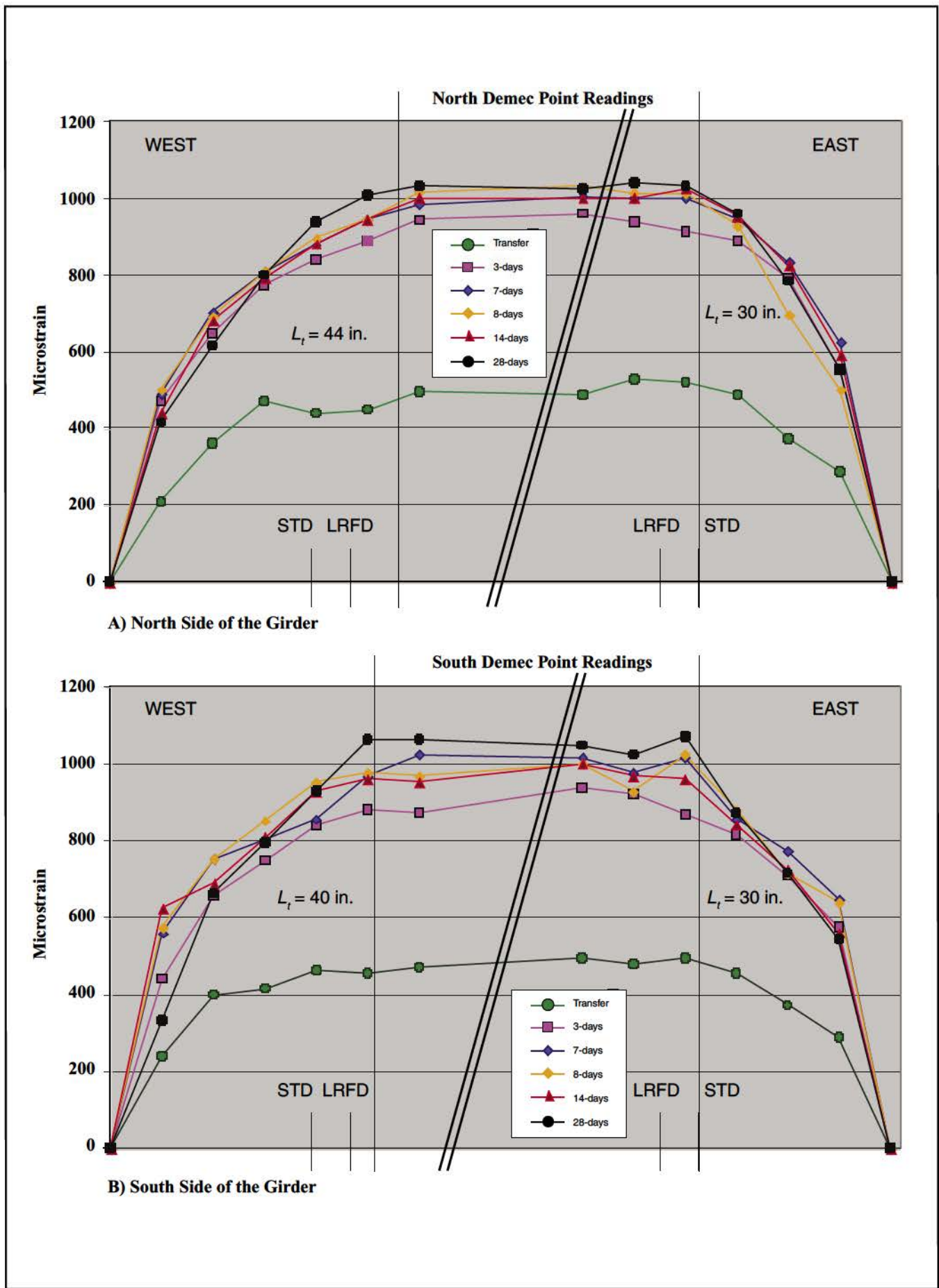
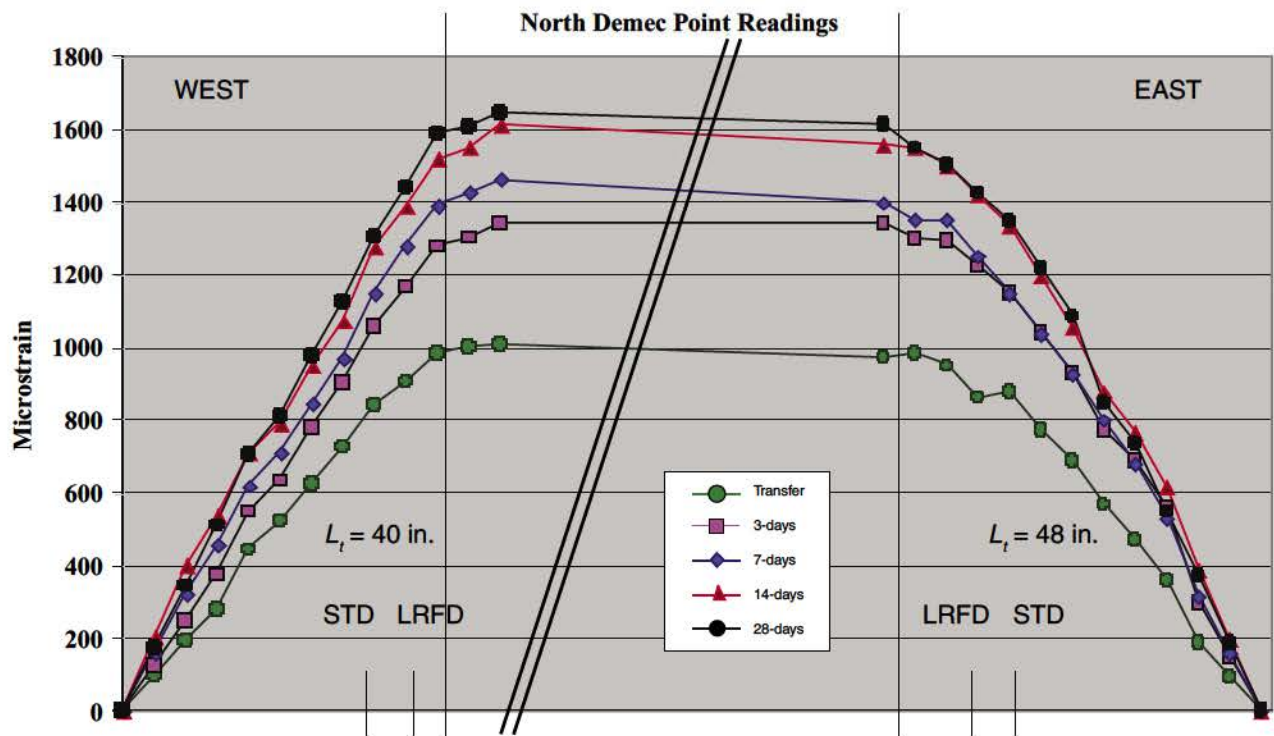
