INVESTIGATING THE EFFECT OF MORTAR STRENGTH AND LOADING RATE ON THE NASP BOND TEST

By

CARISA BETH GRIEVE

Bachelor of Science

Colorado School of Mines

Golden, Colorado

2000

Submitted to the Faculty of the
Graduate College of the
Oklahoma State University
in partial fulfillment of
the requirements for
the Degree of
MASTER OF SCIENCE
May, 2004

INVESTIGATING THE EFFECT OF MORTAR

STRENGTH AND LOADING RATE ON

THE NASP BOND TEST

Thesis Approved:

Bruce W. Russell

Thesis Advisor

Robert N. Emerson

Dean of the Graduate College

ACKNOWLEDGMENTS

The author would like to thank the not only the faculty, staff, and students who assisted in the completion of the research project, but also the companies and organizations that provided funding and materials for the project.

Without the laboratory and writing guidance of Dr. Bruce Russell, the research project could not have been completed. Additionally, Dr. Robert Emerson and Dr. Charles Bowen assisted in the review process. In the laboratory, David Porter was of utmost assistance in the completion of my research project. He was always willing to help fabricate items and fix problems in the lab. The laboratory assistance of Jose Joseph, Jonathon Hisey, and Nandakumar Natarajan assisted is also greatly appreciated.

The primary funding for the project was provided by the National Cooperative Highway research Program. Additionally, materials were provided by Lafarge North America, Dolese Brothers of Stillwater, Mineral Solutions, Master Builders, and the North American Strand Producers.

TABLE OF CONTENTS

Page

LIST OF TABLES

LIST OF FIGURES

Chapter 1

INTRODUCTION

1.1 BACKGROUND

Although prestressed concrete construction was not successful until the twentieth century, the idea to utilize the compressive strength of concrete in combination with the tensile strength of steel was first patented in the 1880s. Over the years, prestressed concrete construction has become economical through advances in the strength of concrete and steel in conjunction with the economy of precast concrete plants. While many advances in the industry have occurred, a precise understanding of prestressing bond remains elusive. As a result, the variability of bond is not fully accounted for in design.

The bond ability of steel prestressing strand is a characteristic that continues to create disagreement between engineering designers and researchers. Through research and construction projects to date, there is a wide range in the bond ability of strand. The variation or scatter in bond ability may stem from a variety of sources. Such sources may include strand manufacturing techniques, the condition of strand (i.e. weathered versus clean strands), strand size, "top bar" effects, strand spacing, confining steel, epoxy coating, and concrete strengths. Some of the possible sources of variation will be explored further through literature review and the testing program.

Sufficient bond is required in prestressed concrete members in order to transfer the pretensioning force in the steel strand to the surrounding concrete resulting in initial compressive stresses in the concrete. With initial compressive stresses in concrete, members can be loaded externally in tension or in bending with small or no tensile stresses developed in the concrete. For prestressed concrete construction to be economical, the bond between the prestressing strand and concrete must be sufficient to result in smaller cross sections than in typical reinforced concrete design. Thus, since bond between strand and concrete is required for precompression of the concrete, an understanding the variation in bond ability of steel prestressing strand should be beneficial to continued improvement the industry.

1.2 OBJECTIVE

The bond ability of prestressing strand is investigated in this research project. The bond ability of strand is important because it affects transfer and development lengths of the strand along with pull-out forces for the strand. The project includes the following:

- Investigate prestressing strand bond issues through literature review of existing data dating back to the 1950s.
- Development of concrete mixture proportions for three concrete compressive strength levels for future research. The 28 or 56 day strengths desired are 10 ksi, 14 ksi, and 18 ksi.
- Investigate effects of grout strength and loading rate for the proposed North American Strand Producers (NASP) Pull-out Test.

1.3 SCOPE

The experimental scope of this project includes trial batching of grout and concrete mixes and strand pull-out tests. The trial batching for this project is required to determine mixes that meet the target one day and 28 or 56 day compressive strength combinations and are workable. After conducting trial batches, the grout mixes will be used for NASP bond testing. The testing will be conducted on two 1/2 in. diameter strand samples with three different water to cement ratios and two different loading frames. The strands will be cast in grout placed in 5 in. diameter by 18 in. long steel pipes. After curing, the strands will be pulled out with strand slip and forces recorded. A total of 12 NASP tests, with six specimens per test, will be conducted.

Chapter 2

DEFINITIONS AND BACKGROUND

2.1 DEFINITIONS

The three prestressing specimen and strand properties of interest in this project are transfer length, development length, and pull-out strength.

2.1.1 Transfer Length

The transfer length of a prestressing strand is the length required to transfer the effective prestressing force, after losses, from the strand to the concrete. Although ACI 318-02 does not specify an equation to define the transfer length of a prestressing strand, the commentary of ACI 318-02 infers that the transfer length of a strand is defined as follows:

$$l_t = \left(\frac{f_{se}}{3}\right)d_b \tag{2.1}$$

where l_t is the transfer length in inches, f_{se} is the effective stress in the prestressed reinforcement after all losses in ksi, and d_b is the strand diameter in inches. ACI 318-02 allows the transfer length of prestressing strand to be simplified to 50 strand diameters in the shear provisions of the code. In the AASHTO LRFD code, the transfer length is defined as follows:

$$l_t = 60d_b \tag{2.2}$$

where l_t and d_b are defined above.

2.1.2 Development Length

The development length of prestressing strand is the length of bond necessary for the strand tension to match the tensile demand for the cross section to attain the nominal flexural strength of the member. The development length is the sum of the transfer and flexural bond length. The additional length beyond the transfer length is referred to as the flexural bond length. Although the stresses in the concrete do not vary linearly, idealized equations for the development length of strand assumes a linear stress increase over the transfer length and then a smaller linear increase over the flexural bond length to the development length. ACI 318-02 defines the development length as follows:

$$l_d = \left(\frac{f_{se}}{3}\right)d_b + \left(f_{ps} - f_{se}\right)d_b \tag{2.3}$$

where l_d is the development length of the strand in inches, f_{ps} is the stress in the prestressed reinforcement at the nominal strength of the member in ksi, and f_{se} and d_b are defined previously. The first term represents the transfer length of the strand, and the second term represents the flexural bond length. The AASHTO LRFD code defines the development length of a prestressing strand as follows:

$$l_d = K\left(f_{ps} - \frac{2}{3}f_{pe}\right)d_b \tag{2.4}$$

where K is a constant with a value of 1.6 for fully bonded strands and 2.0 for debonded strands, f_{pe} is the effective stress in the prestressing strand after losses in ksi, and l_d, f_{ps}, and d_b are defined peviously. Except for the K factor, this equation is the same as the ACI equation. The equation without the K factor will be referred to as the basic development length equation. Alternatively, the AASHTO code allows the development length of a strand in pretensioned beams to be taken as:

$$l_d = \frac{4 f_{pbt} d_b}{f_c^{'}} + \frac{6.4(f_{ps} - f_{pe})(d_b)}{f_c^{'}} + 10 \qquad (2.5)$$

where f_{pbt} is the stress in the prestressing steel immediately prior to transfer in ksi as specified in Table 5.9.3-1 in the AASHTO code and f_c' is the specified compressive strength at 28 days in ksi.

2.1.3 Pull-Out Strength

In general, the pull-out strength of a strand is the force required to break the bond of the prestressing strand and pull it out of its embedment in concrete. The pull-out strength of a prestressing strand varies based on the pull-out test conducted, and thus the criteria for each individual pull-out test must also vary. The pull-out test conducted in this project is the North American Strand Producers (NASP) Pull-out Test which will be described in detail in this chapter and the chapters that follow.

2.2 BACKGROUND

The focus of this research project involves a standardized prestressing strand pull-out test which can be correlated to the bond ability of strands as seen in transfer and development lengths. The transfer length of a prestressing strand is important in the shear calculations of prestressed elements. The development length of prestressing strand is important for flexural stress calculations of prestressed elements. Research programs have revealed a large scatter in transfer and development lengths. The scatter may be a result of many sources including, but not limited to, strand diameter, concrete strength, strand debonding, and strand source.

Current transfer and development length expressions are based on the early work of Hanson and Kaar in the late 1950s on Grade 250, stress-relieved strand (Tabatabai and Dickson, 1995). Since that research, the industry standard for prestressing strand has changed to Grade 270, low relaxation strand. During manufacturing of the Grade 250 strand, convection heating was used which may have burned off much of the surface residues from the wire drawing process. In today's processes, induction heating is utilized which may have lowered the surface temperatures relative to the convection heating process, thus changing the bonding characteristics of the strand by not removing the residues from the drawing process. (Rose and Russell 1997)

Due to the changes in manufacturing of prestressing strand and the noticeable variations in transfer and development lengths, current data must be analyzed to determine reliable transfer and development length expressions. Additionally, since one of the major sources of transfer and development length scatter is strand source, standardized testing to determine the bond ability of prestressing strand will be analyzed.

2.2.1 Transfer Length

Numerous researchers have investigated the transfer length of prestressing strand and many expressions to determine the transfer length have been proposed. The transfer length is typically measured by concrete surface strains and sometimes calculated based on measured end slips. The focus of this review of transfer lengths will be on the effects of strand diameter, concrete strength and debonding of strands.

2.2.1.1 Grade 250 Strand

Research by Tabatabai and Dickson (1995) indicate that results from Hanson, LaFraugh, and Mass reported in 1963 was the basis for the transfer length expressions that we use today. Kaar et al. (1963) investigated the influence of concrete strength on transfer length of seven-wire strand at release. Grade 250, stress relieved prestressing strands of 1/4, 3/8, 1/2 and 0.6 in. diameter were used in rectangular beams with concrete strengths of 1660, 2500, 3330, 4170, and 5000 psi. The strand was unpitted and rust-free except for the 0.6 in. diameter strand. The transfer lengths were determined based on the concrete strains and were measured immediately upon release and periodically for one year.

The prestressed beams varied in size and number of prestressing strands. As the concrete strength increased, so did the number of strands. The size of the specimen varied with the size of prestressing strand. The smallest specimen was 3 x 4-3/16 in. for the 1/4 in. diameter strand, and the largest was 7.5 x 10.5 in. for the 0.6 in. diameter strand. Two, three, four, five, and six strands were used for the concrete strengths of 1660, 2500, 3330, 4170, and 5000 psi, respectfully. Hanson et al. concluded that the strand diameter had an effect on the transfer length and could be represented by a straight line up to 1/2 in. diameter. They also concluded that the concrete strength did not affect the transfer lengths. The results are given in Table 2.1 and Figure 2.1 and 2.2.

The results of the testing by Kaar et al. were used by ACI to develop the transfer length expressions. It should be noted that ACI used the average results from Kaar et al. for their expressions. Since the expressions are based on average results, it should be expected to have transfer length results above the ACI expression. Also, using the

Table 2.1. Average Transfer Length at Maximum Concrete Strain. (Kaar et al. 1963)

Strand Diameter	Average Transfer Length (in.)	
	Dead End	Cut End
1/4	10.4	12.5
3/8	22.3	26.2
1/2	34.6	41.2
0.6	29	45.6

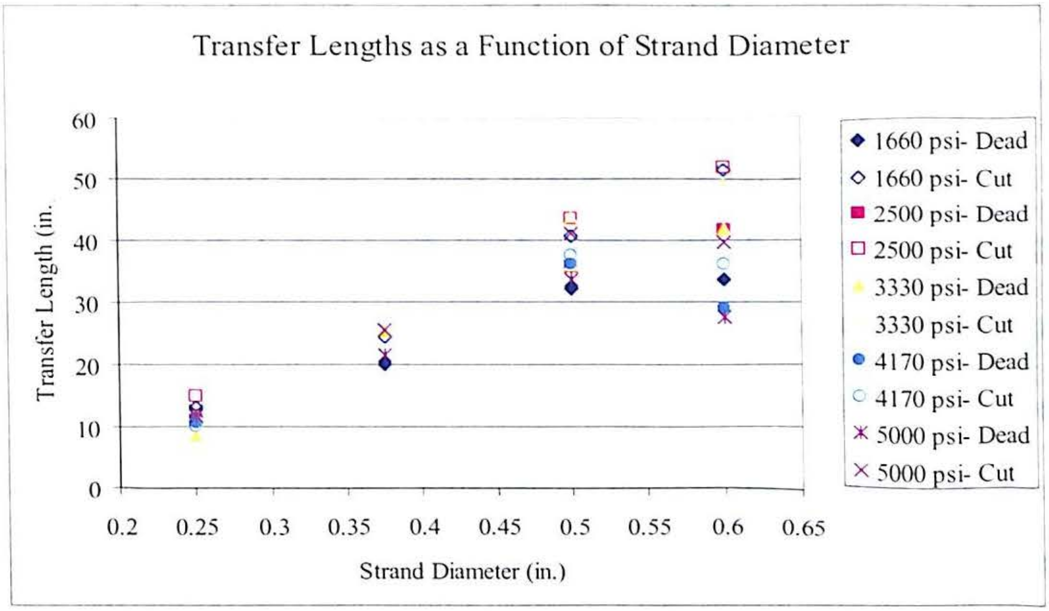


Figure 2.1. Transfer Lengths at Maximum Strain as a Function of Strand Diameter. (Kaar et al. 1963)

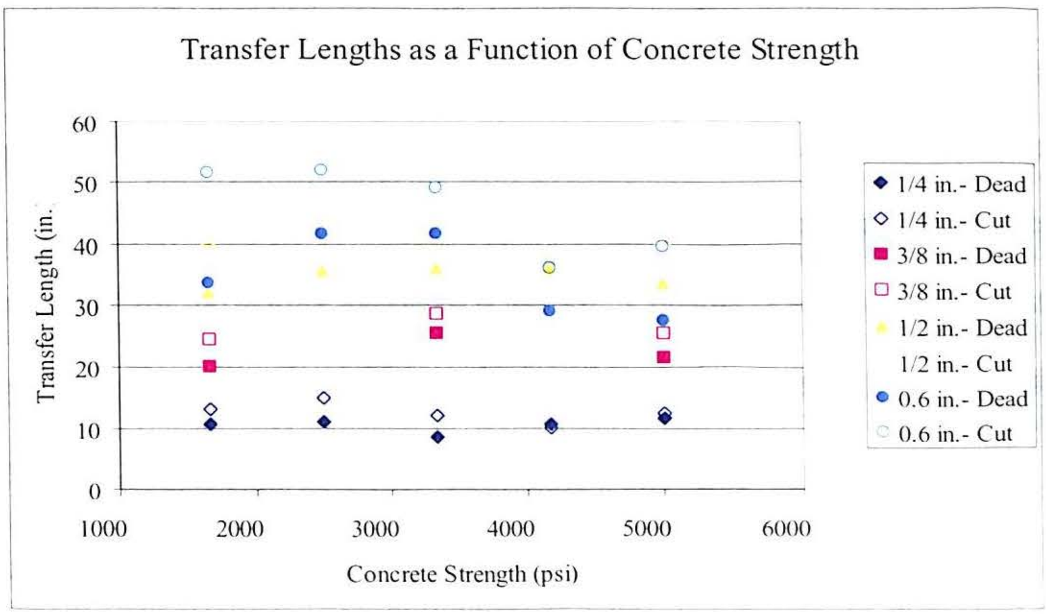


Figure 2.2. Transfer Lengths at Maximum Strain as a Function of Concrete Strength. (Kaar et al. 1963)

maximum concrete strain to determine the transfer length, the results of Kaar et. al are greater than the ACI expressions for 1/2 in. strand.

2.2.1.2 Grade 270 Strand

As the prestress industry began manufacturing Grade 270 strand, the effect of increased strand strength on transfer length needed investigation. Janney (1963) evaluated the transfer length of the higher strength strand. He concluded that Grade 270 strand had slightly longer transfer lengths than Grade 250 strand.

Since the testing conducted by Janney, the strand manufacturing processes have evolved, and now the industry standard is low relaxation, Grade 270 strand. One of the first transfer length studies on "modern" Grade 270 strand was conducted by Cousins, Johnston, and Zia (1990a). Although the focus of the research was epoxy-coated strands, Cousins et al. measured transfer lengths on uncoated 3/8, 1/2, and 0.6 in. diameter strands. There were two types of transfer length specimens. Two specimens for the 3/8 and 1/2 in. diameter uncoated specimens were square with a concentric strand, and the remainder of the uncoated specimens were rectangular with a strand located approximately at the lower kern point of the specimen. While the concentric strands were gradually released, the eccentric strands were flame cut. The concrete compressive strength for the testing averaged 4340 psi. Cousins et al. concluded that the results of the study indicate that the ACI transfer length equation may be unconservative. The results from the study are given in Table 2.2.

Table 2.2. Summary of Transfer Length Results from Cousins et al. (1990a)

Strand Diameter (in.)	Transfer Length				
	ACI Calculated (in.)	High (in.)	Low (in.)	Average (in.)	Coefficient of Variation (C. V.) (%)
3/8	23	42.0	26.0	34	14.4
1/2	30	74.0	33.0	50	20.8
0.6	36	68.0	44.0	57	13.3

As a result of the large transfer lengths observed by Cousins et al., much research has been conducted to determine reliable transfer length expressions. Since this includes tests by numerous researchers, all of the data will not be discussed in this report.

Mitchell, Cook, Khan, and Tham (1993) conducted a study to determine the influence of concrete strength on the transfer and development length of prestressing strand. The program varied the 28 day concrete compressive strength from 4500 to 12,900 psi and used 3/8, 1/2, and 0.62 in. diameter strands. The strands were all used in their as received condition; however, the researchers noted that the 3/8 in. diameter strand was slightly rusted. The authors concluded that as the compressive strength of the concrete increased, the transfer length of the prestressing strands decreased. They proposed the following equation:

$$l_t = 0.33 f_{pi} d_b \sqrt{\frac{3}{f'_{ci}}} \tag{2.6}$$

where f_{pi} is the stress in the strand immediately after transfer and f'_{ci} is the concrete compressive stress at transfer. The results from this study are summarized in Table 2.3; however, the values from the beams that had "problems during stressing" were omitted.

Deatherage, Burdette, and Chew (1994) examined the spacing requirements of prestressing strand. Their testing was conducted on twenty full-scale AASHTO Type I beams with 1/2 in., 1/2 in. special, 9/16 in., and 0.6 in. diameter strands. The strands were spaced at 1.5, 2, and 2.5 in. Based on their results, Deatherage et al. recommended that:

1. The transfer length of 9/16 in, 1/2 in., and 1/2 in. special strands should be calculated as:

   $$l_t = (f_{si}/3)d_b \tag{2.7}$$

   where f_{si} is the initial prestress in steel reinforcement.

2. 0.6 in. diameter strands should be accepted as standard practice, but the transfer length should be investigated further due to conservative results from the accepted transfer length equation.
3. 1/2 in. strands should be permitted to be spaced at 1.75 in. on center.

The results are summarized in Table 2.3.

Cousins, Stallings, and Simmons (1994) investigated spacing requirements of 1/2 in. diameter prestressing strands by measuring transfer lengths and conducting development length tests. They tested six T-shaped beams with 2 in. spacing and six with 1.75 in. spacing. Two concrete mixtures were utilized in order to determine the effect of concrete strengths. Based on their results, the authors made three conclusions and are as follows:

1. The transfer length decreases with increased concrete compressive strength.
2. The spacings of 1.75 and 2.0 do not significantly affect the transfer length.
3. The transfer lengths for the lower strength concrete were significantly greater that the ACI equation transfer length of 33 to 35 in.