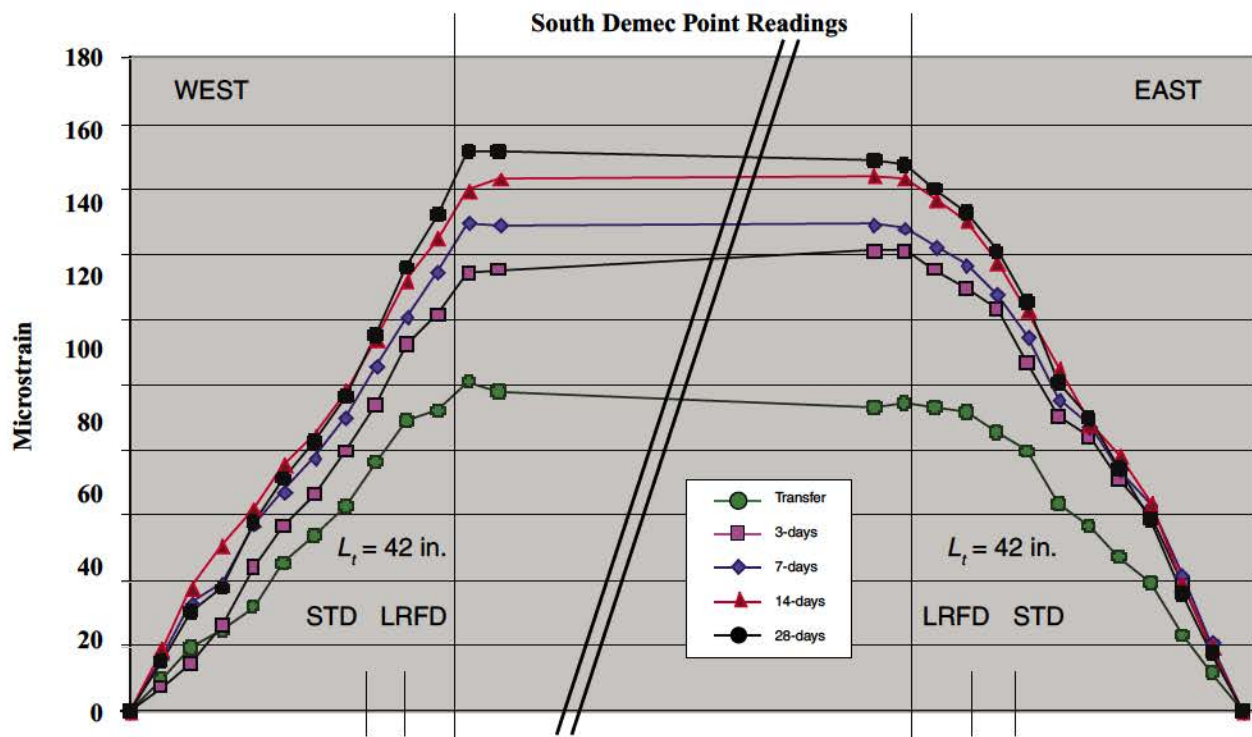


Fig. 15. Concrete strain of Project I along the girder length. Note: 1 in. = 25.4 mm.

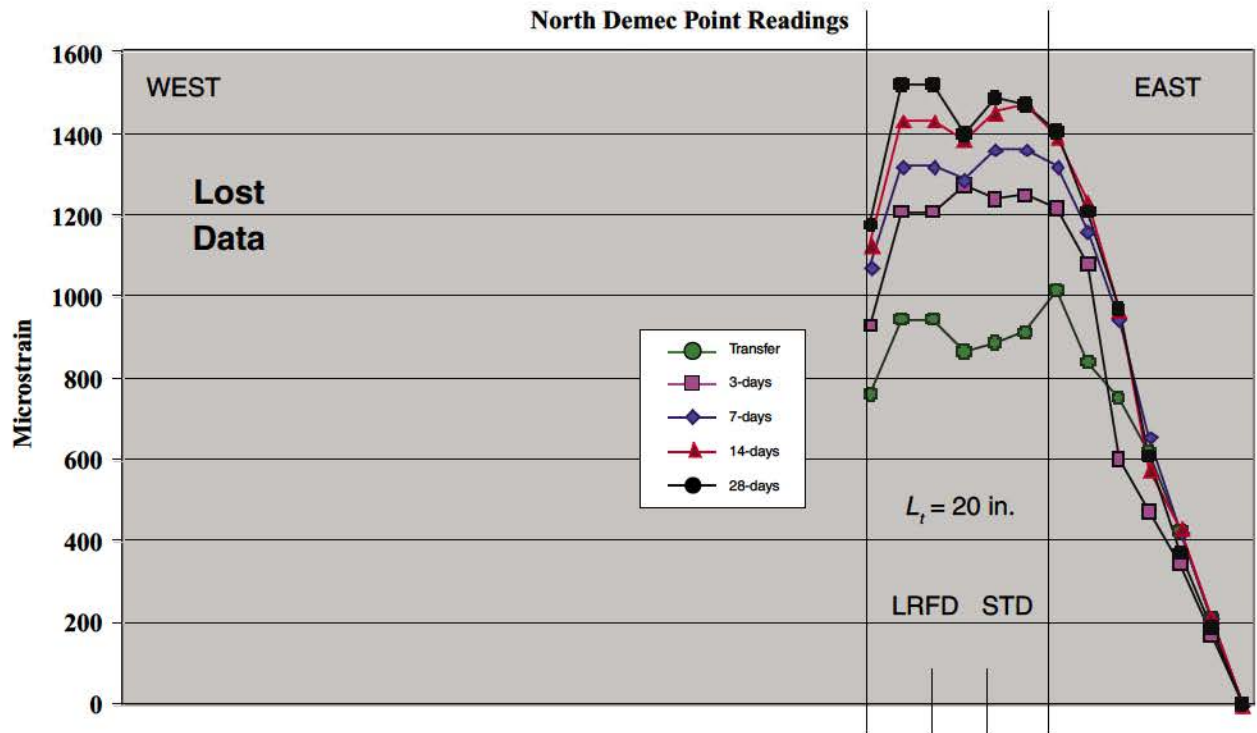


A) North Side of the Girder

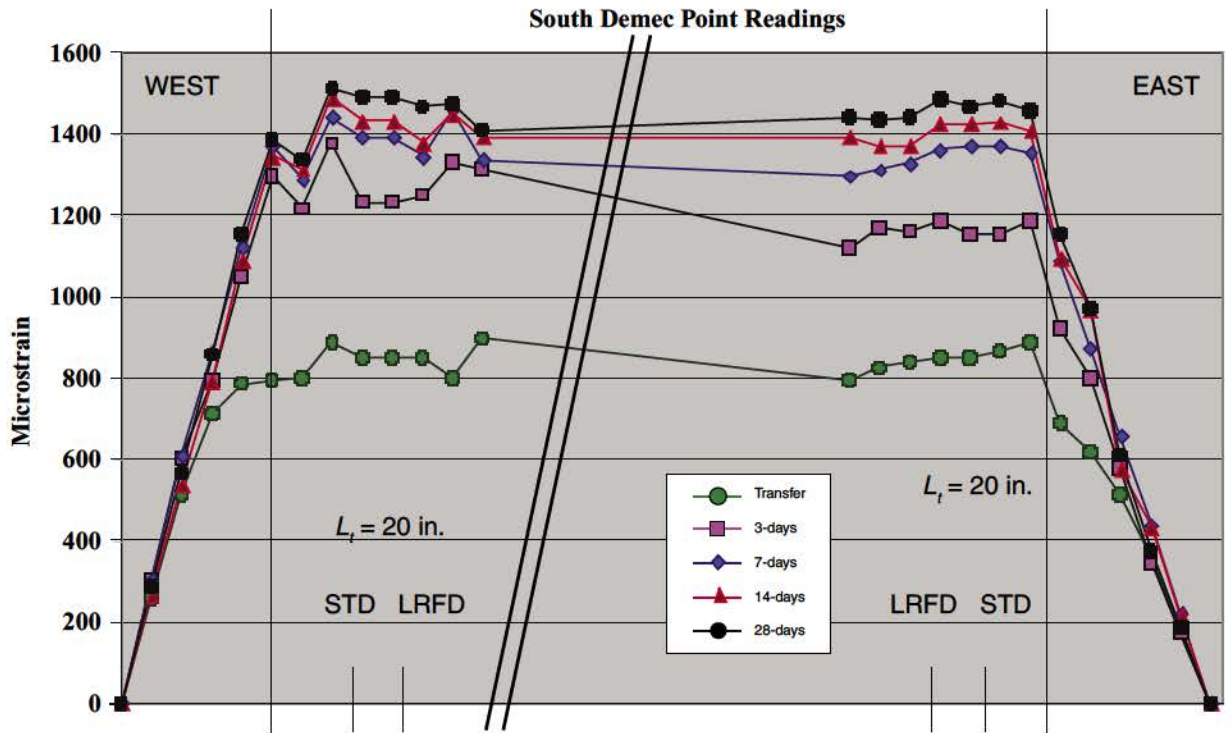


B) South Side of the Girder

Fig. 16. Concrete strain of Project II along the girder length. Note: 1 in. = 25.4 mm.



A) North Side of the Girder



B) South Side of the Girder

Fig. 17. Concrete strain of Project III along the girder length. Note: 1 in. = 25.4 mm.