The transfer length results are summarized in Table 2.3.

Russell and Burns (1996) examined transfer lengths for 1/2 and 0.6 in. strands varying strand spacing, debonding, confining reinforcement, and geometry. The tests were conducted on rectangular and AASHTO-type I-beams. The researchers concluded that the bond behavior of the 0.6 in. diameter strands was similar to that of the 1/2 in. diameter strands. Since the average transfer lengths were only marginally predicted by the ACI and AASHTO codes, the researchers believed that transfer length should be approximated by the more conservative expression $f_{se}d_b/2$. Since debonded strand transfer lengths were less than that of the fully bonded strands, Russell and Burns recommended that the transfer length of debonded strands be taken to be the same as for bonded strands. Based on their results, they also recommended that 0.6 in. diameter strands be allowed to be spaced at 2 in. on center. The results are summarized in Table 2.3.

Logan (1997) led an extensive research program to investigate the bond quality of prestressing strand which included a transfer length investigation. The testing was conducted on 1/2 in. diameter strand from five sources. The transfer length was determined by end slips rather than by concrete strains like the rest of the programs discussed. Since the testing was designed to evaluate the bond quality, no conclusions on transfer length were made. A detailed summary of the testing program as it relates to standardized testing is included in the "Standardized Testing" section. It should also be noted that two of the strands, "D" and "ER," resulted in poor bond quality results in all tests. A summary of the transfer length results are included in Table 2.3.

Russell and Burns (1997) continued their studies of transfer lengths by conducting a research program that measured transfer lengths of 1/2 and 0.6 in. diameter prestressing strands in order to evaluate the ability of the larger strand (0.6 in.) to transfer its prestressing force and to evaluate the current design provisions for the transfer lengths. Eighteen single strand beams were evaluated, eight with 1/2 in. diameter strands and ten with 0.6 in. diameter strands. Four specimens contained debonded strands, while the rest contained fully bonded strands. Based on the results, although the 0.6 in. diameter strands resulted in more damage to the specimen, the authors concluded that the 0.6 in. diameter strands could be safely utilized in pretensioning applications. They also concluded that a more conservative transfer length of $80d_b$ should be adopted into the design codes. A summary of the results is included in Table 2.3; however, the beams that did not achieve the minimum required release strengths are omitted.

Rose and Russell (1997) conducted a study of standardized tests measuring the bond performance of prestressing strands and compared the results to measured transfer lengths. Although only the transfer lengths measurements will be discussed here, an extensive review of the standardized testing is included in the section "Standardized Testing." Transfer lengths were measured on 1/2 in. diameter strand in the as received, cleaned, silane treated, and weathered conditions for one strand source and in the as received condition for two other strand sources. Since the purpose of the project was to investigate standardized tests, conclusions for transfer length values were not discussed by the authors. However, for the strands tested, with the exception of the silane treated strands, the data shows that the measured transfer length was less than the ACI transfer length. The transfer length results are summarized in Table 2.3.

Shing et al. (2000) conducted a research program measuring the development and transfer lengths of strand in high performance concrete box girders. The study was conducted to demonstrate the adequacy of the 0.6 in. diameter strands proposed for a bridge project. The specimens were scaled models of the actual girders for the bridge project. The box girders were 15 in. wide x 21.75 in. deep with nine 0.6 in. diameter prestressing strands spaced at 2 in. on center. The strand was used in the as received condition which included a little rust. The authors conclude that the ACI transfer length equation was adequate for the given girders; however, general design recommendations could not be drawn on this data alone. The transfer length results are given in Table 2.3.

Steinberg, Beier, and Sargand (2001) conducted an experimental program to study the effects of sudden prestress force transfer in pretensioned beams. The study included three prestressed concrete beams with four 1/2 in. diameter strands spaced at 2 in. on center. The transfer lengths were determined by electrical strain gauges on the strands and mounted to the beam surface, strains from surface targets, and by end slip measurements. The results from the surface mounts and the end-slip resulted in similar transfer lengths which were larger than the 25 in. calculated from the expression for transfer length in the ACI of $50d_b$; however, the average transfer length measure by surface targets was less than the ACI 25 in. transfer length. The transfer lengths from the surface targets of two beams are included in Table 2.3.

Shahawy (2001) conducted transfer and development length tests while evaluating the AASHTO provisions for strand development length in prestressed members. Since the focus of the paper was development length, a more extensive review will be presented in the section "Development Length." In the study, Shahawy measured transfer lengths of 1/2 in. and 0.6 in. diameter strands for multiple strand piles and beams. The published results are limited, but Shahawy did comment that concrete release strength did have an effect on the transfer length and that 2 in. spacing was sufficient for 0.6 in. diameter strand. The transfer length results of the study are given in Table 2.3.

Kahn, Dill, and Reutlinger (2002) reported on a research program conducted to verify that the transfer and development lengths of 0.6 in. diameter strand were less than that calculated with AASHTO Specifications when using high performance concrete. The testing was conducted on AASHTO Type II girders with ten 0.6 in. diameter prestressing strands. The results indicated that the transfer length was less than that given by AASHTO and ACI code provisions and that the current code provisions could be used for pretensioned concrete girders with strengths up to 14,490 psi. The transfer length results are included in Table 2.3.

Wan, Harries, and Petrou (2002) reported transfer lengths measured in prestressed concrete piles. The piles contained eight 1/2 in. diameter Grade 270 prestressing strands. Three types of concrete were used in fabricating the piles, one contained no admixtures, one contained a set retarder, and one contained a high range water reducer. The results of the testing indicate that the transfer length of "bottom strands" is regularly less than that given by the ACI expression; however, the "top strands" result in transfer lengths that exceed that given by the ACI expression. The transfer length results determined by surface strains using 1/2 in. diameter strand are given in Table 2.3.

Brown (2003) reported transfer lengths measured on single strand and double strand rectangular beams. The transfer lengths were conducted in conjunction with Moustafa Pull-Out Tests, NASP Pull-Out Tests, PTI Bond Tests, and Development Length Tests. The testing was conducted on 1/2 in. diameter Grade 270 strand from four manufacturers cast in the bottom of 6.5 in. x 12 in. x 18 ft long beams. While the single

Table 2.3. Summary of Reported Transfer Lengths at Release.

Research Program	Strand Diameter	Strand Condition	No. of Strands	Strand Spacing (in.)	f'_{ci} (psi)	f_{si} (ksi)	Release Method	Transfer Length Avg. (C. V) (in.) (%)
Mitchell et al. (1993)	3/8 in.	As Received	1		3000-3975		Gradual	19.2 (14.9)
	3/8 in.	As Received	1		6950-7310		Gradual	13.9 (15.4)
	1/2 in.	As Received	1		3000-3975		Gradual	24.0 (9.3)
	1/2 in.	As Received	1		6950-7310		Gradual	17.2 (15.7)
	0.62 in.	As Received	1		3000		Gradual	30.3 (9.3)
	0.62 in.	As Received	1		6950-7310		Gradual	19.0 (12.2)
Deatherage et al. (1994)	1/2 in.	As Received	10	2.0	3780-4170	203	Simultaneous	32.5 (10.8)
	1/2 in.	Weathered 1 Day	10	2.0	4775-5235	203	Simultaneous	23.3 (8.1)
	1/2 in.	Weathered 3 Day	10	1.75	4775-5553	203	Simultaneous	19.5 (7.3)
	1/2 in. Special	As Received	11	2.0	4950-5340	203	Simultaneous	32.5 (5.3)
	1/2 in. Special	Weathered 3 Day	10	2.0	5300-5410	203	Simultaneous	31.0 (20.6)
	9/16 in.	As Received	10	2.0	3360-3750	203	Simultaneous	34.5 (17.3)
	9/16 in.	Weathered 3 Day	7	2.0	4950-5060	203	Simultaneous	27.5 (12.1)
	0.6 in.	As Received	7	2.5	4100-4280	203	Simultaneous	26.5 (10.0)
	0.6 in.	As Received	7	2.5	5230-5450	203	Simultaneous	22.3 (4.3)
Cousins et al. (1994)	1/2	As Received	9	1.75	5305-5353	199-203	Flame	56.3 (20.1)
	1/2	As Received	9	1.75	7663-8223	204-207	Flame	39.0 (16.1)
	1/2	As Received	9	2.0	5305-5353	200-205	Flame	56.0 (11.7)
	1/2	As Received	9	2.0	7663-8223	205-207	Flame	35.3 (21.2)
Russell and Burns (1996)	1/2 in.	As Received	1		4480	203	70% Flame	29.4 (10.7)
	1/2 in.	As Received	3	2.0	4200-4320	203	70% Flame	29.2 (6.6)
	1/2 in.	As Received	5	2.0	3850	203	70% Flame	38.2 (9.9)
Fully Bonded Data	1/2 in.	As Received	5	2.0	4040-4640	203	Flame	20.1 (10.2)
	1/2 in.	As Received	8	2.0	5150-5580	203	Flame	36.1 (16.0)
	0.6 in.	As Received	1		3850	203	70% Flame	47.0 (3.0)

Table 2.3 Continued. Summary of Reported Transfer Lengths at Release.

Research Program	Strand Diameter	Strand Condition	No. of Strands	Strand Spacing (in.)	f'_{ci} (psi)	f_{si} (ksi)	Release Method	Transfer Length Avg. (C. V) (in.) (%)
Russell and Burns (1996) Continued Fully Bonded Data	0.6 in.	As Received	3	2.0	4200-4760	203	70% Flame	41.0 (11.5)
	0.6 in.	As Received	3	2.5	4760	203	70% Flame	43.3 (4.8)
	0.6 in.	As Received	5	2.0	4480	203	70% Flame	48.0 (4.1)
	0.6 in.	As Received	4	2.0	4460-4880	203	Flame	32.0 (9.3)
Russell and Burns (1996) Debonded Data	1/2 in.	As Received	1			203		22.6 (18.3)
	1/2 in.	As Received	3			203		24.8 (17.5)
	1/2 in.	As Received	8			203		30.0 (13.8)
	0.6 in.	As Received	1			203		35.5 (9.8)
	0.6 in.	As Received	3			203		28.8 (20.9)
Logan (1997)	1/2 in. - TW	Weathered	1			185	Flame & Saw	15
	1/2 in. - TA	As Received	1			185	Flame & Saw	13
	1/2 in. - A	As Received	1			185	Flame & Saw	15
	1/2 in. - B	As Received	1			185	Flame & Saw	14
	1/2 in. - D	As Received	1			185	Flame & Saw	24
	1/2 in. - ER	As Received	1			185	Flame & Saw	34
Russell & Burns (1997) Fully Bonded Data	1/2 in.	As Received	1		3770-4190	203	Flame	33.6 (25.7)
	0.6 in.	As Received	1		3520-4380	203	Flame	36.7 (20.0)
Russell & Burns (1997) Debonded Data	1/2 in.	As Received	1		3770	203	Flame	18.5 (9.3)
	0.6 in.	As Received	1		4380	203	Flame	50.3 (8.3)
Rose & Russell (1997)	1/2 in. - AA	As Received	2	2	4050	203	Flame	19.1 (26.7)
	1/2 in. - BA	As Received	2	2	4470	203	Flame	15.7 (35.7)
	1/2 in. - CA	As Received	2	2	3990	203	Flame	14.4 (21.5)
	1/2 in. - CC	Cleaned	2	2	4080	203	Flame	15.4 (43.5)
	1/2 in. - CS	Silane Treat	2	2	4450	203	Flame	65.8 (72.9)
	1/2 in. - CW	Weathered	2	2	4690	203	Flame	12.5 (25.6)

Table 2.3 Continued. Summary of Reported Transfer Lengths at Release.

Research Program	Strand Diameter	Strand Condition	No. of Strands	Strand Spacing (in.)	f'_{ci} (psi)	f_{si} (ksi)	Release Method	Transfer Length Avg. (C. V) (in.) (%)
Shing et. al (2000)	0.6 in.	As Received	9	2	About 7800	204	Flame	23.4 (4.8)
Steinberg et al. (2001)	1/2 in.	As Received	4	2	3280-3600		Flame	41.3 (18.6)
Shahawy (2001)	1/2 in.	As Received	8-40	2-4				30
	0.6 in.	As Received	8-40	2-4.5				34
Kahn et al. (2002)	1/2 in.	As Received	10	2	About 10140	190	Flame	16.0 (0)
	1/2 in.	As Received	10	2	About 14490	190	Flame	14.4 (12.4)
Wan et al. (2002)	1/2 in.	As Received	8	5	3540-5770	203	Flame	42.6 (31.1)
Brown (2003)	1/2 in.- AA	As Received	1		3980			10.7 (14.4)
	1/2 in. - FF	As Received	1		3980			21.1 (2.1)
	1/2 in. - HH	As Received	1		3980			20.2 (4.6)
	1/2 in. - II	As Received	1		3980			25.7 (4.7)
	1/2 in.- AA	As Received	2	2	4060			13.1 (2.7)
	1/2 in. - FF	As Received	2	2	4060			22.9 (4.9)
	1/2 in. - HH	As Received	2	2	4060			22.4 (4.1)
	1/2 in. - II	As Received	2	2	4060			41.3 (9.6)

strand specimens contained minimal shear reinforcement, the double strand specimens contained shear reinforcement of #3 stirrups at 6 in. on center. Transfer lengths were derived from strand end slip using the equation:

$$L_t = (Slip) \times (2 \times E_{ps}) / f_{si} \tag{2.8}$$

Transfer length measurements were taken at release, 24 hours, 4 days, 7 days, 14 days, 19 days, 21 days, 28 days, and at flexural testing. The results are summarized in Table 2.3.

2.2.1.3 Summary and Conclusions of Transfer Length

As seen from Table 2.3, the transfer lengths of strand vary greatly. Researchers have continually attempted to investigate the cause of such variance. Variance has been attributed to many factors, which include strand diameter, concrete strength, and debonding of strands.

As seen in the results given in Table 2.3, as the strand diameter increases, the transfer length also increases. Although this trend is not seen in all data, researchers have often noted evidence of weathering on strands when the trend deviates.

Based on the data presented in Table 2.3, the transfer length of prestressing strand appears to vary directly with concrete strength. Like the strand diameter trend, deviations occur and this is likely due to other parameters not held constant.

Based on the research performed by Russell and Burns, debonded strands have smaller transfer lengths than bonded strands. Due to the smaller transfer lengths, debonded strands can be designed under the same equations as fully bonded strands.

With quality control testing to "weed out" the poor performing strands, the transfer length data will converge to a smaller range. Although the transfer length expression should be changed to reflect a larger majority of strands, the data will be more consistent without "poor performers."

2.2.2 Development Length

The current ACI and AASHTO development length expressions are based on the results of testing by Hanson and Kaar (1959) on Grade 250 strand. The development length is considered to be the length at which the beam failure switches from bond to flexural. Like the transfer length testing, a large gap in research occurred between the early testing on Grade 250 strand and the current testing on Grade 270 strand. The focus of the development length review will be on the effects of strand diameter, concrete strength and debonding of strands. The development length information will be divided into two section, fully bonded strands and debonded strands.

2.2.2.1 Fully Bonded Strand

Fully bonded strand is strand that has nothing attached to break the bond between the strand and concrete. The transfer of the prestressing force begins at the end of the member in fully bonded strands.

2.2.2.1.1 Grade 250 Strand

The research by Hanson and Kaar (1959) was conducted from 1955 to 1957 at Portland Cement Association Laboratories in order to determine the maximum diameter of strand that one could safely use in a particular beam. The testing was conducted on 1/4, 3/8, and 1/2 in. Grade 250, stress-relieved prestressing strand which was placed in beams with a Type I cement concrete with a 2 in. slump. The testing was conducted to determine the influence of strand diameter, concrete strength, number of strands and surface condition.

The testing conducted to determine the influence of strand diameter was performed at a variety of embedment lengths. Since the authors did not state the development length in the paper, one can only infer that the development length is greater than the maximum embedment length of bond failure and less than the minimum embedment length for flexural failure. The data suggests that the development length increases as the strand diameter increases. The results for all testing parameters are summarized in Table 2.4.

The testing conducted to determine the influence of concrete strength, number of strands, and surface condition were performed at a single embedment length for the variable in question. The results from the concrete strength testing indicate that more flexural failures occur with lower strength concrete mixtures. From the results for varying concrete strength, Hanson and Kaar concluded that "reduction of concrete strength has more influence on the ultimate flexural strength than on ultimate bond strength." Based on the results from varying the number of strands, the risk of bond

Table 2.4. Summary of Development Length Testing From Hanson and Kaar (1959)

Strand Diameter (in.)	Strand Condition	f_c' (psi)	f_{se} (ksi)	Embedment Length (in.)	
				Maximum for Bond Failure	Minimum for Flexural Failure
1/4	As Received	5980-7800	141.0	42	48
3/8	As Received	5130-5730	129.7-144.6	60	60*
1/2	As Received	5090-6300	132.0-148.0	80	80*

* Although a specimen failed in flexure at this embedment length, some specimens with the same embedment length failed in bond. The minimum embedment length in which no bond failures were recorded was 90 in.

failure is greater for lower quantities of strand. Based on the results from the surface condition testing, rusted strands perform as well as or better than clean, smooth strand.

2.2.2.1.2 Grade 270 Strand

Extensive development length research involving Grade 270, low-relaxation strand began with a research program by Cousins, Johnston, and Zia (1990b). The testing program was designed to study epoxy coated strand; however, the results from the uncoated strand tests sparked a Federal Highway Administration (FHWA) memorandum that led to many research programs investigating bond of prestressing strand.

Cousins, Johnston, and Zia utilized 3/8, 1/2, and 0.6 in. diameter strands. Each specimen contained one strand and was designed to have a minimum of 5000 psi 28 day compressive strength concrete. The authors concluded that the experimental development lengths were longer than calculated by the ACI expression. A summary of the development length results is given in Table 2.5.

As a result of this study, the FHWA issued a memorandum on October 26, 1988. The memorandum imposed the following restrictions:

1. The use of 0.6 in. diameter strand was prohibited.
2. Minimum center-to-center spacing of four times the strand diameter was required.

Table 2.5. Summary of Development Length Results from Cousins et al. (1990b)

Strand Diameter (in.)	Strand Condition	Embedment Length (in.)		Calculated ACI Development Length (in.)
		Maximum for Bond Failure	Minimum for Flexural Failure	
3/8	As Received	54	57	48.4
1/2	As Received	119	*	62.1
0.6	As Received	126	132	76.7

* No flexural failures were reported for 1/2 in. diameter strand

3. An additional multiplier of 1.6 to the AASHTO equation (identical to the ACI equation) was required when calculating the development length of all strands.
4. Where a strand is debonded, the additional multiplier to the AASHTO equation was 2.0 instead of 1.6.

Due to the new restrictions that caused a large increase in project costs, several research projects were initiated.

As discussed earlier, Mitchell, Cook, Khan, and Tham (1993) conducted a study to determine the influence of concrete strength on the transfer and development length of prestressing strand. The authors concluded that as the compressive strength of the concrete increased, the development length of the prestressing strands decreased. They proposed the following equation:

$$l_d = 0.33 f_{pi} d_b \sqrt{\frac{3}{f'_{ci}}} + \left(f_{ps} - f_{se}\right) d_b \sqrt{\frac{4.5}{f'_c}} \qquad (2.9)$$

The results from this study are summarized in Table 2.6.