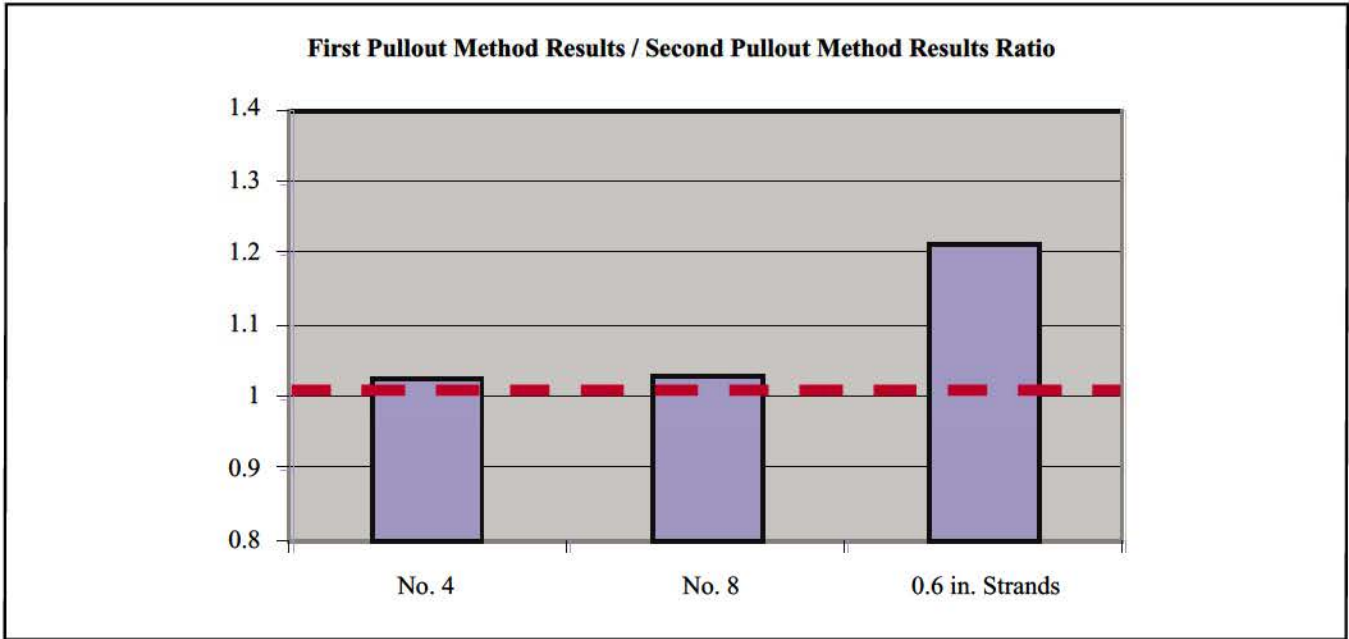


Fig. 18. Method 1 to Method 2 bond strength ratio. Note: 1 in. = 25.4 mm.

Projects II and III. The calculated transfer length of Project II was 30 percent greater than that of Project III, but is much less than the measured value. The calculated ratio between the transfer lengths is compared to the ratio of measured transfer lengths L_t , as follows:

$$\frac{L_t [\text{Project II}]}{L_t [\text{Project III}]} = \frac{0.6\sqrt{6970}}{0.5\sqrt{5977}}$$

$$= 1.30 < \frac{L_t \text{ measured} = 43.0 \text{ in.}}{L_t \text{ measured} = 20.0 \text{ in.}} = 2.15$$

The bottom flanges of the NU-I girders in all three projects have the same cross-sectional properties and very similar reinforcement details. Consequently, any shape or confinement effect on the transfer length should be negligible. The only differences among the girders in the three projects were the section depth and the number of strands, which could be sig-

nificant parameters, but are not included in the transfer length formulas specified by ACI and AASHTO.

The amount of force that is transferred to the concrete along the girder can also be estimated from the concrete strain plots. Short transfer lengths may cause excessive concrete stress at transfer and may result in splitting or bursting cracks in the girder end zone. Long transfer lengths may reduce girder shear resistance and imply long development lengths, which may adversely affect the flexural strength of the girder.

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. (2):¹⁶

$$e_c = f_{pi} A_{ps} \left[\frac{1}{A_{tr-rel} E_c} + \frac{e_{tr-rel} y_{tr-rel-C.G.}}{I_{tr-rel} E_c} \right] - \frac{M_g y_{tr-rel-C.G.}}{I_{tr-rel} E_c} \quad (2)$$

where

Table 5. Maximum concrete strains due to prestressing release (calculated versus measured) at measured transfer length location.

Project	Bridge Girder	Measured L_t (in.)	$f_{pi} A_{ps}$ (kip)	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	1494	8.50	2.75	686.50	112,356	1459
Project III	NU1350	21	1673	4.29	3.90	791.91	308,147	18,132

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

*At the prestressing strands' centroid in the bottom flange at the measured transfer length.

Note: 1 in. = 25.4 mm; 1 kip = 4.448 kN.