Deatherage, Burdette, and Chew (1994) examined the spacing requirements of prestressing strand. The specimens and transfer length results were discussed earlier. In addition to the recommendations discussed earlier, Deatherage et al. also recommended the following equation for development length:

$$l_d = f_{si}/3 + 1.50\left(f_{ps} - f_{se}\right) \qquad (2.10)$$

The results of the development length tests are summarized in Table 2.6.

As discussed earlier, Cousins, Stallings, and Simmons (1994) investigated spacing requirements of 1/2 in. diameter prestressing strands by measuring transfer lengths and conducting development length tests. Based on their results, three conclusions for development lengths were made and are as follows:

1. The development length decreases with increased concrete compressive strength.
2. The spacings of 1.75 and 2.0 do not appear to affect the development length.
3. The development length for the lower strength concrete were significantly greater than the ACI equation development length of 68 to 72 in.

The development length results are summarized in Table 2.6; since no difference was seen in the spacings, the results are grouped only by concrete strength.

Logan (1997) led an extensive research program to investigate the bond quality of prestressing strand which included a development length investigation. The testing was conducted on 1/2 in. diameter strand from five sources. The development length tests were conducted with simple spans utilizing an embedment length equal to the ACI development length (73 in.) and an embedment length equal to 80 percent of the ACI development length (58 in.). Additionally, the development length tests were conducted with cantilevered spans utilizing an embedment length equal to the ACI development length and an embedment length equal to the ACI transfer length (29 in.).

For three of the strand sources using as received strands, in all four tests, the specimens failed due to flexure, meaning that the development length was less than the embedment length. For the weathered strand, the specimens failed due to flexure except for at an embedment length equal to the ACI transfer length where the specimen failed in bond. The results indicate that the development length of the weathered strand was less than 80 percent of the ACI development length. For the other two strand sources, all failures were bond failures which indicate that the development length was greater than the calculated ACI development length. It should also be noted that two of the bond failure strands, "D" and "ER," resulted in poor bond quality results in all tests.

A detailed summary of Logan's testing program as it relates to standardized testing is included in the "Standardized Testing" section. No conclusions were made by Logan about the development length of strands. However, based on the data, it can be concluded that the ACI and AASHTO equation for development length was sufficient for the strands that performed will in all bond quality tests.

Peterman, Ramirez, and Olek (2000) conducted development length testing on two sources of strand in single strand rectangular and multiple strand T-shaped semi-lightweight beams. The single strand beams contained shear reinforcement in the center of the beam, and the multiple strand beams contained shear reinforcement the entire length of the beam. The beams were designed with 7000 psi 28 day compressive strength concrete. Each specimen was tested with an embedment length based on the authors' "worst case" development length, 73.5 in., based on the ACI code equation.

For the single strand specimens, six development length tests were conducted for each of the two strand sources. For strand source "A", four tests failed in flexure while the other two tests failed in shear. For strand source "B", five tests failed in flexure and the other one test failed in shear.

For the multiple strand specimens, two tests were conducted on strand source "A" with six in. spacing for shear reinforcement and four tests were conducted on strand source "B" with varying shear reinforcement. For strand source "A" with 6 in. shear reinforcement spacing, the two beams failed in flexure. For strand source "B", the specimen with 3 in. shear reinforcement spacing failed in flexure, while the two specimens with 6 in. spacing and the specimen with 15 in. spacing experienced bond failure.

Based in their single strand development length test results, Peterman et al. concluded that the AASHTO and ACI development length equations were sufficient to develop the full capacity of semi-lightweight concrete specimens. Based in their multiple strand development length test results, they made the following recommendations:

1. The development length should be enforced at a distance d_p, equal to the distance from the extreme compression fiber to the centroid of the prestressed reinforcement, from the maximum moment.
2. Beam sections should be designed so that the prestress force can theoretically be developed at a distance d_p toward the free end of the strand.
3. In lieu of the above two recommendations, the designer may "provide enough transverse reinforcement to minimize the shift in tensile demand that will occur in the event of diagonal cracking."

Shing et al. (2000) conducted a research program, which was discussed earlier, measuring the development and transfer lengths of strand in high performance concrete box girders. The study was conducted to demonstrate the adequacy of the 0.6 in. diameter strands proposed for a bridge project. The authors conclude that the ACI development length equation was adequate for the given girders; however, general design recommendations could not be drawn on this data alone. The results are given in Table 2.6.

Shahawy (2001) conducted transfer and development length tests to evaluate the AASHTO provisions for strand development length in prestressed members. The development length tests were conducted on 1/2 in., 1/2 in. special, and 0.6 in. diameter strands. The testing was conducted on solid and voided slabs, piles, and AASHTO beams. Shahawy recommended two new equations for development length, one for sections with depths equal to or less than 24 in. and one for section with depths greater than 24 in. The equation for sections with depths less than or equal to 24 in. is as follows:

$$l_d = \left(\frac{f_{si}}{3}\right)d_b + \frac{\left(f_{su}^{*} - f_{se}\right)d_b}{1.2} \tag{2.11}$$

where f^{*}_{su} is the stress in prestressed reinforcement at nominal strength. The equation for sections with depths greater than 24 in. is as follows:

$$l_d = \left(\frac{f_{si}}{3}\right)d_b + \frac{\left(f_{su}^{*} - f_{se}\right)d_b}{1.2} + 1.47h \tag{2.12}$$

where h is the overall thickness of the section. Shahawy concluded that shear-flexural interaction has a significant effect on the development length of prestressing strand and recommends the equations above to account for the shear interaction. The development length results of the study are given in Table 2.6.

As discussed earlier, Kahn, Dill, and Reutlinger (2002) reported on a research program conducted to verify that the transfer and development lengths of 0.6 in. diameter strand were less than that calculated with AASHTO Specifications when using high performance concrete. The results indicated that the development length was less than that given by AASHTO and ACI code provisions and that the current code provisions could be used for pretensioned concrete girders with strengths up to 14,490 psi. The development length test results are included in Table 2.6.

Table 2.6. Summary of Reported Development Length Test Results.

Research Program	Strand Diameter (in.)	Strand Condition	f_c' (psi)	f_{se} (ksi)	Calculated ACI Development Length	Embedment Length (in.)	
						Maximum for Bond Failure	Minimum for Flexural Failure
Mitchell et al. (1993)	3/8	As Received	4500	157	49	*	43.3
	3/8	As Received	6240	159	51	39.4	53.1
	3/8	As Received	9430	162	52	*	31.5
	3/8	As Received	10,880	165-175	50	*	27.6
	3/8	As Received	12,900	170-171	52	*	22.6
	1/2	As Received	4500	182	67	47.2	49.2
	1/2	As Received	6240	149-151	73	*	49.2
	1/2	As Received	9430	182	71	*	25.6
	1/2	As Received	10,880	167-169	72	*	37.4
	1/2	As Received	12,900	184-185	68	25.6	37.4
	0.62	As Received	4500	149-158	86	70.9	73.4
	0.62	As Received	9430	159	89	28.6	27.6
	0.62	As Received	12,900	121-122	98	38.4	#
Deatherage et al. (1994)	1/2	As Received	5476-6746	191	71^	85	#
	1/2	Weathered 1 Day	6858-7600	191	71^	77.4	81.25
	1/2	Weathered 3 Days	5341-5989	200	68^	77.4	73.5
	1/2 Special	As Received	6624-6800	199	72^	82.5	#
	1/2 Special	Weathered 3 Days	5967-6181	200	71^	75	81
	9/16	As Received	5533-5921	185	83^	106	#
	9/16	Weathered 3 Days	6119-6237	187	82^	104.4	104.4
	0.6	As Receieved	5126-7984	184-191	87^	83.5	85.8

Notes:
* No specimens reported with bond failure.
# No specimens reported with flexural failure.
^ No stress at nominal strength was given, so 270 ksi was assumed for calculation.
+ Slab specimen
§ d_p greater than 24 in.

Table 2.6 Continued. Summary of Reported Development Length Test Results.

Research Program	Strand Diameter (in.)	Strand Condition	f_c' (psi)	f_{se} (ksi)	Calculated ACI Development Length	Embedment Length (in.)	
						Maximum for Bond Failure	Minimum for Flexural Failure
Cousins et al. (1994)	1/2	As Receieved	6310-8010	153-165	68-72	114	126
	1/2	As Receieved	10,070-11,620	165-168	69-70	*	108
Shing et al. (2000)	0.6	As Received	11,000-11,200		92	60	65
Shahawy (2001)	1/2+	As Received			74	*	65
	1/2+	As Received			78	65	70
	1/2	As Received	6500		70	*	34
	0.6	As Received	7500		88.6	60	69
	1/2	As Received	7000		71	42	36
	1/2 Special	As Received	7500		72	36	36
	1/2	As Received	7000		68	56	66
	1/2 Special	As Received	7500		64.5	54	48
	1/2§	As Received	6000		96	78	66
Kahn et al. (2002)	0.6	As Received	15,170		96	75	88
	0.6	As Received	16,770		96	80	80
Brown (2003)	1/2 - AA	As Received	5300-6220		73		58
	1/2 - FF	As Received	5460-6260		73	73	73
	1/2 - HH	As Received	5140-6330		73	58	58
	1/2 – II	As Received	5360-6290		73	73	73

Notes:
* No specimens reported with bond failure.
# No specimens reported with flexural failure.
^ No stress at nominal strength was given, so 270 ksi was assumed for calculation.
+ Slab specimen
§ d_p greater than 24 in.

As discussed earlier, Brown (2003) reported development length testing in addition to other strand tests. The development length tests were conducted at varying embedment lengths. The beam specimen was simply supported and the load was applied using a spreader beam. The results are summarized in Table 2.6.

2.2.2.1.3 Summary and Conclusions of Fully Bonded Strand Development Length

The data for development length is more limited than for transfer length. As such, there is not sufficient data to determine the effects of certain variables. It can be seen from the data that in most cases the measured development length is less than the calculated development length. The research programs conducted by Cousins et al. (1990b) and Deatherage et al. (1994) are the exceptions; their development lengths were significantly greater than the calculated development lengths.

Based on the discussed results, while increasing concrete strength appears to decrease the development length of strands, increasing strand diameter appears to increase the development length of the strand.

Research must continue into the development length of prestressing strands in order to develop a more reliable equation. In order to converge development length data, a standardized test to evaluate bond quality could be utilized. With a qualifying criteria, the variation in strand development length should be decreased.

2.2.2.2 Debonded Strand

Debonding or blanketing of strands can be done to reduce concrete stresses at the end regions of pretensioned concrete beams and girders. It is an alternative to draping or harping strands. Since many fabricators consider the practice of debonding safer than draping as well as more economical, many prefer debonding to draping. The current code provisions are based on research conducted by Kaar and Magura (1965) and Rabbat et al. (1979). The research by Kaar and Magura resulted in the 2.0 multiplier of development length for debonded strands, and the research by Rabbat et al. resulted in an exception to the 2.0 multiplier in debonded strands with no tension at service load. Since the early development length research, new investigations into the cause of bond failures have been conducted under the direction of Dr. Ned Burns of the University of Texas at Austin.

2.2.2.2.1 Grade 250 Strand

Kaar and Magura (1965) conducted the first research of debonded strands utilizing Grade 250 seven-wire strands. The testing was conducted on half-scale AASHTO-PCI Type III girders that measured 22.5 in. deep using 5000 psi 28 day compressive strength concrete and twelve 3/8 in. diameter strands. Three flexural test girders were designed, fabricated, and tested. One girder contained only fully bonded prestressing strands, and the other two contained debonded strands. The two debonded strand specimens were labeled "fully blanketed" and "partially blanketed" by the researchers. The "fully blanketed" girder contained six staggered debonded strands and six fully bonded strands, and the "partially blanketed" girder contained four staggered debonded strands and eight fully bonded strands.

Debonded strands in the "fully blanketed" girder had an embedment length from the end of debonding to the point of maximum moment equal to the ACI development length for fully bonded strands. Debonded strands in the "partially blanketed" girder had an embedment length twice the ACI development length required for fully bonded strands. Since all beams were loaded identical, the embedment length of the fully bonded girder was significantly greater than the ACI development length.

All of the girders were subjected to five million cyclic loads and then statically loaded to failure. All of the beams were adequate for the cyclic loading; howver, they did not all perform the same in the static testing. The failure load of the fully bonded, "fully blanketed", and "partially blanketed" girders were 98 percent, 84 percent, and 96 percent, respectively, of the calculated failure moment. Based on these results, Kaar and Magura concluded that the performance of the debonded strand beam with an embedment length twice the ACI development length matched the performance of the fully bonded beam. It should be noted that the fully bonded beam's embedment length was significantly larger than the ACI development length and that no testing of debonded strands was conducted between one and two times the ACI development length. The current code provision requiring a factor of two be applied to the basic development length equation is based on this research.

Rabbat, Kaar, Russell, and Bruce (1979) conducted a research program in which the objective was to:

1. Determine whether tension under service loads affects the development length.
2. Determine whether one or two development lengths are required.
3. Determined whether confinement ties are beneficial for blanketed strands.

The study was conducted on six full-scale Type II AASHTO-PCI girders. Two of the girders contained draped strands and the others contained strands debonded to varying lengths.

Each of the girders contained twenty-two 7/16 in. diameter Grade 250 seven-wire prestressing strands. The draped strand girders contained twelve straight strands and ten draped strands. The girders with debonded strands contained eighteen fully bonded strands and four debonded straight strands. The girders were 36 in. deep with a 5 in. deep composite deck, both with 5000 psi concrete. Table 2.7 gives the details of the beams tested.

The beams were tested with loads applied at four points, at 13 ft. from each end and at 22 ft. from each end. The beams contained crack formers beginning 17 ft. from the ends, which caused cracks to form at specific locations. Five million cycles were applied to the beams and static loading was applied at 1 million and 2.5 million cycles. The beams were loaded to failure at 5 million cycles.

During fatigue testing, some beams failed or were prematurely halted due to large cracks. Specimen G11 developed large cracks at the outer crack former, so testing was

Table 2.7. Details of Test Specimens (Rabbat et al. 1979)

Specimen No.	Bottom Fiber Tensile Stress (psi)	Blanketing Lengths	Embedment Length	Confinement Reinforcement
G11	$6\sqrt{f_c'}$	11'-6" and 16'-6"	L_d	No
G13	$6\sqrt{f_c'}$	6' and 11'	$2L_d$	No
G10	$6\sqrt{f_c'}$	Draped	Draped	No
G14	0	11'-6" and 16'-6"	L_d	Yes
G12	0	11'-6" and 16'-6"	L_d	No
G10-A	0	Draped	Draped	No

halted at 3.78 million cycles. Increasing slip of debonded strands was occurring during loading. Specimen G13 develop large cracks at an outer and an inner crack former, so testing was halted at 3.2 million cycles. No slip in debonded strands was observed. Specimen G10 failed after 3.63 million cycles. It was concluded that all three of the specimens G10, G11, and G13 failed from fatigue.

The remainder of the specimens, G14, G12, and G10-A, failed in flexure during static loading. While small slip of less than 0.01 in. was observed in G14, no strand slip was observed in G12 and G10-A.

Based on the results of the investigation, it was concluded that:

1. Blanketing of strands is "a feasible technique that could lead to safer and more economical...bridge girders."
2. Tension at service load significantly affected the performance of the beams. For beams with zero tension at the peak load and designed for 1.0 times the development length, the behavior and capacity was similar to the beams with draped strands.
3. For girders designed with tension at service load, 2.0 times the development length provided adequate bond length.
4. Confining ties did not provide substantial improvement in the girder behavior.

The results of this study led to the exception to the 2.0 multiplier for debonded strands when the member is designed with no tension in the concrete.

2.2.2.2.2 Grade 270

One of the first research programs conducted on debonded, Grade 270, stress relieved strands was reported by Russell and Burns (1993). The testing was conducted on full-size, pretensioned composite bridge girders made of high strength concrete. The purpose of the research program was to observe the behavior of pretensioned girders made with high strength concrete and to compare the behavior of pretensioned girders made with debonded strands to girders made with draped strands.

Three pretensioned Texas Type C girders were constructed and tested. The girders had concrete strengths exceeding 10,000 psi, and the composite unshored concrete deck had concrete strengths exceeding 6,000 psi. The testing was conducted on 40 in. deep girders with an 8.25 in. deck. Each girder contained twenty-four 1/2 in. diameter strands. In one girder, FZ2450-3, six strands were draped. In the other two girders, DZ2450-1 and DZ2450-2, eight strands were debonded.

The test setup is shown in Figure 2.3. The girders were first statically loaded to "precrack" the specimen. The girders were then subjected to cyclic loading between 50 percent and 100 percent of the service load, which was defined as the load that produces a bottom fiber tension of $6\sqrt{f_c'}$. Overloads of 130 to 160 percent of the service loads were applied to the specimen periodically during the cyclic loading. After the cyclic loading, the girders were statically loaded to failure.

Girder FZ2450-3 and Girder DZ2450-1 experienced flexural failure at 100 percent and 96 percent, respectfully, of the calculated failure load. Although Girder DZ2450-2 experienced a horizontal shear failure at 89 percent of the calculated failure load, the failure could be predicted due to web shear cracking.

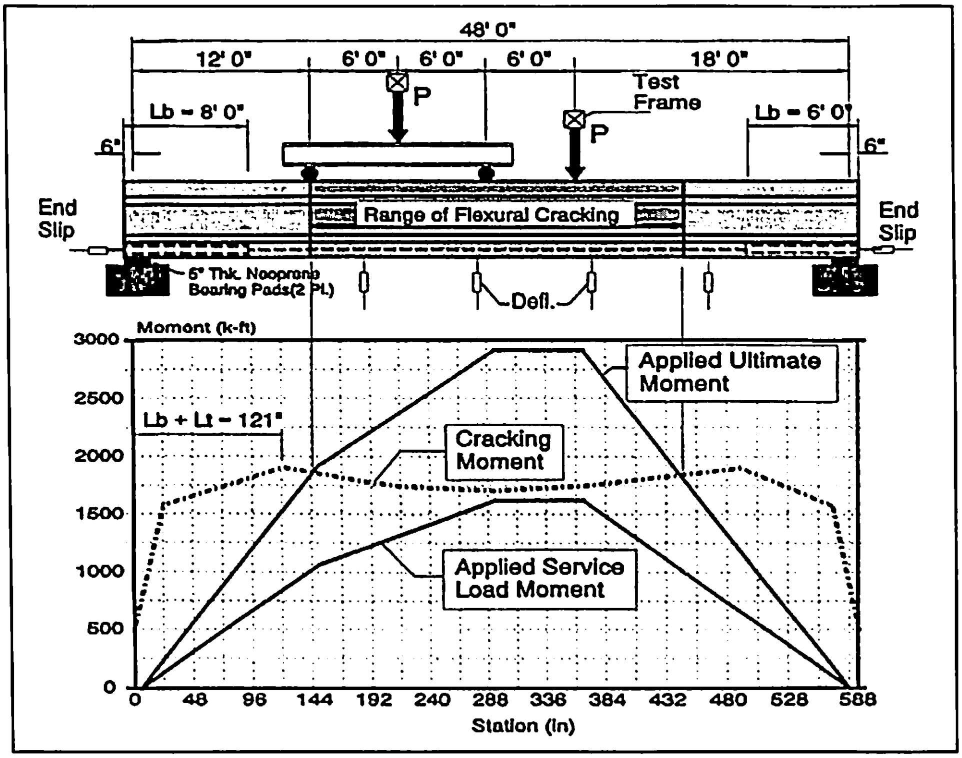


Figure 2.3. Test setup and dimensions (top). Applied moment vs. cracking moment (bottom). (From Russell and Burns 1993)

Since the behavior of the high strength concrete girders matched the behavior of previously investigated normal strength concrete, Russell and Burns concluded that high strength concrete up to 10 ksi could be used within the constraints of the current codes. It was also concluded that debonded strands should be allowed as an alternative to draped strands. Although debonded strands should be allowed, the debond/transfer zone should not extend into the flexural cracking region, and if the ultimate shear exceeds the allowable concrete shear, both horizontal and vertical shear reinforcement should be used.