**SCC / Conventional Concrete
Bond Strength Ratio**

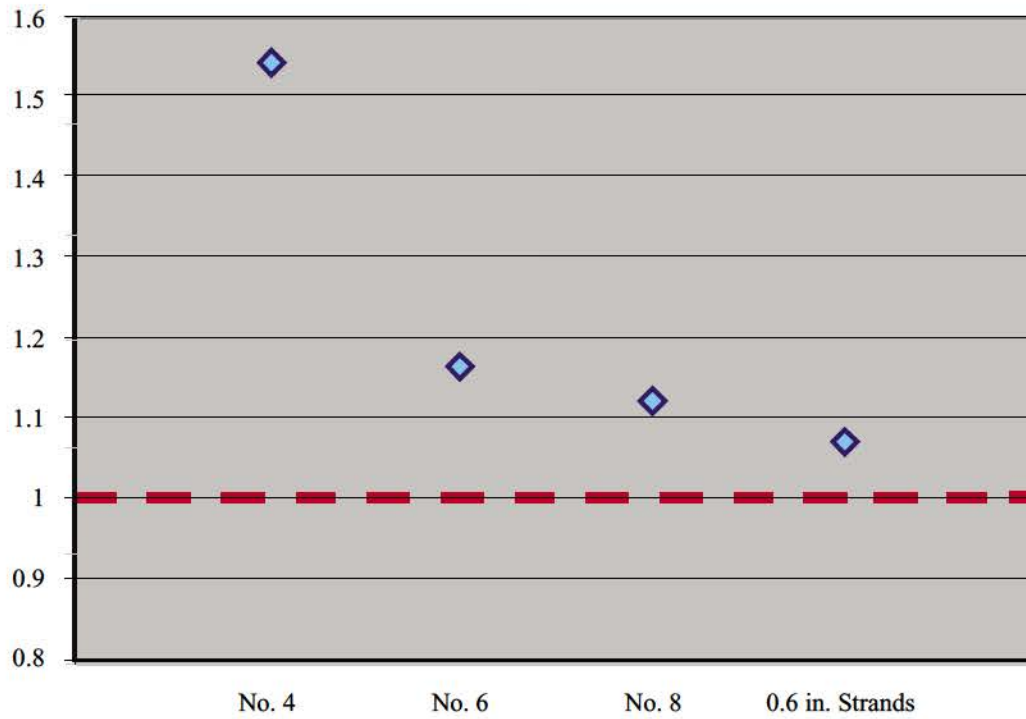


Fig. 19. SCC (Mix 2) to conventional concrete (Mix 3) bond strength ratio at 28 days. Note: 1 in. = 25.4 mm.