In two studies, a rational design method to predict debonded strand anchorage failures was investigated. Russell, Burns, and ZumBrunnen (1994) investigated the static loading bond behavior of prestressed concrete beams, and Russell and Burns (1994) investigated fatigue on prestressed beams with debonded strands.

In the first study by Russell, Burns, and ZumBrunnen (1994) a rational design method was tested to determine the accuracy in predicting the failure mode based on the predicted location of cracking. The design aid can be used for concurrent debonding or staggered debonding and predicts the mode of failure based on the embedment length plus debonded length. Figure 2.4 shows the design aid with the results overlaid for the beams tested.

To test the failure prediction model, six flexural static load tests were conducted on four beams. Each beam was 23 in. deep with eight Grade 270 strands with four of the eight strands debonded. Table 2.8 gives the variables and results for the test specimens. The test results given in Table 2.8 and shown on Figure 2.4 demonstrate that the mode of failure was accurately predicted by the behavioral model proposed by the researchers. The behavioral model is based on the prediction of cracking through or near the transfer zone of a debonded strand. If cracking is expected through the transfer zone of a

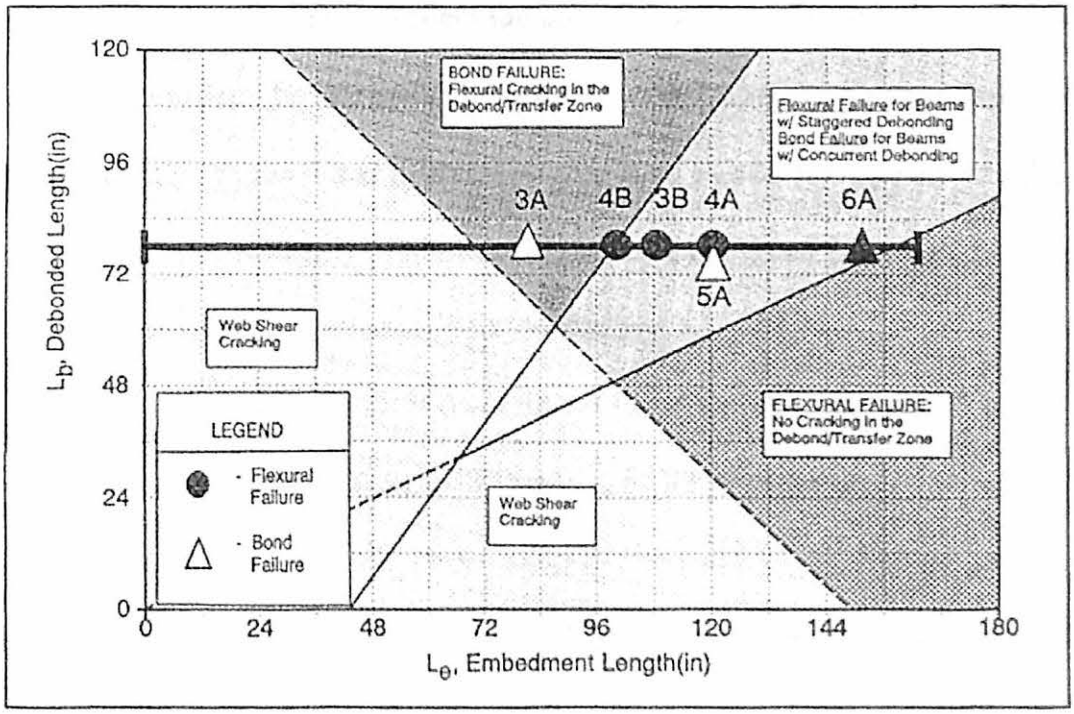


Figure 2.4. Design aid with overlay of results. (From Russell et al. 1994)

Table 2.8. Variables for Test Specimen (From Russell et al. 1994)

Test	Beam Length, L (in.)	Debonded Length, L_b (in.)	Embedment Length, L_e (in.)	Type of Debonding	Failure
DB850-3A	480	2@39, 2@78	80	Staggered	Bond
DB850-3B	480	2@39, 2@78	108	Staggered	Flexure
DB850-4A	480	2@39, 2@78	120	Staggered	Flexure
DB850-4B	480	2@39, 2@78	100	Staggered	Flexure
DB850-5A	480	4@78	120	Concurrent	Bond
DB850-6A	480	4@78	150	Concurrent	Flexure

debonded strand, then bond failure is predicted. Conversely, if cracking does not occur within the transfer zone of a debonded strand, then a flexural failure can be expected. The results demonstrate that the length of embedment alone is insufficient to predict behavior. Staggered debonding allows a more gradual buildup of prestressing forces from the end of the member, thus helping to mitigate the flexural cracking that caused bond failure in the concurrent debonding beam.

The researchers concluded that the provisions in the ACI and AASHTO codes of that time did not adequately reflect the bond behavior of the strands and should be rewritten to reflect the relationship between cracking and anchorage failures. Essentially, the researchers showed that debond/transfer zone should not extend into the regions where flexural cracking is predicted. They also concluded that staggered debonding should be employed over concurrent debonding.

Russell and Burns (1994) continued the research of debonded strands by investigating the effects of repeated loading on the anchorage of debonded strands. The purpose of the investigation was to compare the results from fatigue testing to the results of the static testing and determine what if any differences exist that could be attributed to fatigue testing.

Four prestressed beams were constructed identical to the beams in Russell et al. (1994). Each beam was 23 in. deep with eight Grade 270 strands with four of the eight strands debonded. Six flexural tests were conducted on the beams with the setup shown in Figure 2.3. Each of the beams was initially "precracked" with a static load, and then each beam was subjected to at least one million cyclic loads. The flexural loads cycled from 25 to 100 percent of the equivalent service load. The service load was defined as the load that produces a bottom fiber tension of $6\sqrt{f_c'}$. Static overload tests periodically interrupted the cyclic testing with loads varying from 130 to 160 percent of the service loads. After at least one million cycles, a static load test was conducted to failure. Table 2.9 gives the variables and results for the test specimens. Figure 2.5 shows the results of the tested overlaid on the behavioral model.

The test results given in Table 2.9 and shown on Figure 2.5 confirm that the mode of failure was accurately predicted by the behavioral model proposed in Russell et al. (1994). The researchers concluded that the behavior of debonded strand beams was predictable and reliable. The use of debonded strands is safe as long as flexural cracking is not permitted in the transfer zone of debonded strands. Fatigue loading had only a small detrimental effect on strand anchorage, and beam failure was governed by beam

Table 2.9. Variables for Test Specimen (From Russell and Burns 1994)

Test	Beam Length, L (in.)	Debonded Length, L_b (in.)	Embedment Length, L_e (in.)	Type of Debonding	Failure
DB850-F1A	480	2@39, 2@78	100	Staggered	Fexure
DB850-F1B	480	2@39, 2@78	80	Staggered	Bond
DB850-F2A	480	2@39, 2@78	80	Staggered	Bond
DB850-F2B	480	2@39, 2@78	110	Staggered	Flexure
DB850-F3	480	4@78	120	Concurrent	Bond
DB850-F4	480	4@78	100	Concurrent	Bond

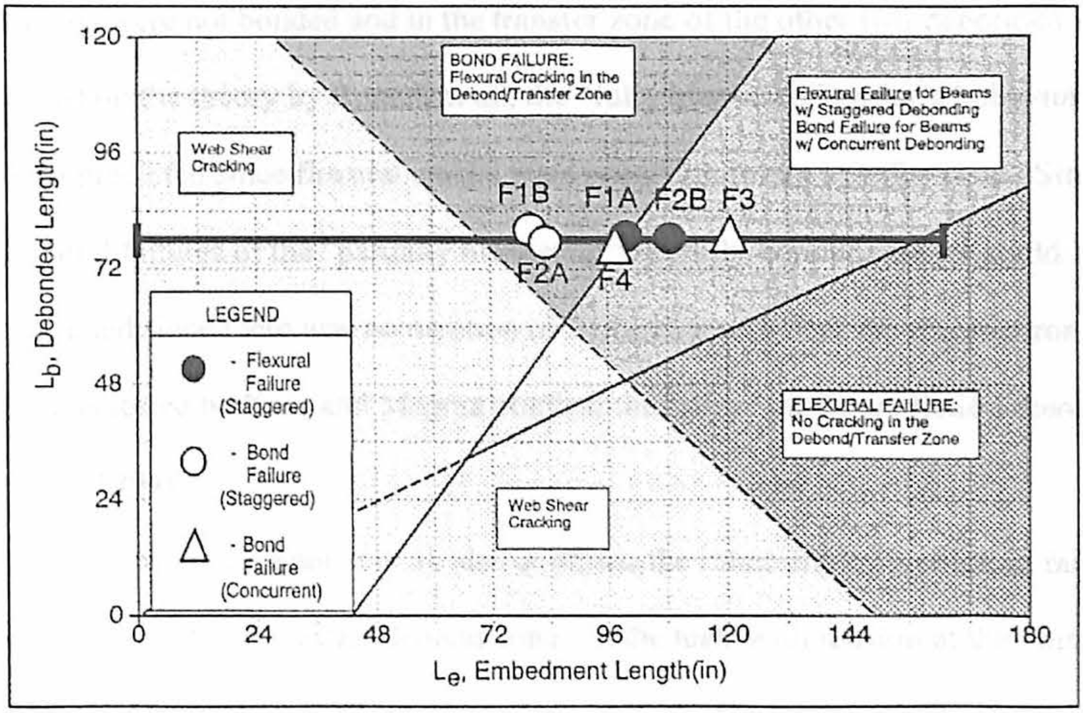


Figure 2.5. Design aid with overlay of results. (From Russell and Burns 1994)

behavior under static conditions. The researchers concluded that debonding should be staggered to increase the beam's resistance to cracking in the transfer zone.

2.2.2.2.3 Analysis of Debonded Strand Failures

An evaluation of the theory by Russell et al. (1994) that the cracking of the specimen will predict the failure mode of beams is made based on the results of the research previously mentioned.

The testing by Kaar and Magura placed a load within the transfer zone of the debonded prestressing strands of the "fully blanketed" girder. The fully bonded and the "partially blanketed" girders did not have loading within the transfer zones of the prestressing strands. Based on the information given in the report, the "fully blanketed" girder had cracks that propagated through the transfer zones of debonded strands. The beams fractured nine feet from the beam end. The fracture occurred where two of the strands were not bonded and in the transfer zone of the other two debonded strands. Based on the theory by Russell et al., the "fully blanketed" girder's behavior could have been predicted since flexural cracks were occurring in the transfer zone. Similarly, the flexural failures of the "partially blanketed" and fully bonded girders could have been predicted since there was no mention of flexural cracks near the transfer zone. The girders tested by Kaar and Magura confirm the failure mode prediction theory by Russell et al. (1994).

Testing by Rabbat et al. also confirms the reliability of predicting failures based on cracking in the transfer/debond zone. In the tests with tension at the bottom fiber, G11 with the longer debonded lengths experienced significant strand slip during Fatigue testing whereas G13 with the shorter debonded lengths did not have strand slip until the static loading to failure. Since all three beams failed in fatigue, it is difficult to make conclusive comparisons to the behavioral model other than the cracking was predicted for G11 to pass through the transfer zones of debonded strands and that strand slips were significant as a result.

The prediction of failure behavior based on cracking is confirmed with testing by Russell and Burns (1993). Specimens DZ2450-1 and DZ2450-2 contained debonded strands and Specimen FZ2450-3 contained draped strands. Prior to repeated load testing, Specimens FZ2450-3 and DZ2450-2 were loaded asymmetrically to develop web shear cracking. The web shear cracking in Specimen FZ2450-3 was not in the transfer zone of the strands since there were not debonded strands. However, web shear cracking in Specimen DZ2450-2 propagated through the transfer zone of the debonded strands, and the data demonstrated that strand slips coincided precisely with the formation of web shear cracks. Specimen FZ2450-3 failed as expected in flexure since the cracks were not within the transfer region of the strands, and Specimen DZ2450-2 failed due to a shear/bond failure as expected since the cracks were within the transfer region of the strands. The last beam, Specimen DZ2450-1 was not statically loaded in shear. Hence it did not experience web shear cracking. During subsequent repeated load testing and the static test to failure, DZ2450-1 did not sustain any flexural or shear cracking in the transfer zones. Consequently, the strands were able to develop their tension at nominal strength; the girder failed in flexure as expected. The research by Russell and Burns further demonstrated that the failure mode can be predicted by the presence of cracking.

The testing by Russell et al. (1994) and Russell and Burns (1994) was specifically designed to test the failure prediction model. As previously discussed, both of the testing programs confirmed the theory that failure modes can be predicted by the location of cracking.

2.2.2.2.4 Conclusions and Recommendations of Debonded Strands

The current code based on the early research by Kaar and Magura (1965) and Rabbat et al. (1979) should be modified to more closely model actual behavior. While the current codes may provide adequate designs in a large majority of cases, some unsafe designs may also be allowed. The cited tests demonstrate that anchorage failure of blanketed strands can be predicted by determining the potential crack locations. Either flexural cracking or web shear cracking propagating through the transfer zone of debonded strands can result in girder failures at less than the calculated nominal strength.

Based on the research cited, anchorage or bond failure will not occur if cracking is limited to the regions beyond the transfer zone of debonded and fully bonded strands. Thus the codes should be modified so as to prevent cracking in the transfer zones of pretensioned strands. For prevention of cracking in the transfer zone, the transfer length used for debonded strands may be taken to be the same as for fully bonded strands. As discussed earlier, this will yield conservative transfer lengths, but given the scatter in measured transfer lengths it will not be inaccurate.

2.2.3 Standardized Testing

Research has been conducted to determine a reliable standardized test to evaluate the bond ability of prestressing strand. These tests include single and multiple strand specimens, concrete and grout specimens, and tensioned and untensioned strand specimens. While the early testing on prestressing strand bond quality was conducted on Grade 250, stress relieved strand, the recent testing on prestressing strand bond quality was conducted on Grade 270, low relaxation strand.

2.2.3.1 Untensioned Multiple Strand Pull-Out Tests

The untensioned multiple strand pull-out test accepted by many researchers and professionals is referred to as the "Moustafa Test." In 1974, Saad Moustafa initially performed simple pull-out tests on Grade 250, stress relieved strand to determine pull-out capacity of lifting hoops. The tests were conducted with eight 1/2 in. strands embedded 18 in. into a large concrete block. The average pull-out force was 38.2 kips with a 3 percent standard deviation. (Logan 1996) The result is shown in Table 2.10.

After lifting hoop failures, Moustafa conducted more tests in 1992. Grade 270, low-relaxation strand from seven manufacturers was tested in the as received and cleaned conditions. Three of the seven strands had capacities greater than the 1974 value, while the other four strands had capacities lower than the previous values. After the strand was cleaned with hydrochloric acid, the three high pull-out strength strands experienced a small decrease in strength, while the four low pull-out strength strands experienced a large increase in strength. (Logan 1996) The results of the testing are shown in Table 2.10.

In 1994, Logan conducted Moustafa pull-out tests when his company's strand manufacturer's plant closed. Except for one weathered strand test, the tests were conducted on as received strand utilizing either calcium stearate or sodium stearate manufacturing processes. One manufacturer supplied the calcium stearate and two sets of the sodium stearate strands, and the other manufacturer supplied three sets of sodium stearate strands, one set of which was weathered before the testing. In the testing, the sodium stearate strands exhibited higher pull-out strengths than the calcium stearate strands. (Logan 1996) The results are shown with Moustafa's in Table 2.10.

Although transfer and development length tests were not conducted in conjuction with the tests by Moustafa and Logan, it can be seen that a wide variety of bond ability exists among different strand manufacturers.

In 1997, Logan proposed acceptance criteria for bond ability based on the Moustafa Test. The strand pull-out tests were conducted on 1/2 in. Grade 270 low relaxation strand with concrete utilizing Type III cement, natural sand, crushed gravel coarse aggregate, and a normal water-reducing admixture. Additionally, end slip and

Table 2.10. Results from Moustafa and Logan (Moustafa 1974, Logan 1996)

Strand		Average Pull-Out strength (kips)	Coefficient of Variation (%)
Moustafa 1974		38.2	3.3
1992 Moustafa As received	A	22.8	44.6
	B	41.2	6.0
	C	41.6	3.6
	D	21.5	13.1
	E	19.6	39.2
	F	42.8	1.5
	G	23.5	21.9
1992 Moustafa Cleaned	A	33.3	7.2
	B	37.4	8.8
	C	38.4	5.2
	D	34.7	7.5
	E	36.5	13.4
	F	39.6	4.8
	G	34.5	9.6
1994 Logan As recieved	2_{CS}	31.2	10.7
	2_{CS}	30.0	8.6
	2_{SS}	41.6	4.1
	2_{SS}	41.0	5.4
	3_{SS}	41.5	7.8
	3_{SS}	39.3	6.2
1994 Logan, Weathered	3_{SS}	41.6	6.0

Notes:
1. The Moustafa 1992 strand designations are alphabetical and each letter represents a different manufacturer.
2. The Logan 1994 designations are given to identify the manufacturer numerically and the drawing lubrication with the subscript letters. "CS" represents calcium stearate lubrication and "SS" represents sodium stearate lubrication.

transfer and development lengths were measured to determine the correlation of actual transfer and development lengths with the proposed Moustafa Test criteria.

Logan utilized procedures used by Moustafa in 1974 and by Moustafa and Logan since 1990. The procedure used follows:

1. 34 in. lengths of prestressing strand were visually inspected, subjected to towel-wipe tests for residue, and tied to light reinforcing bar cages in 2' x 2' x 6'-8" forms such that 18 in. embedment of the strands beyond a 2" tube would occur.
2. 4000 psi one day strength concrete was poured into forms and internally vibrated.
3. The test block was heat cured overnight.
4. The strands were pulled out the morning after casting at 20 kips per minute. The maximum capacity, load at first noticeable movement, pull-out distance at maximum load, and description of failure were recorded.

Table 2.11 gives the results of the maximum pull-out capacity tests.

In order to determine a minimum pull-out value that should be required, Logan conducted transfer length tests and recorded strand end slips. The transfer length and strand slip were measured on 6.5 in. wide x 12 in. deep x 18 ft long beams. The beams were cast in 90 ft beds then saw cut to 18 ft lengths. The measurements were taken at release at one day and at 21 days. The ACI expression for transfer length gives a value of 29 in. The results are given in Table 2.12.