**Bond Strength
ksi**

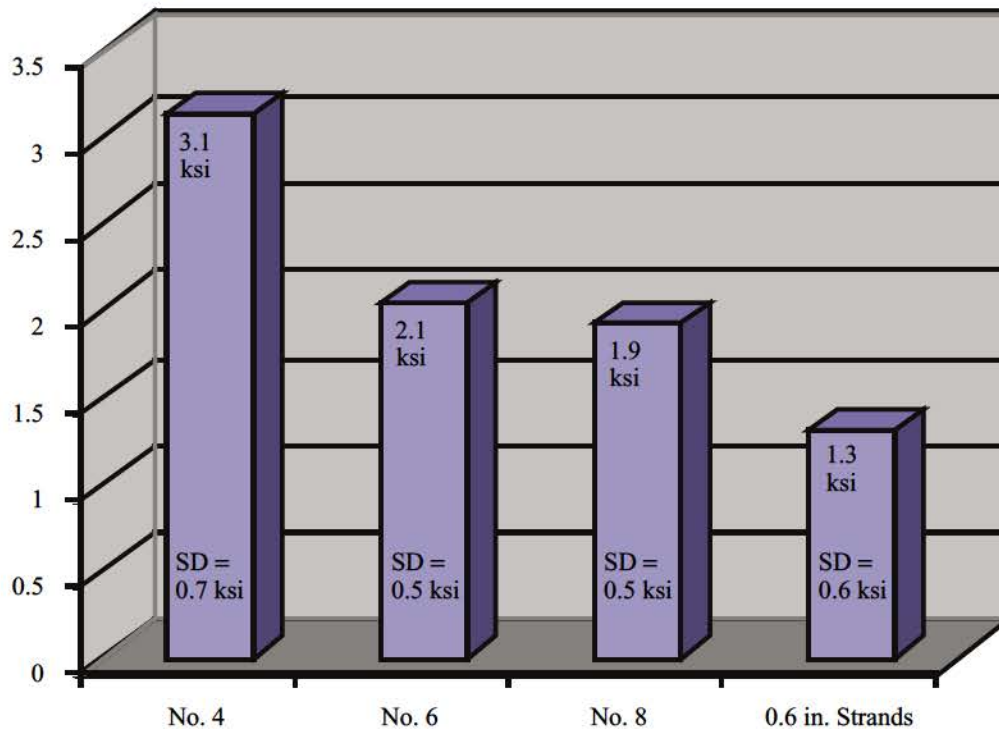


Fig. 20. SCC (Mix 2) 28-day bond strengths. Note: 1 in. = 25.4 mm; 1 ksi = 6.895 MPa.

- A_{ps} = area of prestressing strands
- A_{tr-rel} = area of transformed section at transfer length
- e_c = predicted concrete strain
- e_{tr-rel} = eccentricity of strands with respect to initial transformed section
- E_c = elastic modulus of concrete
- f_{pi} = stress in prestressing strand just before release
- I_{tr-rel} = moment of inertia of transformed section at transfer length
- M_g = moment due to girder self-weight at transfer length
- $y_{tr-rel-C.G.}$ = distance from transformed section centroid to center of gravity of prestressing strands of I-girder bottom flange

Table 5 compares the predicted and measured concrete strains at the centroid of the bottom flange's strands. It should be noted that errors are introduced when a uniaxial stress state in the concrete is assumed, while the difference between measured and calculated concrete strains may be improved by conducting a three-dimensional analysis using finite element modeling.

Small Pullout Tests

Bond strength results from the first method were compared to those from the second method, as shown in Fig. 18. The bond strength was computed by dividing the pullout force by the product of reinforcing bar or prestressing strand circumference with the embedment length, as given in Eq. (3). Comparisons were made for No. 4 and No. 8 bars and 0.6 in.

(15.2 mm) diameter strands.

As shown in Fig. 19, the bond strengths of SCC (Mix 2) at 28 days were higher than those of the conventional concrete (Mix 3). Figs. 20 and 21 show the pullout test results from Mix 2 and Mix 3, respectively, for the various bar diameters. Fig. 22 shows a typical specimen after bond failure in a pullout test. The average bond strength may be determined from:

$$u_b = \frac{P_u}{\pi d_b (L_e)} \quad (3)$$

where

- u_b = average bond strength
- d_b = diameter of reinforcing bar or prestressing strand
- L_e = embedment length of prestressing strand
- P_u = ultimate pullout force

CONCLUSIONS

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

1. Limited test data have shown that the bond strength of SCC with deformed reinforcing bars is adequate. However, the use of a VMA may adversely affect the early compressive strength and the bond strength of SCC with pretensioning strands, which leads to longer transfer lengths. Further investigations into SCC bond strength issues are warranted.
2. SCC mixtures may experience significantly longer transfer lengths than those of conventional

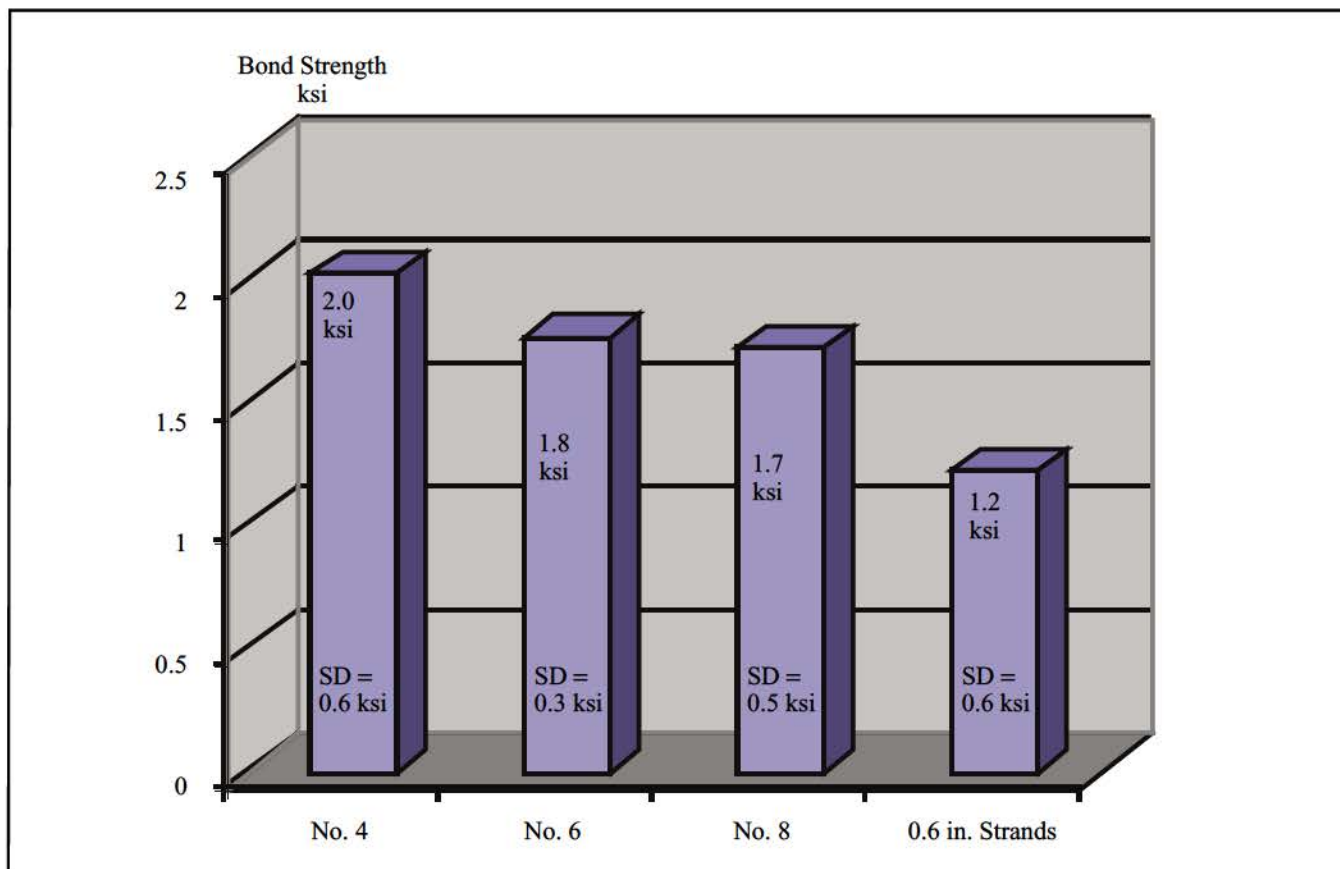


Fig. 21. Conventional concrete (Mix 3) 28-day bond strengths. Note: 1 in. = 25.4 mm; 1 ksi = 6.895 MPa.



Fig. 22a. Pullout failure.



Fig. 22b. Pullout failure.

concretes, more than 50 percent in some cases.

3. Moustafa pullout tests failed to reveal any early age bond strength reduction when using SCC with prestressing strands (see Fig. 14). A probable cause is that the three-dimensional stress state in a pretensioned concrete girder cannot be duplicated by the test.
4. The transfer length measurements from the three pretensioned bridge girders indicated that SCC had lower early bond strength than conventional concrete.
5. The SCC had higher bond strength than that of the conventional concrete at 28 days, which may warrant shorter development length requirements for SCC than for conventional concrete.
6. As expected, results from the pullout tests using both SCC and conventional concrete showed that the smaller the deformed bar diameter, the higher the bond strength.
7. Large-scale flexural tests using pretensioned concrete girders cast with SCC should be conducted to address development length issues.

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APPENDIX—NOTATION

α	= proportionality constant
A_{ps}	= area of prestressing strands
A_{tr-rel}	= area of transformed section at transfer length
d_b	= diameter of reinforcing bar or prestressing strand
e_c	= predicted concrete strain
e_{tr-rel}	= eccentricity of strands with respect to initial transformed section
E_c	= elastic modulus of concrete
f_{ci}^i	= specified compressive strength of concrete at time of initial prestress
f_{pt}	= stress in prestressing strand just before release
f_{pt}	= stress in prestressing strand immediately after release
I_{tr-rel}	= moment of inertia of transformed section at transfer length
L_t	= transfer length of prestressing strand
L_e	= embedment length of prestressing strand
M_g	= moment due to girder self-weight at transfer length
P_u	= ultimate pullout force
u_b	= average bond strength
$y_{tr-rel-C.G}$	= distance from neutral axis to center of gravity of prestressing strands of I-girder bottom flange



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