Additionally, development length tests were conducted on the same beam specimens as the transfer and end slip measurements. The simple spans were tested at

Table 2.11. Summary of Pull-Out Results. (Logan 1997)

Strand	Max. Pull-Out Load	
	Avg. (kips)	C. V. (%)
TW	41.6	3.9
TA	40.0	6.9
A	37.7	10.8
B	36.8	11.9
D	11.2	8.6
ER	10.6	4.6

Table 2.12. Summary of Transfer Length Results. (Logan 1997)

Strand group	Pull-Out capacity	At release End slip (in.)	At release Transfer length (in.)	At 21 days End slip (in.)	At 21 days Transfer length (in.)
TW	41.6 kips		.		
Maximum recorded		0.078	24	0.080	25
Average flame-cut		0.068	21	0.068	21
Average saw-cut		0.043	13	0.064	20
Combined average		0.050	15	0.065	20
TA	40.0 kips				
Maximum recorded		0.062	19	0.066	20
Average flame-cut		0.047	14	0.059	18
Average saw-cut		0.041	13	0.056	17
Combined average		0.042	13	0.057	17
A	37.7 kips				
Maximum recorded		0.063	19	0.105	32
Average flame-cut		0.047	14	0.066	20
Average saw-cut		0.049	15	0.081	25
Combined average		0.049	15	0.079	24
B	36.8 kips				
Maximum recorded		0.063	19	0.072	22
Average flame-cut		0.055	17	0.068	21
Average saw-cut		0.045	14	0.058	18
Combined average		0.047	14	0.060	18
D	11.2 kips				
Maximum recorded		0.109	34	0.160	49
Average flame-cut		0.094	29	0.156	48
Average saw-cut		0.074	23	0.122	38
Combined average		0.078	24	0.129	40
ER	10.7 kips				
Maximum recorded		0.172	53	0.188	58
Average flame-cut		0.117	36	0.149	46
Average saw-cut		0.109	34	0.157	48
Combined average		0.111	34	0.156	48

100 percent of the ACI development length value (6.08 ft) and at 80 percent of the ACI development length value (4.83 ft). The cantilever spans were tested at 100 percent of the ACI development length value (6.08 ft) and at 100 percent of the ACI transfer length value (2.42 ft). The results are given in Table 2.13.

Table 2.13. Summary of Development Length Tests. (Logan 1997)

Span	Strand Group					
	TW	TA	A	B	D	ER
Simple Span 12.87 ft $L_e = 6.08$ ft	F/SB	F/SB	F/SB	F/SB	B	B
Cantilever Span 5.75 ft $L_e = 6.08$ ft	F/SB	F/CS	F/CS	F/CS	B	B
Cantilever Span 2.08 ft $L_e = 2.42$ ft	CS/B	F/SB	F/CC	F/CC	B	B
Simple Span 11.37 ft $L_e = 4.83$ ft	F/SB	F/SB	F/SB	F/SB	B	B

Notes:
F/SB = Flexure/strand break
B = Bond
F/CS = Flexure/concrete spall
CS/B = Concrete split/bond
F/CC = Flexure/concrete crush

Based on the Moustafa tests, transfer length measurements, and development length tests, Logan concluded that strands with maximum pull-out values greater than 36 kips had transfer and development lengths less than calculated by the ACI expression. He also concluded that strands with maximum pull-out values less than 12 kips had transfer and development lengths greater than calculated by ACI expressions. As a result, until further testing was conducted, Logan recommended that all 1/2 in. diameter prestressing strand be required to have a maximum pull-out value of at least 36 kips and maximum standard deviation of 10 percent for a six strand sample group. (Logan 1997)

Rose and Russell (1997) investigated three standardized tests to measure the bond performance of prestressing strand. Simple pull-out tests, tensioned pull-out tests, and measured end slips were compared to transfer length data for Grade 270, low relaxation 1/2 in. strands. The tensioned pull-out and strand end slip measurements are discussed in later sections. One strand source was tested with four strand conditions, as received

(CA), cleaned (CC), silane treated (CS), and weathered (CW). An additional two strand sources were tested in their as received condition (AA and BA).

The simple pull-out tests were conducted on strands in 2 x 3 x 4 ft pull-out blocks. Each block contained 12 strands in a 4 x 3 grid, spaced at 9 in. on center with an 18 in. embedment. The pull-out blocks were intended to replicate the tests by Moustafa and Logan using a large concrete block tested with a compressive strength of approximately 4000 psi. The results of the tests are shown in Table 2.14.

For each strand manufacturer and surface condition, three transfer length beams were cast for the transfer length and end slips measurements. Each beam was 6 in. wide x 12 in. deep x 17 ft long except for the silane treated strand beams which were 24 ft long. The beams contained two 1/2 in. strands tensioned to 75 percent of the ultimate strength of the strand in the bottom tensile zone and two #6 reinforcing bars in the top compression zone of the beam. The beams also contained smooth 1/4 in. diameter closed loop stirrups at 6 in. on center at the ends and at 9 in. on center in the middle of the beam. The beams were cast with approximately 4 ksi one day compressive strength and 6 ksi 28 day compressive strength concrete. The results of the tests are shown in Table 2.15.

Table 2.14. Summary of Simple Pull-Out Strengths. (Rose and Russell 1997)

Strand	Pull-Out Strength			
	At 0.005 in. slip		At maximum force	
	Avg. (kips)	C. V. (%)	Avg. (kips)	C. V. (%)
AA	10.4	16.3	15.3	8.5
BA	19.8	8.1	27.4	5.5
CA	23.9	5.9	31.9	5.0
CC	24.2	13.2	33.1	9.4
CS	17.4	25.3	30.7	9.1
CW	36.4	8.2	38.2	3.4

Table 2.15. Summary of Transfer Length. (Rose and Russell 1997)

Strand	Release Compressive Strength (psi)	Transfer Length	
		Avg. (in.)	C. V. (%)
AA	4050	19.1	26.7
BA	4470	15.7	35.7
CA	3990	14.4	21.5
CC	4080	15.4	43.5
CS	4450	65.8	72.9
CW	4690	12.5	25.6

Russell and Paulsgrove (1999a) investigated the repeatability of the Moustafa Test at multiple sites along with the PTI test which will be discussed in a later section. The Moustafa testing was conducted at Florida Wire and Cable (FWC) in two series and at Stresscon in one series. Eleven 1/2 in. strand samples were tested in two of the series and nine strands in the other series. The test procedures required that the concrete compressive strengths were between 3.5 ksi and 5.9 ksi at the time of testing. The concrete strength for the first series at FWC was unknown, the second series at FWC was 3.7 ksi, and the Stresscon series was 5.05 ksi. Russell and Paulsgrove concluded that there was weak correlation between the data for the three series. The results are presented in Table 2.16.

Russell and Paulgrove (1999b) continued their repeatability testing with another round of Moustafa testing and PTI testing. Additionally, they tested another procedure referred to as the NASP Test. The PTI and NASP tests will be discussed in a later section. The Moustafa tests were conducted in one series at three sites, Stresscon, FWC, and University of Oklahoma (OU). The tests were conducted using concrete with compressive strengths between 4100 psi and 4590 psi. Russell and Paulsgrove concluded that the Moustafa test was a good indictor of relative bond quality, but a significant

Table 2.16. Moustafa Test Results. (Russell and Paulsgrove 1999a)

Strand	FWC- Series One		FWC- Series One		Stresscon	
	Average (ksi)	C. V. (%)	Average (ksi)	C. V. (%)	Average (ksi)	C. V. (%)
A	34.2	7.8	32.7	6.4	38.0	3.3
B	33.6	9.8	33.2	4.6	36.3	3.0
C	32.1	6.5	28.6	9.0	33.9	4.8
D	25.0	13.8	21.1	11.4	28.7	10.8
E	38.4	4.4	34.8	10.1	33.2	13.4
F	29.3	5.8	29.5	5.4	35.6	7.5
G	32.8	9.3	28.5	10.4	34.2	9.8
H	28.0	10.7	24.7	7.4	29.7	13.5
I	29.0	5.3	25.3	6.2	33.0	8.7
IN			32.1	10.5	36.9	6.5
DD			13.0	4.3	15.5	13.5

difference in pull-out values could be seen between the FWC test and the other two test sites. As a result, none of the FWC strands would have been considered to pass the requirements recommended by Logan in 1997. The results are shown in Table 2.17. As discussed earlier, Brown (2003) conducted Moustafa tests. The tests were conducted at two sites. The concrete strengths were 4270 psi for the tests conducted at Florida Wire and Cable and 3580 to 4970 psi for the tests conducted at OU. The results are given in Table 2.18.

Table 2.17. Moustafa Test Results. (Russell and Paulsgrove 1999b)

Strand	OU		FWC		Stresscon	
	Average (ksi)	C. V. (%)	Average (ksi)	C. V. (%)	Average (ksi)	C. V. (%)
A	32.7	12.8	27.0	9.1	35.0	2.7
B	39.3	7.1	30.2	10.1	37.4	11.7
C	40.7	7.0	34.2	5.5	39.7	4.7
J	14.0	24.1	14.6	33.4	22.1	12.8
K	37.5	4.6	29.4	5.7	37.0	1.0
M	35.3	12.7	26.6	6.6	37.1	3.5
P	40.8	4.2	29.8	14.1	37.2	7.4
W	41.5	3.9	29.9	11.0	37.2	6.3
Z	33.1	5.3	22.5	26.3	31.3	6.2

Table 2.18. Moustafa Test Results. (Brown 2003)

Strand	OU		FWC	
	Average (ksi)	St. Dev. (%)	Average (ksi)	C. V. (%)
AA	33.0	3.2	26.6	21.1
BB	29.5	7.7	28.2	5.4
CC	31.1	7.7	22.2	4.6
DD	36.5	10.5	27.2	6.3
EE	37.4	6.1	26.0	6.8
FF	22.6	4.7	21.0	9.0
GG	26.3	13.8	26.2	2.8
HH	35.0	7.4	25.1	5.6
II	18.9	11.7	13.4	13.0
JJ	25.1	9.7	31.9	9.1

Although the Moustafa Test does produce repeatable results for relative bond ability at different sites, the test can not be used as a standardized test as it is run at this time. Before the test can be used as a standardized test, the variability of the maximum pull-out capacities from site to site must be addressed.

2.2.3.2 Untensioned Single Strand Pull-Out Tests

Untensioned single strand pull-out tests have taken many forms over the years. The individual researchers have not only varied the shape and size of the test specimen they have also varied the concrete from typical concrete mixtures to grouts and mortars. The two tests that appear to have promise to become the standardized test of choice are the Post-tensioning Institute (PTI) Test and the North American Strand Producers (NASP) Test.

Like the Moustafa test, the early single strand pull-out tests were not conducted as a proposed standard bond ability test, but as a test for actual untensioned strand purposes. Salmons and McCrate (1977) reported such results from untensioned bond tests. The purpose of the experiments was to develop design equations for untensioned prestressing strand used in precast elements. The testing was conducted on frayed and unfrayed straight strands and unfrayed bent strands with strand diameters of 3/8, 7/16, 1/2 , and 0.6 in. The concrete compressive strengths ranged from 3750 to 6900 psi. The results of the test indicate that the steel stress at general slip varied linearly with embedment length and the concrete strength had no apparent effect on the bond characteristics of untensioned strand prior to general slip.

Brearley and Johnston (1990) conducted a study on pull-out testing utilizing epoxy-coated and uncoated Grade 270 low relaxation strand with diameters of 3/8, 1/2, and 0.6 in. The testing was conducted on 8 x 8 x12 in. specimens with untensioned strand centered in the 8 x 8 direction and running parallel to the 12 in. length of the specimen. The concrete used had a design strength of 4,000 psi at 4-7 days and 5,000 psi minimum at 28 days. Based on the testing, Brearley and Johnston concluded that the pull-out test was not a good predictor of the transfer bond stress, but the pull-out bond stresses did vary in a pattern reflecting the actual strand bond performance.

In an effort to determine a standardized test for strand used in anchors, Hyatt, Dube, and Bawden (1994) tested 0.6 in. diameter strands with a test later adapted by the Post-Tensioning Institute (PTI). The PTI Test is conducted on uncleaned, undisturbed strand cast in an 18 in. grout cylinder with a 5 in. outer diameter and a 1/8 in. wall thickness. The bond is broken on the bottom two inches of the strand in the cylinder by means of tape. The grout is made with a 0.45 water to cement ratio. The testing is conducted when the grout reaches a strength of 3500 to 4000 psi as determined by 2 in. cubes. A force is applied to the strand at 0.10 in./minute until the unloaded end of the strand shows a displacement of 0.01 in. Three tests are required with an average force of

8000 lbf and a minimum value not more than 13 percent lower. The results from Hyatt et al. are given in Table 2.19.

Russell and Paulsgrove (1999a) reported on one series of tests conducted at FWC utilizing the PTI method on 1/2 in. strands from nine manufacturers. The test was conducted on mortar strengths between 3.74 ksi and 3.985 ksi. The pull-out force at 0.01 in. free end slip was recorded. The PTI test did correlate with the results on the same strands in Moustafa testing. The coefficient of determination for the PTI test was 0.61 to 0.84 when compared to the Moustafa tests. The results are given in Table 2.20.

Russell and Paulsgrove (1999b) continued their investigation into the repeatability of standard testing with the PTI Test. Additionally, they conducted a test, the NASP Pull-out Test, which is similar to the PTI test except a sand-cement mortar is used instead

Table 2.19. PTI Pull-Out Results. (Hyett et al. 1994)

Strand	Pull-Out at 0.01 in. Free End Slip	
	Average (kips)	C. V. (%)
A	11.9	10.2
B	4.0	11.8
C	3.4	6.0
D	7.1	9.6
E	9.0	7.7
F	9.1	13.9
G	8.4	12.4

Table 2.20. PTI Pull-Out Force Results. (Russell and Paulsgrove 1999a)

Strand	Average (K)	C. V. (%)
A	11.9	6.7
B	14.2	24.3
C	11.8	8.8
D	4.25	21.0
E	12.6	17.6
F	11.5	9.4
G	10.9	5.3
H	9.2	16.3
I	7.7	12.4

of a cement grout and the pull-out force is reported at 0.01 in. free end slip, 0.10 in. free end slip, and maximum. The mortar used had a sand to cement ratio of 2:1 and a 0.45 water to cement ratio. The NASP tests were conducted with Type III cement except for the first series at FWC which was conducted with Type I cement. The tests were conducted at OU and FWC. The results indicate that for both the PTI and NASP test the 0.10 in. and maximum pull-out forces may be better for determining bond acceptance since the range of values is wider. Russell and Paulsgrove concluded that the NASP test demonstrated less variation in data between test sites. Due to the larger range in values, they recommend the pull-out force at 0.10 in. of slip be used in both tests. The results are shown in Table 2.21, 2.22, 2.23, and 2.24.

Continuing the research by Russell and Paulsgrove, Brown (2003) conducted NASP and PTI Tests. The procedures are identical to those previously discussed. The results are summarized in Table 2.25 and 2.26. Data from Brown's research will be analyzed with results from this study. The raw data that will analyzed from Brown is given in Appendix A. The proposed NASP Test procedure resulting from the testing is given in Appendix B.

2.2.3.3 Tensioned Single Strand Pull-Out Tests

Since many researchers feel strand should be tensioned prior to pull-out testing in order to account for the Hoyer effect, researchers have conducted tensioned pull-out testing. The tests to be discussed have been performed using "normal" concrete mixtures with no admixtures.

Table 2.21. PTI Test Results. (Russell and Paulsgrove 1999b)

Strand	Maximum Pull-Out Force				0.10 in. Slip Pull-Out Force				0.01 in. Slip Pull-Out Force			
	OU		FWC		OU		FWC		OU		FWC	
	Avg. (K)	St. Dev. (%)	Avg. (K)	St. Dev. (%)	Avg. (K)	St. Dev. (%)	Avg. (K)	St. Dev. (%)	Avg. (K)	St. Dev. (%)	Avg. (K)	St. Dev. (%)
A	12.6	6.7	14.2	5.9	10.9	11.9	13.1	12.0	6.3	34.9	9.6	24.8
B	11.7	7.6	14.9	8.7	9.5	12.0	12.1	10.6	5.2	25.5	9.5	12.0
C	13.4	9.1	15.3	13.1	11.5	12.1	13.7	15.6	5.6	23.9	10.5	12.0
J	5.1	18.4	6.0	14.1	2.6	21.1	4.1	10.8	2.0	7.8	3.5	15.9
K	12.6	9.9	14.5	12.7	11.1	20.0	12.5	17.8	6.9	30.3	9.0	24.7
M	12.0	11.4	14.1	7.2	9.5	18.9	11.6	12.7	4.4	46.3	7.5	23.4
P	13.8	7.5	16.6	10.8	11.6	10.3	13.2	19.6	6.3	29.6	9.1	39.6
W	11.0	10.6	12.7	17.4	9.4	13.2	8.9	16.5	5.6	19.6	7.0	12.5
Z	9.8	8.4	8.0	10.4	6.3	15.7	6.0	12.2	3.5	21.3	5.5	21.7

Table 2.22. NASP Test Maximum Pull-Out Force. (Russell and Paulsgrove 1999b)

Strand	OU Series I		OU Series II		FWC Series I		FWC Series II	
	Average (K)	St. Dev. (%)	Average (K)	St. Dev. (%)	Average (K)	St. Dev. (%)	Average (K)	St. Dev. (%)
A	19.2	9.7	16.6	8.9	14.7	29.3	16.6	18.4
B	15.2	11.4	14.4	11.8	10.1	38.8	12.5	23.3
C	21.6	7.8	18.5	12.3	15.0	24.3	20.4	17.6
J	4.9	19.7	4.4	22.0	3.5	32.3	6.9	22.4
K	15.8	11.5	15.6	9.4	11.2	24.8	13.4	10.4
M	17.9	11.0	16.2	6.7	12.5	25.3	14.0	17.3
P	21.1	6.8	18.3	6.5	15.6	19.6	17.7	27.6
W	13.2	14.8	12.6	9.0	7.8	27.2	12.5	25.2
Z	7.9	16.3	9.1	13.7	6.1	26.9	11.3	17.0

Table 2.23. NASP Test 0.10 in. Slip Pull-Out Force. (Russell and Paulsgrove 1999b)

Strand	OU Series I		OU Series II		FWC Series I		FWC Series II	
	Average (K)	St. Dev. (%)	Average (K)	St. Dev. (%)	Average (K)	St. Dev. (%)	Average (K)	St. Dev. (%)
A	17.7	11.8	15.9	7.1	12.5	27.4	14.5	18.2
B	11.8	10.2	11.8	23.2	8.0	33.6	10.2	19.0
C	19.6	10.0	17.8	12.4	12.9	20.6	17.0	19.1
J	2.6	21.7	3.3	24.0	2.8	23.2	5.0	25.4
K	13.8	12.4	14.6	11.2	9.3	29.9	11.8	9.7
M	14.9	13.5	14.9	4.6	10.7	23.3	12.2	13.4
P	17.1	9.6	17.3	6.9	12.5	14.2	15.1	23.5
W	10.4	14.9	11.3	11.0	6.8	24.7	9.7	14.5
Z	5.7	21.0	7.9	13.0	5.2	26.2	7.8	17.3

Table 2.24. NASP Test 0.01 in. Slip Pull-Out Force. (Russell and Paulsgrove 1999b)

Strand	OU Series I		OU Series II		FWC Series I		FWC Series II	
	Average (K)	St. Dev. (%)	Average (K)	St. Dev. (%)	Average (K)	St. Dev. (%)	Average (K)	St. Dev. (%)
A	15.0	16.3	11.2	37.4	9.9	28.6	11.0	15
B	9.7	10.0	9.5	23.7	7.3	32.5	8.4	15.5
C	15.5	8.8	14.4	15.4	11.3	15.9	14.1	18.2
J	2.3	31.4	3.3	28.4	3.4	31.4	4.6	19.2
K	11.1	18.7	11.9	14.5	8.2	34.2	9.1	8.9
M	11.2	24.8	11.9	6.7	9.1	29.7	10.3	10.9
P	9.0	14.7	13.7	10.0	8.8	17.2	12.4	16.9
W	8.9	8.8	9.8	10.4	6.1	17.9	7.8	9.3
Z	5.6	22.6	7.4	7.5	5.3	25.0	6.9	15.0

Table 2.25. PTI Test Results. (Brown 2003)

Strand	0.10 in. Slip Pull-Out Force				0.01 in. Slip Pull-Out Force			
	OU		FWC		OU		FWC	
	Avg. (K)	St. Dev. (%)	Avg. (K)	St. Dev. (%)	Avg. (K)	St. Dev. (%)	Avg. (K)	St. Dev. (%)
AA	9.6	6.0	11.2	16.9	4.5	8.7	7.8	28.4
BB	6.7	9.6	7.1	21.2	4.9	10.9	5.7	14.2
CC	5.6	15.3	7.9	28.1	4.5	12.6	5.9	17.0
DD	6.1	19.7	11.0	24.6	4.4	13.2	7.2	21.2
EE	6.9	7.7	10.1	22.2	4.9	11.0	7.0	16.4
FF	4.6	11.9	7.3	39.4	4.6	6.3	7.8	11.5
GG	7.2	5.2	7.3	15.4	3.6	10.6	4.3	17.9
HH	9.0	14.0	8.6	18.9	6.9	12.0	6.7	28.6
II	5.5	8.1	5.0	58.5	4.5	10.8	4.9	29.5
JJ	7.4	9.6	10.7	67.5	5.5	10.3	7.9	37.6

Table 2.26. NASP Test Results. (Brown 2003)

Strand	0.10 in. Slip Pull-Out Force				0.01 in. Slip Pull-Out Force			
	OU		FWC		OU		FWC	
	Avg. (K)	St. Dev. (%)	Avg. (K)	St. Dev. (%)	Avg. (K)	St. Dev. (%)	Avg. (K)	St. Dev. (%)
AA	13.9	9.0	16.0	16.9	9.7	7.0	11.6	14.2
BB	6.8	10.6	10.4	9.3	5.4	10.7	8.9	11.7
CC	9.9	25.2	8.8	15.7	7.7	22.9	8.0	15.2
DD	14.3	4.2	15.3	11.5	10.9	7.7	11.5	14.6
EE	14.1	4.2	16.0	26.0	10.0	7.4	9.7	15.6
FF	6.3	6.5	8.3	15.6	7.3	5.5	8.7	14.9
GG	7.2	14.0	12.4	10.1	5.0	10.4	9.1	13.0
HH	11.1	9.0	10.3	15.9	9.5	9.1	8.1	19.6
II	3.0	10.7	5.3	15.9	3.7	6.6	5.7	13.0
JJ	19.7	7.1	17.6	17.9	14.9	5.9	13.0	23.0

Cousins, Badeaux, and Moustafa (1992) proposed a single tensioned strand test specimen for determining bond characteristics of prestressing strand. The proposed standardized test was conducted on uncoated and epoxy coated Grade 270, low relaxation prestressing strand with diameters of 3/8, 1/2, and 0.6 in. The uncoated strand was to be used in a rust-free state, but the researchers noted that the 3/8 in. diameter strands were slightly rusted. Using a 4000 psi 3 day compressive strength concrete mixture and no admixtures, the 8 x 8 x 12 in. specimens were cast. The tests were conducted in a 10 ft reaction frame with the specimen, a hydraulic actuator, and two chucks. The reference transfer length specimens were 3 ½ x 3 ½ in. x 8 ft with an identical concrete mixture with 3/8 in. coated and uncoated strands.

The standardized test procedure proposed and followed by Cousins et al. is as follows:

1. Strand is prestressed to desired level. (12.5 k for 3/8 in. strand, 25 k for 1/2 in. strand, and 33 k for 0.6 in. strand.)
2. Concrete is placed in formwork around the strand within 24 hours of strand pretensioning.
3. The concrete is cured for 3 days.
4. The test is performed by using the hydraulic actuator to force the block off the strand and the force in strand versus strand slip is recorded.

The test results are shown in Table 2.27 with the values ignored that were ignored in the paper. The value U_s is the average bond stress over the length of the bond, and the value U'_s is U_s divided by $\sqrt{f'_{ci}}$.

To correlate the data to transfer lengths, the 3 ½ x 3 ½ in. x 8 ft specimens were fabricated and tested with 3/8 in. coated and uncoated strand. The results from these tests are shown in Table 2.28. (Cousins et al. 1992)

One should note that the failure load of the standardized test and the transfer length of the specimens do not demonstrate a direct relationship. While the failure load for the 3/8 in. coated and uncoated strands are essentially equal, the transfer length of the uncoated strand is approximately twice that of the coated strand.

Table 2.27. Summary of Test Results. (Cousins et al. 1992)

Strand diameter and surface condition	P_i Avg. (kips)	Failure load Avg. (kips)	f'_{ci} Avg. (psi)	U_s Avg. (psi)	U'_s Avg. (psi)	U'_s St. Dev. (%)
3/8 in. - Coated	13.0	15.5	3820	1096	17.7	3.2
3/8 in. - Uncoated	12.5	14.5	3760	1026	16.7	12.4
1/2 in. - Coated	23.4	19.7	3740	1391	22.7	10.6
1/2 in. - Uncoated	21.7	7.8	3490	554	9.4	18.4
0.6 in. - Coated	32.9	21.3	3260	1509	26.4	31.9
0.6 in. – Uncoated	33.9	12.5	3857	884	14.2	7.7

Table 2.28. Results from Transfer Length Prisms. (Cousins et al. 1992)

Strand diameter and surface condition	Transfer length		U_t	U'_t	
	Avg. (in.)	St. Dev. (%)	Avg. (psi)	Avg. (psi)	St. Dev. (%)
3/8 in. - Coated	17.5	14.4	776	12.8	14.4
3/8 in. - Uncoated	35.75	4.2	389	6.42	3.6

Along with untensioned pull-out tests and end slip measurements, Rose and Russell (1997) conducted tensioned pull-out tests. The tensioned pull-out test specimens were 5.5 x 5.5 x 12 in. with a strand tensioned to an unspecified value. The tension was gradually released on one side of the specimen, and then the strand was pulled out of the specimen. The results of the test are shown in Table 2.29.

Based on the testing to date, tension pull-out tests result in more complicated testing procedures that do not yield results better than other less complicated tests.

2.2.3.4 End Slip Measurement

Many researchers have noted the direct relationship that appears to be present between transfer lengths and end slips. The end slip measurements of a specimen are typically taken at the same time as the transfer length measurements and on the same specimen.

Table 2.29. Summary of Tensioned Pull-Out Strengths. (Rose and Russell 1997)

Strand	Pull-Out strength				Transfer Length	
	At 0.005 in. slip		At maximum force			
	Avg. (kips)	St. Dev. (%)	Avg. (kips)	St. Dev. (%)	Avg. (in.)	St. Dev. (%)
AA	5.8	1.7	12.4	1.6	19.1	26.7
BA	14.8	23.6	21.2	6.6	15.7	35.7
CA	7.5		23.9		14.4	21.5
CC	6.1	11.5	10.4	2.9	15.4	43.5
CS	3.2	6.3	12.0	2.5	65.8	72.9
CW	No slip observed		27.8	0.7	12.5	25.6

Anderson and Anderson (1976) conducted flexural bond tests on 36 factory produced saw-cut hollow core planks from five manufacturers on the West Coast. The end slips of the prestressing strands were measured, and the beams were tested to failure. Based on the results, Anderson and Anderson determined that end slip measurement is a reliable assurance criterion for flexural bond. An end slip of $(f_{si}d_b)/950$, where f_{si} is the initial stress in the prestressing steel and d_b is the nominal diameter of the prestressing strand, was determined to be the upper limit of safe end slips.

In addition to untensioned and tensioned single strand tests, Rose and Russell (1997) measured the strand end slip of transfer length specimens. The results are shown in Table 2.30. Based on their research, Rose and Russell concluded that for their data the measured end slip was the most conclusive identifier of pretensioned bond. In the testing program, the simple pull-out test data was determined inconclusive, and the tensioned pull-out test was difficult to perform and its results were inconsistent with other tests. (Rose and Russell 1997)

Although measured end slip has proven to have a correlation with transfer length, the end slip measurements can not be taken until the specimens are cast. It should also be

Table 2.30. Summary of End Slip Measurements (Rose and Russell 1997)

Strand	End Slip		Transfer Length	
	Avg. (in)	St. Dev. (%)	Avg. (in.)	St. Dev. (%)
AA	0.066	10.6	19.1	26.7
BA	0.058	17.2	15.7	35.7
CA	0.055	12.7	14.4	21.5
CC	0.047	19.1	15.4	43.5
CS	0.265	59.2	65.8	72.9
CW	0.044	6.8	12.5	25.6

noted that the end slip measurements were taken on the same specimens as the transfer length tests instead of being a stand alone test for strand bond quality.

2.2.3.5 Summary and Conclusions of Standardized Testing

Based on the research to date, the NASP test and end slip measurements appear to be the most reliable means of predicting bond behavior. Since a standardized test should be able to be a stand alone test, the NASP test appears to be the most promising for standardized testing. This research program is useful toward investigating the NASP bond test to refine the procedure to a reliable standardized test.

Chapter 3

EXPERIMENTAL PROGRAM

3.1 INTRODUCTION

The experimental program was designed to produce the following:

- Concrete mixtures of three desired strength combinations,
- Mortars with varying strengths,
- Results that determine the effect of mortar strength on the bond pull-out values of the NASP test.
- Results that determine the effect of testing frame stiffness on the bond pull-out values of the NASP test

The procedures for this project will be broken into two main categories, preliminary batching and NASP testing. The preliminary batching was required to develop mixture designs to achieve desired strengths and workability for concrete mixtures used in future research and for mortar mixtures used in the NASP mortar tests. The results of the NASP mortar testing will be used to analyze the effect of mortar strength and test frame stiffness on the bond strength results of the NASP test.

3.2 MATERIALS

The materials used in the experimental procedures were Type III cement, coarse aggregate, fine aggregate, water, fly ash, silica fume, and admixtures.

The Type III cement was supplied by Lafarge North America from their plant in Tulsa, Oklahoma. The cement is a Portland Type III cement meeting the specifications in ASTM C 150. The chemical analysis of the cement is given in Appendix C.

Table 3.1. Aggregate Properties

Material	Specific Gravity	Absorption Content	Fineness Modulus	ASTM Designation
Original Fine Agg.	2.59	0.5%	2.93	
Original Coarse Agg.	2.63	1.07%		#6
New Dolese Fine Agg.	2.63	0.5%	2.63	
New Dolese Coarse Agg.	2.67	0.8%		#8

The coarse and fine aggregates for the initial concrete and mortar batching were from an unknown source, and the aggregates used in the remainder of the batching and in the NASP testing were provided by Dolese Brothers Company from their Stillwater, Oklahoma and Guthrie, Oklahoma plants. Detailed information on the aggregate gradations can be found in Appendix C, and the aggregate properties are summarized in Table 3.1.

The fly ash was supplied by Mineral Solutions from Oklahoma Gas and Electric's Sooner power plant in Red Rock, Oklahoma. The fly ash was Class C.

The silica fume was Rheomac SF100 supplied by Master Builders, a division of Degussa, from their Plano, Texas office. Rheomac SF100 is a dry, densified mineral admixture that meets ASTM C 1240.

The other admixtures used include a mid range water reducer (WR), a water reducing set retarder (SR), and three high range water reducers (HRWR). All of the admixtures were provided by Master Builders from their Plano office. The mid range water reducer was Polyheed 997. The water reducing set retarder was Pozzolith 100 XR. The high range water reducers were Glenium 3200 HES, Glenium 3030 NS, and Rheobuild 1000. Dosage ranges often exceeded those recommended by the manufacturer to overcome problems with workability.

3.3 PRELIMINARY BATCHING

The first component of the research project was preliminary batching of concrete and mortar. The concrete preliminary batching will be used for NASP concrete tests and transfer and development length specimens, both of which will be conducted in subsequent research. The mortar trial batching was used for the NASP mortar tests.

3.3.1 Concrete Batching

The concrete batching was conducted in a pan mixer. The concrete was mixed and 4 by 8 in. test cylinders were made according to ASTM C 192. The concrete temperature according to ASTM C 1064, the slump according to ASTM C 143, the air-content according to ASTM C 231, and the unit weight according to ASTM C 138 were recorded during batching. The test specimens were cured according to ASTM C 192. The compressive and tensile tests were conducted according to ASTM C 39 and C 496. The compressive tests were conducted on four cylinders at 1, 3, 7, 28, and 56 days. The tensile splitting tests were conducted on two cylinders at 1 and 28 days.

3.3.1.1 Target Properties

The ultimate goal of the concrete mixtures was to reach three desired compressive strength combinations with a workable mixture possessing a slump of 6 to 8 in. One combination's target strengths were 6,000 psi one day strength and 10,000 psi 28 or 56 day strength. The next combination's target strengths were 8,000 psi one day strength and 14,000 psi 28 or 56 day strength. The last combination's target strengths were

10,000 psi one day strength and 18,000 psi 28 or 56 day strength. The target mixtures will be referred to by their one day strength.

3.3.1.2 Selecting Concrete Mixtures

First, mixtures were selected based on the desired properties from previous work conducted by Hale (2002) and Freyne (2003). After selecting mixtures, one was chosen to begin the batching.

The first mixture was based on a batch reported by Hale (2000). Although the mixture was intended to be refined for the 6,000 psi mixture, the mixture became the basis for the 10,000 psi mixture since the first batch resulted in stronger compressive strengths than expected. The first task was to determine how much HRWR would be required to reach the desired slump. The HRWR used in the mixtures was Glenium 3200 HES. In all of these mixtures, the original aggregates were used in batching. After multiple batches were made, it was determined that a different high range water reducer was needed due to workability and volatility of slump as the amount of HRWR was varied. The mixture designs are given in Table 3.2. Appendix D contains all mixture proportions, fresh properties, and hardened properties.

The next mix design batched was based on a mix reported by Freyne (2003). This mixture was intended for the 6,000 psi mixture. The mixture was batched with varied HRWR using the Glenium 3200 HES. The water to cement ratio was lowered to try to achieve an 8,000 psi mixture. In all of the mixtures, the original aggregates were used in batching. Like the first batches, a different HRWR was desired. The mixture designs are given in Table 3.3.

Table 3.2. Glenium 3200 HES Concrete Mixtures with 800 PCY Cement.

	8-28-1	8-28-2	8-28-3	8-28-4	8-28-5	8-28-6
Cement (PCY)	888.8	795.4	795.4	795.4	800	800
Fly Ash (PCY)	-	-	-	-	-	-
Silica Fume (PCY)	-	-	-	-	-	-
Coarse Agg. (PCY)	1608.0	1795.7	1795.7	1795.7	1806.1	1806.1
Fine Agg. (PCY)	1400.5	1252.5	1252.5	1252.5	1259.8	1259.8
Water (PCY)	253	222.7	222.7	222.7	224	224
WR (fl. oz/cwt)	-	-	-	-	3	-
HRWR(fl. oz/cwt)	22.5	15	12.5	10	10	12.5
SR(fl. oz/cwt)	-	-	-	-	-	3
w/cm	0.285	0.280	0.280	0.280	0.28	0.28

Table 3.3. Glenium 3200 HES Concrete Mixtures with 650 PCY Cement.

	6.5-40-1	6.5-40-2	6.5-40-3	6.5-36-1
Cement (PCY)	650	650	650	650
Fly Ash (PCY)	-	-	-	-
Silica Fume (PCY)	-	-	-	-
Coarse Agg. (PCY)	1784.6	1784.6	1784.6	1784.6
Fine Agg. (PCY)	1311.0	1311.0	1311.0	1378.4
Water (PCY)	260	260	260	234
WR (fl. oz/cwt)	3	3	3	21
HRWR(fl. oz/cwt)	6	3	4.5	-
SR(fl. oz/cwt)	-	-	-	-
w/cm	0.40	0.40	0.40	0.36

When the new admixtures and aggregates arrived, the first batches were made to determine which HRWR was to be used. After these batches, Glenium 3030 NS was used for the remainder of the batches. The mixture designs are given in Table 3.4.

In the final batching, the original mixture design with 800 pcy of cement was slightly modified. For the batching, the water to cement ratio was varied, and fly ash and silica fume replaced percentages of cement. The goal of this was to find workable mixtures for all strength ranges. The mixtures are given in Table 3.5 and 3.6.

Table 3.4. Concrete Mixtures to Determine HRWR.

	R-8-36-1	G3030-8-36-1
Cement (PCY)	800	800
Fly Ash (PCY)	-	-
Silica Fume (PCY)	-	-
Coarse Agg. (PCY)	1800	1800
Fine Agg. (PCY)	1144	1144
Water (PCY)	288	288
WR (fl. oz/cwt)	-	-
HRWR(fl. oz/cwt)	13	7.5
SR(fl. oz/cwt)	-	-
w/cm	0.36	0.36
Notes: R = Rheobuild 1000 G3030 = Glenium 3030NS		

Table 3.5. Glenium 3030NS Concrete Mixtures Varying Fly Ash and Silica Fume.

	G3030-						
	8-32-1	8-32-2	8-32-3	8-32-4	8-32-5	8-32-6	8-32-7
Cement (PCY)	800	760	720	720	720	720	720
Fly Ash (PCY)	-	-	-	80	-	40	-
Silica Fume (PCY)	-	40	80	-	80	40	80
Coarse Agg. (PCY)	1800	1800	1800	1800	1800	1800	1800
Fine Agg. (PCY)	1228	1228	1228	1228	1228	1228	1228
Water (PCY)	256	256	256	256	256	256	256
WR (fl. oz/cwt)	-	-	-	-	-	-	-
HRWR(fl. oz/cwt)	12.5	14.2	15	18.5	15	13.5	19
SR(fl. oz/cwt)	3	3	3	3	3	3	3
w/cem	0.32	0.32	0.32	0.32	0.32	0.32	0.32

Table 3.6. Glenium 3030NS Concrete Mixtures Varying Water to Cement Ratio.

	G3030-8-30-1	G3030-8-28-1
Cement (PCY)	800	800
Fly Ash (PCY)	-	-
Silica Fume (PCY)	-	-
Coarse Agg. (PCY)	1800	1800
Fine Agg. (PCY)	1270	1312
Water (PCY)	240	224
WR (fl. oz/cwt)	-	-
HRWR(fl. oz/cwt)	20	22.5
SR(fl. oz/cwt)	3	3
w/cem	0.30	0.28

3.3.2 Mortar Batching

The mortar mixtures were batched in a bowl mixer meeting ASTM C 305. The mortar was mixed and 2 in. cubes were made according to ASTM C 305. The flow of the mortar was determined according to ASTM C 1437 on a flow table meeting the standards of ASTM C 230. The air content was determined based on ASTM C 185. The test specimens cured according to ASTM C 305. The compressive tests were conducted on four cubes at 1, 3, 7, and 28 days according to ASTM C 109.

3.3.2.1 Target Properties

The ultimate goal of the mortar mixtures was to reach three desired one day compressive strengths. One mixture design was to match the proportions previously used in the NASP testing. The other mixture designs were to achieve one day strengths on either side of the original batch.

3.3.2.2 Selecting Mortar Mixtures

The first mortar mixture was batched to match the proportions of previous NASP testing. The batch had a 0.45 water to cement ratio and a 2:1 fine aggregate to cement ratio. The mixture was batched several times with the original aggregate. Then it was batched with the new aggregate. The original aggregate mortar batching quantities are given in Table 3.7, and the new aggregate mortar batching quantities are given in Table 3.8.

Since mortar mixture strengths can vary based on aggregate source, cement source, and curing conditions, a range of mortar strengths were desired. Mortars were

Table 3.7. Original Aggregate Mortar Batching Quantities

	NASP-A	NASP-B	NASP-C	NASP-D	NASP-E	NASP-F
Cement (PCF)	39.6	39.6	39.7	39.7	39.7	39.7
Fine Agg. (PCF)	79.6	79.6	79.4	79.4	79.4	79.4
Water (PCF)	17.8	17.8	17.9	17.9	17.9	17.9
w/c	0.45	0.45	0.45	0.45	0.45	0.45

Table 3.8. New Aggregate Mortar Batching Quantities.

	NASP-AA	NASP-BB	NASP-CC	NASP-DD	NASP-EE
Cement (PCF)	39.7	39.7	29.7	38.8	41.4
Fine Agg. (PCF)	79.4	80.0	74.2	77.5	82.8
Water (PCF)	17.9	19.1	19.9	19.4	16.5
w/c	0.45	0.48	0.50	0.50	0.40

batched to produce mixtures that have compressive strengths from 4,000 psi to 6,000 psi. First the water to cement ratio was varied, and since the cement quantity per cubic foot was kept constant, the cement to fine aggregate ratio also varied. Then, to keep the mortar mixture more constant with the original proportions, the water to cement ratio was varied, but the fine aggregate to cement ratio was kept at 2:1.

3.3.3 NASP Testing

After initial batching, NASP bond tests were conducted. The bond tests were tested using similar procedures as in the NASP Round Two testing by Russell and Paulsgrove (1999b).

3.3.3.1 Test Variables

The testing was conducted with four mortar mixtures. The mortar mixtures are given in Table 3.9. The mortar mixtures were intended to produce compressive strengths ranging from approximately 4 ksi to 6 ksi.

Table 3.9. NASP Mortar Mixture Designs.

	0.40	0.45	0.475	0.5
Fine Agg. : Cement	2:1	2:1	2:1	2:1
W/C	0.40	0.45	0.475	0.50

The testing was conducted with two testing frames that will be discussed in more detail later in this chapter. One testing frame was intended to be a "stiff" testing frame and one a "flexible" testing frame. The "stiff" testing frame has an axial stiffness of 195.5 x 10^3 k/in. and the "flexible" testing frame has an axial stiffness of 45.6 x 10^3 k/in. Figures 3.1 and 3.2 show the test set-ups. The reported stiffnesses were calculated from the expression *AE/L* for the channels and rods. Table 3.10 gives the test matrix for the NASP testing. The 0.475 water to cement ratio mixture was added after Strand "AA" reached the capacity of the testing machine with the 0.45 water to cement ratio mixture.

3.3.3.2 Preparation of the Test Specimens

The strand specimens conformed to ASTM A 416 and were intended for use in pretensioned or post-tensioned application. Strand specimens for a single test were taken from the same lot or the same reel of prestressing strand. Six strand specimens are required in each test; however, five satisfactory results from individual test results are required for a set to be complete based on the proposed NASP test procedure.

Table 3.10. NASP Test Matrix.

Water/Cement Ratio	Strand AA		Strand FF	
	"Stiff"	"Flexible"	"Stiff"	"Flexible"
0.40			X	X
0.45	X	X	X	X
0.475	X	X		
0.50	X	X	X	X
Note: Each "X" represents a sample size of six tests				

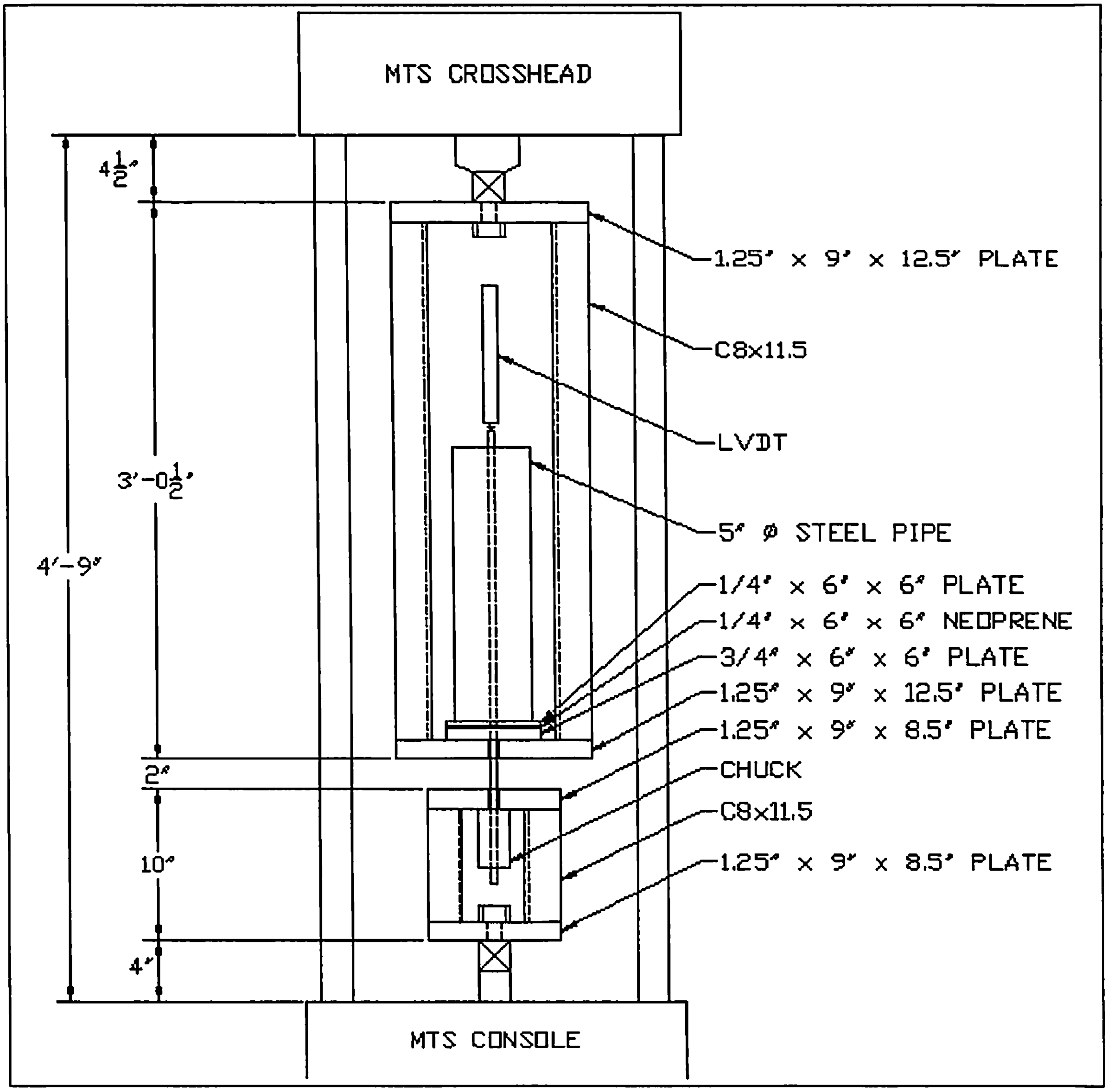


Figure 3.1. "Stiff" NASP Test Set-up.

The concrete mortar mixture consisted of sand, cement, and water mixed thoroughly with a 2:1 aggregate to cement ratio and with the water to cement ratios stated above. The sand confirmed to ASTM C 33 requirements for fine aggregate. The saturated surface dry (SSD) unit weight of the aggregate was used to compute the batch weight. The moisture content of the aggregate was measured and the batch weights were adjusted accordingly. The materials were handled in conformance with ASTM C 192.

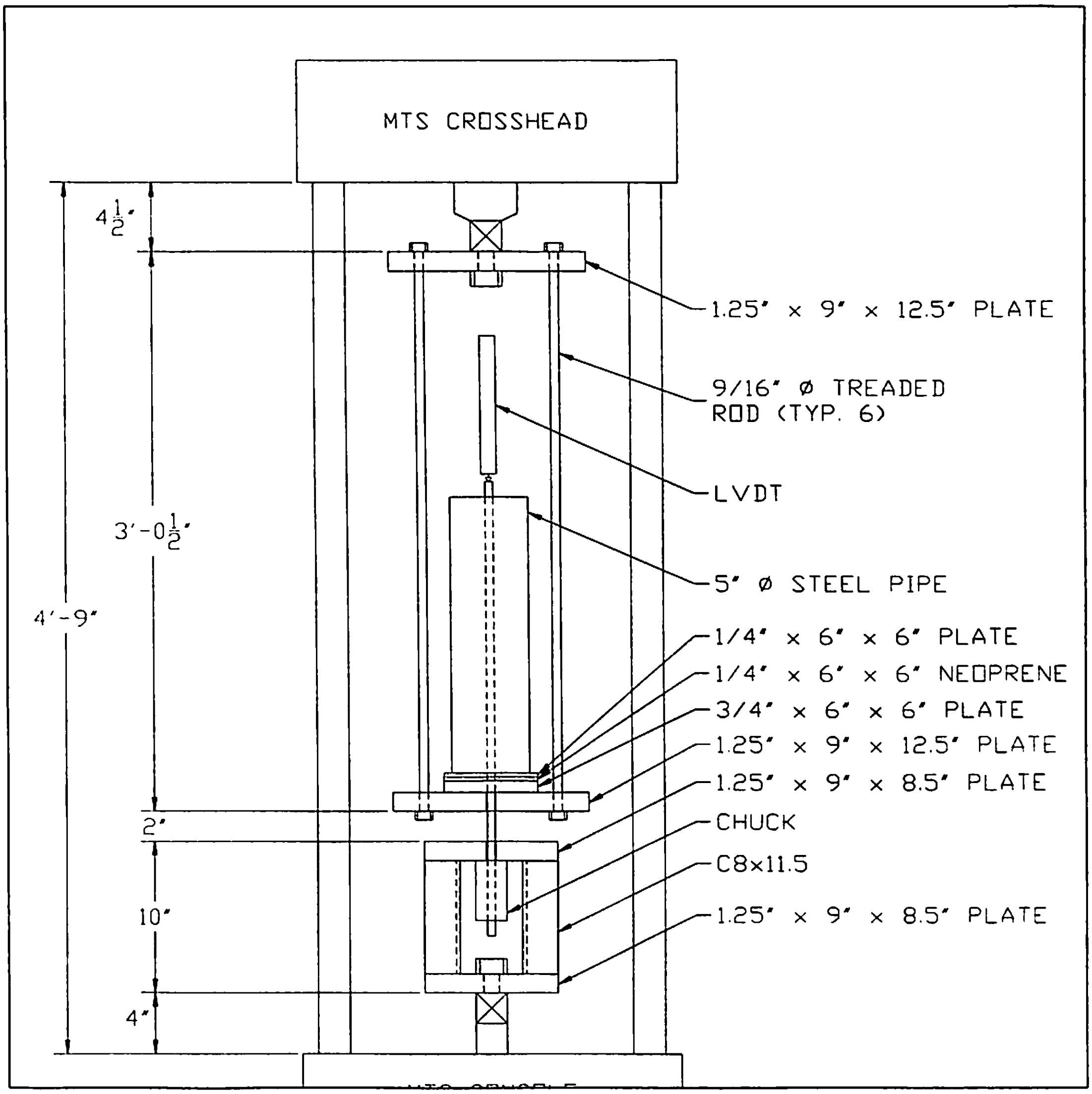


Figure 3.2. "Flexible" NASP Test Set-up.

The cement used conformed to ASTM C 150 requirements for Type III cement. The water was potable and suitable for making concrete.

The cement mortar was mixed in a pan mixer. The water, cement and approximately 1/4 of the fine aggregate were placed in the pan. Then, the mixer was started and the remainder of the fine aggregate was added. The mortar was mixed for three minutes, set without mixing for three minutes, and then a mixed for another two minutes. The test specimens were made in conformance with ASTM C 192. The specimens were made with a single strand cast concentrically in the concrete mortar within a 5 in. diameter by 18 in. long steel casing as shown in Figures 3.1 and 3.2. The specimen mold was made with 5 in. outer diameter, 1/8 in. thick, 18 in. long steel pipe welded to a 1/4 x 6 x 6 in. steel plate with a centered 9/16 in. hole. The bonded length of the strand was 16 in., with a 2 in. long bond breaker. The bond breaker was made with 1-3/4" long piece of styrofoam and tape. Attaching the styrofoam to the strand with tape resulted in a 2" long bond break. The specimens were cast with the pipe vertical. The grout was mechanically consolidated by vibration in conformance with ASTM C 192. Nine 2 in. cubes were made according to ASTM C 305 in order to provide four compression tests before pull-out tests, four compression tests after pull-out tests, and one extra cube in case of any problems with the other tests. During batching, the flow, unit weight, concrete temperature, air temperature, and air relative humidity were recorded.

The concrete mortar test specimens and 2 in. cubes were cured in conformance with ASTM C 192. The concrete mortar was cured at 73 ± 3°F from the time of molding until the time of test. The moisture surrounding the free surfaces of the specimens was held at approximately 100% by placing plastic bags around the top of the specimens and securing with rubber bands. The moisture surrounding the free surfaces of the cubes was held at approximately 100% by securing a glass plate to the top with rubber bands. During the curing period, the specimens were in a vibration-free environment.

The mortar strength was evaluated in conformance with ASTM C 109 using 2 in. cubes, except that the mixture proportions for mortar given in Section 4.1 was not used.

3.3.3.3 Test Procedure

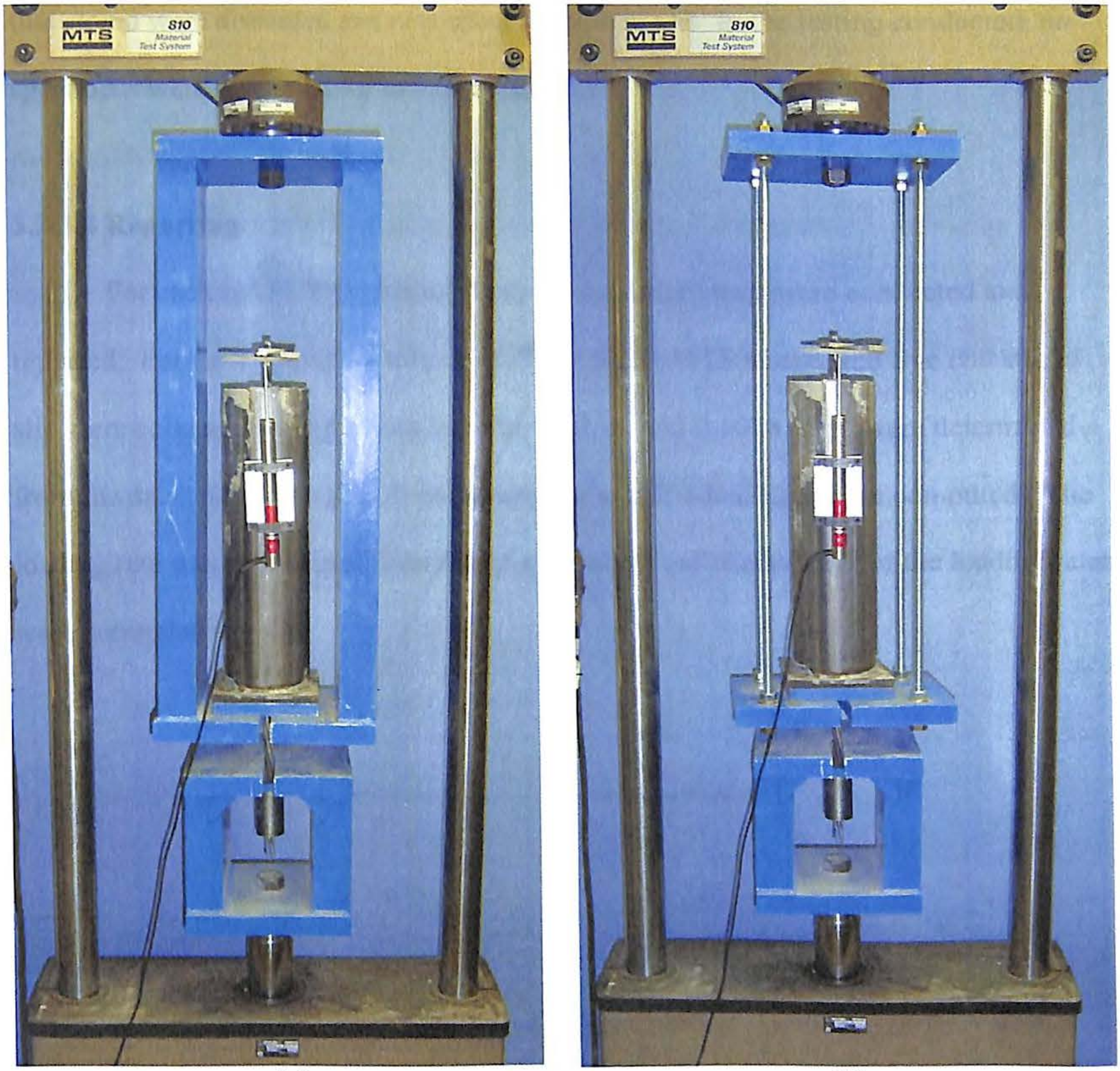
The NASP Bond Test was conducted 24 ± 2 hours from the time of casting the specimens. The compressive strength of the mortar was tested at the beginning of bond testing and at the end of bond testing.

The test was conducted with the test set-ups shown in Figures 3.1, 3.2, and 3.3. The testing was conducted with a "stiff" and "flexible" test frame. The specimen molds were placed on a 1/4 x 6 x 6 in. neoprene pad with a 9/16 in. hole. The neoprene rested on a 3/4 x 6 x 6 in. steel plate with a 9/16 in. hole. The plate rested on the upper loading frame. The upper loading frame consisted of two 1.25 in. thick plates and either channels or rods as depicted in the figures. The upper plate had a 17/16 in. hole to attach the loading frame to the MTS console, and the lower plate had a 9/16 in. slot to place the specimens in the frame. The lower loading frame consisted of two 1.25 in. thick plates and two channels as depicted in the figures. The lower plate had a 17/16 in. hole to attach the loading frame to the MTS actuator, and the upper plate had a 9/16 in. slot to place the specimens in the frame.

The pull-out forces were measured through the load cell of the MTS controller. The relative movement of the strand was measured on the free end through an LVDT and on the fixed in by the MTS actuator.

The MTS actuator pulled the strand at a rate of 0.10 in. per minute. The strand was loaded approximately 6 in. from the end of the specimen.

The pull-out force, MTS stroke, and free end (top of the strand) strand end slip were collected in an electronic data acquisition system. The data was recorded two times per second. The data was then analyzed to determine the pull-out force at 0.01 in. and



(a) (b)

Figure 3.3. Photos of (a) "Stiff" and (b) "Flexible" NASP Testing System.

0.10 in. of free end strand slip. The loading rate was also determined from the data recorded.

Based on the test procedure, if the mortar exhibited cracking in one of the six individual tests, then that test was discarded, but the remainder of the tests were valid. If the mortar exhibited cracking in two or more of the six individual tests, then the tests for that strand were discarded and new specimens prepared. In the testing conducted, no specimens were discarded for radial cracking.

3.3.3.4 Reporting

For each NASP Strand Bond Test, six individual tests were conducted and reported. For each individual test, the pull-out force, MTS stroke, and free end strand slip were collected. The pull-out force for 0.01 in. and 0.10 in. slips were determined from the data. The average pull-out forces of the individual tests were computed. The loading rate was determined from the data recorded, and the average of the loading rates were computed.

Chapter 4

TEST RESULTS

4.1 INTRODUCTION

The results from the testing program will be separated into the following categories:

- Concrete batching.
- Mortar batching.
- NASP Testing

4.2 CONCRETE BATCHING

The concrete was batched in order to obtain fresh properties and 1, 3, 7, 28, and 56 day strength results. The compressive tests were conducted on four cylinders at 1, 3, 7, 28, and 56 days. The tensile splitting tests were conducted on two cylinders at 1 and 28 days.

4.2.1 Fresh Properties

The fresh properties of the concrete mixtures were obtained while batching. The fresh properties of concrete including temperature, slump, unit weight and air content were recorded along with the air temperature and humidity.

4.2.1.1 Glenium 3200 HES

The first batches were made using Glenium 3200 HES as the high range water reducer. The mixtures utilized approximately 800 or 650 pcy of cement. Along with the fresh concrete properties, the air temperature and humidity are given in Tables 4.1 and 4.2. Appendix D contains all mixture proportions, fresh properties, and hardened properties.

4.2.1.2 Rheobuild 1000 and Glenium 3030 NS

Since a change in high range water reducer was desired, a Rheobuild 1000 and a Glenium 3030 NS mixture with 0.36 water to cement ratio was batched next. After choosing the Glenium 3030 NS, two series of batching were conducted. First, while keeping the water to cementious ratio at 0.32, the quantity of silica fume and fly ash replacement was varied. Then, with no silica fume or fly ash replacement, the water to cement ratio was varied. Along with the fresh concrete properties, the air temperature and humidity are given in Tables 4.3, 4.4, and 4.5.

Table 4.1. Glenium 3200 HES Concrete Mixtures with 800 PCY Cement Properties.

Property			8-28-1	8-28-2	8-28-3	8-28-4	8-28-5	8-28-6
Fresh	Air Temperature (°F)		-	73	70	79	76	84
	Relative Air Humidity (%)		-	88.5	92	79	75	70.5
	Concrete Temperature (°F)		80	84	82	86	87	90
	Slump (in.)		11.75	10	9.5	1.75	1.75	9.5
	Unit Weight (pcy)		155	155	155	155	154	154
	Air Content (%)		1.2	3.5	1.8	3.0	3.0	1.9
Hardened	Compressive Strength in psi (C.O.V.)	1 Day	9080 (0.5%)	10,180 (0.8%)	10,090 (1.2%)	9430 (5.1%)	9250 (0.6%)	9540 (1.1%)
		3 Day	11,110 (1.2%)	10,530 (0.4%)	11,090 (1.9%)	11,010 (1.2%)	10,070 (1.1%)	11,200 (1.0%)
		7 Day	11,400 (0.2%)	12,220 (2.5%)	11,190 (3.1%)	11,070 (0.8%)	10,620 (2.3%)	11,760 (0.8%)
		28 Day	13,330 (4.1%)	13,930 (2.5%)	13,150 (4.1%)	11,930 (3.6%)	11,710 (2.3%)	13,800 (0.3%)
		56 Day	14,200 (4.9%)	14,000 (5.5%)	12,830 (3.8%)	12,300 (-)	12,150 (5.0%)	14,810 (6.4%)
	Tensile Strength in psi	1 Day	-	805	630	605	700	735
		28 Day	691	773	483	-	605	941

Table 4.2. Glenium 3200 HES Concrete Mixtures with 650 PCY Cement Properties.

Property			6.5-40-1	6.5-40-2	6.5-40-3	6.5-36-1
Fresh	Air Temperature (°F)		70	71	78	82
	Relative Air Humidity (%)		85	92	79	82
	Concrete Temperature (°F)		80	78	-	87.5
	Slump (in.)		10.25	3.5	8.75	0.75
	Unit Weight (pcy)		149	149	147	149
	Air Content (%)		3.0	3.2	3.9	3.6
Hardened	Compressive Strength in psi (C.O.V.)	1 Day	5780 (1.8%)	4960 (1.9%)	5130 (2.2%)	5930 (3.4%)
		3 Day	7050 (3.7%)	5830 (1.0%)	5860 (1.6%)	5430 (25.1%)
		7 Day	7440 (1.2%)	6360 (2.1%)	6580 (1.4%)	5300 (18.5%)
		28 Day	8430 (4.2%)	7570 (1.4%)	7510 (2.4%)	6240 (12.9%)
		56 Day	9220 (2.3%)	7990 (0.6%)	8460 (3.4%)	5600 (29.8%)
	Tensile Strength in psi	1 Day	485	343	490	205
		28 Day	640	510	490	615

Table 4.3. Rheobuild 1000 Versus Glenium 3030 NS Properties.

Property			R-8-36-1	G3030-8-36-1
Fresh	Air Temperature (°F)		85	81
	Relative Air Humidity (%)		67	67.5
	Concrete Temperature (°F)		91	89
	Slump (in.)		8.25	9
	Unit Weight (pcy)		147	147
	Air Content (%)		4.0	3.5
Hardened	Compressive Strength in psi (C.O.V.)	1 Day	5180 (1.5%)	6220 (0.6%)
		3 Day	6470 (1.2%)	7510 (1.5%)
		7 Day	7400 (2.0%)	8320 (3.9%)
		28 Day	8780 (2.6%)	9280 (2.1%)
		56 Day	9120 (1.6%)	10,110 (-)
	Tensile Strength in psi	1 Day	400	390
		28 Day	570	740

Table 4.4. Glenium 3030 NS Concrete Mixtures with 0.32 W/Cm Properties.

Property			G3030-8-32-						
			1	2	3	4	5	6	7
Fresh	Air Temperature (°F)		79	-	77	70	72	73	75
	Relative Air Humidity (%)		48	-	58	-	78	66	60
	Concrete Temperature (°F)		92	94	90	85	84	83	86
	Slump (in.)		7.5	4.5	3.5	5	9.75	5.25	3.5
	Unit Weight (pcy)		150	150	150	150	152	151	151
	Air Content (%)		-	3.6	3.3	3.2	2.3	2.3	3.3
Hardened	Compressive Strength in psi (C.O.V.)	1 Day	8040 (1.3%)	8400 (0.8%)	8010 (0.4%)	7890 (1.4%)	6950 (3.2%)	7810 (3.3%)	7780 (0.7%)
		3 Day	9430 (2.8%)	9750 (1.2%)	9750 (7.5%)	9550 (1.2%)	8940 (5.0%)	9260 (2.3%)	9860 (1.6%)
		7 Day	- (-)	11,000 (1.8%)	10,600 (2.1%)	11,180 (1.7%)	10,060 (1.5%)	10,130 (2.2%)	11,180 (1.7%)
		28 Day	11,230 (3.7%)	12,810 (3.0%)	13,170 (2.0%)	13,620 (3.6%)	11,930 (1.8%)	12,630 (2.3%)	13,890 (0.6%)
		56 Day	12,500 (0.7%)	14,040 (3.2%)	14,310 (0.6%)	14,940 (1.5%)	12,770 (5.4%)	13,820 (1.1%)	14,740 (3.2%)
	Tensile Strength in psi	1 Day	550	515	625	-	-	570	465
		28 Day	700	730	610	660	595	700	655

Table 4.5. Glenium 3030 NS Concrete Mixtures with Varying W/C Properties.

Property			G3030-8-30-1	G3030-8-28-1
Fresh	Air Temperature (°F)		75	82
	Relative Air Humidity (%)		62	51
	Concrete Temperature (°F)		88	95
	Slump (in.)		7.5	3.5
	Unit Weight (pcy)		151	152
	Air Content (%)		2.6	3.5
Hardened	Compressive Strength in psi (C.O.V.)	1 Day	8580 (0.8%)	9430 (0.7%)
		3 Day	10,040 (1.5%)	10,860 (2.2%)
		7 Day	11,320 (0.8%)	11,900 (2.2%)
		28 Day	12,880 (2.0%)	13,170 (2.6%)
		56 Day	13,840 (1.6%)	13,950 (2.8%)
	Tensile Strength in psi	1 Day	560	580
		28 Day	800	660

4.2.2 Hardened Properties

The hardened properties of the concrete mixtures were obtained after batching. The compressive tests were conducted on four cylinders at 1, 3, 7, 28, and 56 days. The tensile splitting tests were conducted on two cylinders at 1 and 28 days.

4.2.2.1 Glenium 3200 HES

The first batches were made using Glenium 3200 HES as the high range water reducer. The mixtures utilized 800 pcy of cement and 650 pcy of cement. It should be noted that for 6.5-36-1, the samples were extremely honey-combed due to setting prior to completion of making test cylinders, so the best cylinders were used for the one day testing. During batching, the mixtures using Glenium 3200HES began setting prior to the completion of making all specimens. Also, the temperatures of the mixtures increased rapidly. The compressive and tensile test results are given in Table 4.1 and 4.2.

4.2.2.2 Rheobuild 1000 and Glenium 3030 NS

Since a change in high range water reducer was desired, a Rheobuild 1000 and a Glenium 3030 NS mixture with 0.36 water to cement ratio was batched next. After choosing the Glenium 3030 NS, two series of batching was conducted. First, while keeping the water to cementious ratio at 0.32, the quantity of silica fume and fly ash replacement was varied. Then, with no silica fume or fly ash replacement, the water to cement ratio was varied. The hardened concrete properties are given in Tables 4.3, 4.4, and 4.5.

4.3 MORTAR BATCHING

The mortar was batched in order to obtain fresh properties and 1, 7, and 28 day strength results. The compressive tests were conducted on four cubes at 1, 7, and 28 days.

4.3.1 Fresh Properties

The fresh properties of the mortar mixtures were obtained while batching. The flow, unit weight, and air content were recorded. All mixture proportions, fresh properties and hardened properties are given in Appendix D.

4.3.1.1 Original Aggregate

The first batches were made with the original fine aggregate. The mixtures were meant to be batched in 0.45 water to cement ratio and 2:1 fine aggregate to cement ratio; however, due to calculation errors in accounting for the moisture content, not all batches were exactly 0.45 water to cement ratios. However, the water to cement ratios rounded to two significant figures were still 0.45. The fresh mortar properties are given in Table 4.6.

4.3.1.2 New Aggregate

The next batches were made using the new fine aggregate. The water to cement ratio and quantity of fine aggregate per cubic yard were varied. The fresh mortar properties are given in Table 4.7.

Table 4.6. Original Aggregate Mortar Properties.

Property			NASP-					
			A	B	C	D	E	F
W/C			0.45	0.45	0.45	0.45	0.45	0.45
Fresh	Flow (%)		106.3	116.5	132.5	121.5	91.5	80.0
	Unit Weight (pcy)		138	139	138	138	139	139
	Air Content (%)		1.4	0.6	1.6	1.1	0.6	0.7
Hardened	Compressive Strength in psi (C.O.V.)	1 Day	4040 (7.3%)	3190 (21.2%)	4720 (3.4%)	4905 (3.8%)	4850 (3.9%)	- (-)
		7 Day	6795 (3.4%)	6795 (3.2%)	7165 (5.2%)	7160 (4.0%)	7300 (2.4%)	7160 (8.0%)
		28 Day	7755 (3.7%)	7500 (2.4%)	7040 (6.0%)	7980 (4.9%)	7820 (11.1%)	8215 (1.5%)

Table 4.7. New Aggregate Mortar Properties.

Property			NASP-				
			AA	BB	CC	DD	EE
W/C			0.45	0.48	0.50	0.50	0.40
Fresh	Flow (%)		110.0	123.8	131.0	127.0	91.3
	Unit Weight (pcy)		136	136	136	137	-
	Air Content (%)		2.5	2.9	2.7	2.3	-
Hardened	Compressive Strength in psi (C.O.V.)	1 Day	4775 (3.1%)	4395 (1.6%)	4405 (2.9%)	4195 (2.1%)	5720 (4.0%)
		7 Day	7295 (4.0%)	7305 (1.6%)	6860 (2.3%)	6770 (4.9%)	8085 (4.7%)
		28 Day	8100 (3.8%)	7670 (1.6%)	7565 (2.3%)	7695 (4.4%)	9245 (3.0%)

4.3.2 Hardened Properties

The hardened properties of the mortar mixtures were obtained after batching. The compressive tests were conducted on four cubes at 1, 7, and 28 days. The hardened mortar compressive strengths for the original and new aggregates are given in Tables 4.6 and 4.7, respectfully.

4.4 NASP TESTING

After initial batching, NASP bond tests were conducted on two strand samples with three water to cement ratios for each strand and two testing frames for each strand and water to cement ratio combination.

4.4.1 Batching Conditions

During batching, the flow, unit weight, concrete temperature, air temperature, and air relative humidity were recorded. The values are reported in Table 4.8.

4.4.2 Test Results

The results for NASP testing include mortar strengths, load rates, force at 0.01 in. free end strand slip, and force at 0.10 in. free end strand slip. The air temperature was also recorded during curing. The mortar strength testing was conducted during the first two NASP specimen load tests and during the last two NASP specimen load tests. The load rates and forces at free end strand slips were calculated from the data collected. The load rate was the load rate over the flat portion of the load versus load rate curve. The flat portion of the curve was also the maximum loading rate of the specimen. Figures 4.1 and 4.2 show examples of the slip versus load curves, and Figure 4.3 shows an example of the load versus load rate curve. Appendix D contains graphs of slip versus load and load versus load rate, and Appendix E contains graphs from average results. Tables 4.9, 4.10, 4.11, 4.12, 4.13 and 4.14 give the results. Values were discarded when they were outliers. Outliers were the values that were more than 1.5 times the standard deviation from the upper and lower quartile. The compressive strength value used in the graphs

Table 4.8. NASP Mortar Fresh Properties.

STRAND	"AA"			"FF"		
Water to Cement Ratio	0.45	0.475	0.50	0.40	0.45	0.50
Flow (%)	98	121	121	80	117.5	125
Unit Weight (pcy)	138.4	140.1	137.8	140.0	139.5	137.7
Air Temp. (°F)	72	72	70	69	72	59
Air Rel. Humidity (%)	61	72	53	30	35	36
Concrete Temp. (°F)	82	82	72	74	78	68

is the value midway through the testing of the individual frame ("stiff" or "flexible") calculated by assuming a linear increase from the beginning of the four hour testing window to the end.

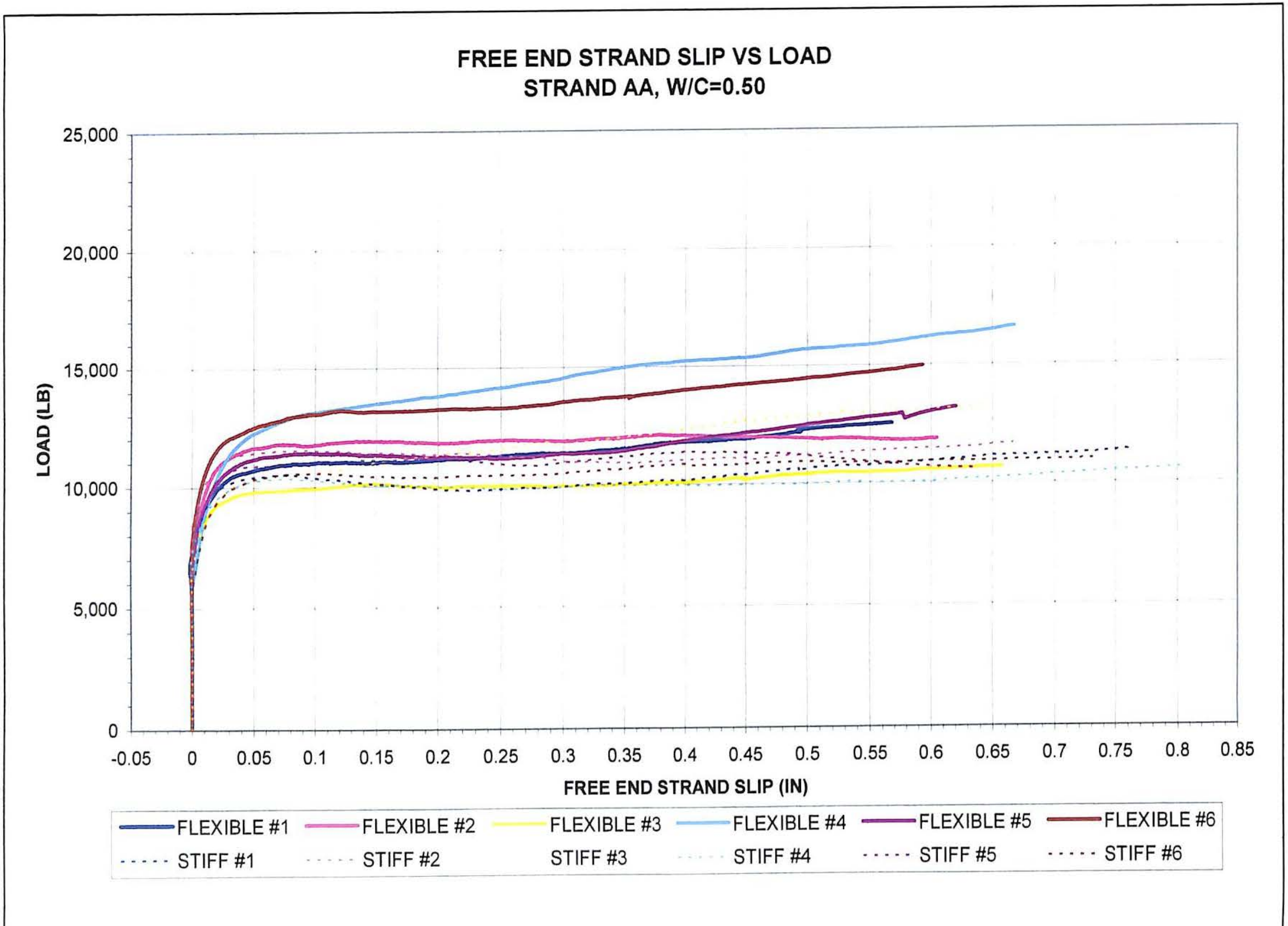


Figure 4.1. Slip Vs. Load Curves for Strand "AA" with W/C = 0.50.

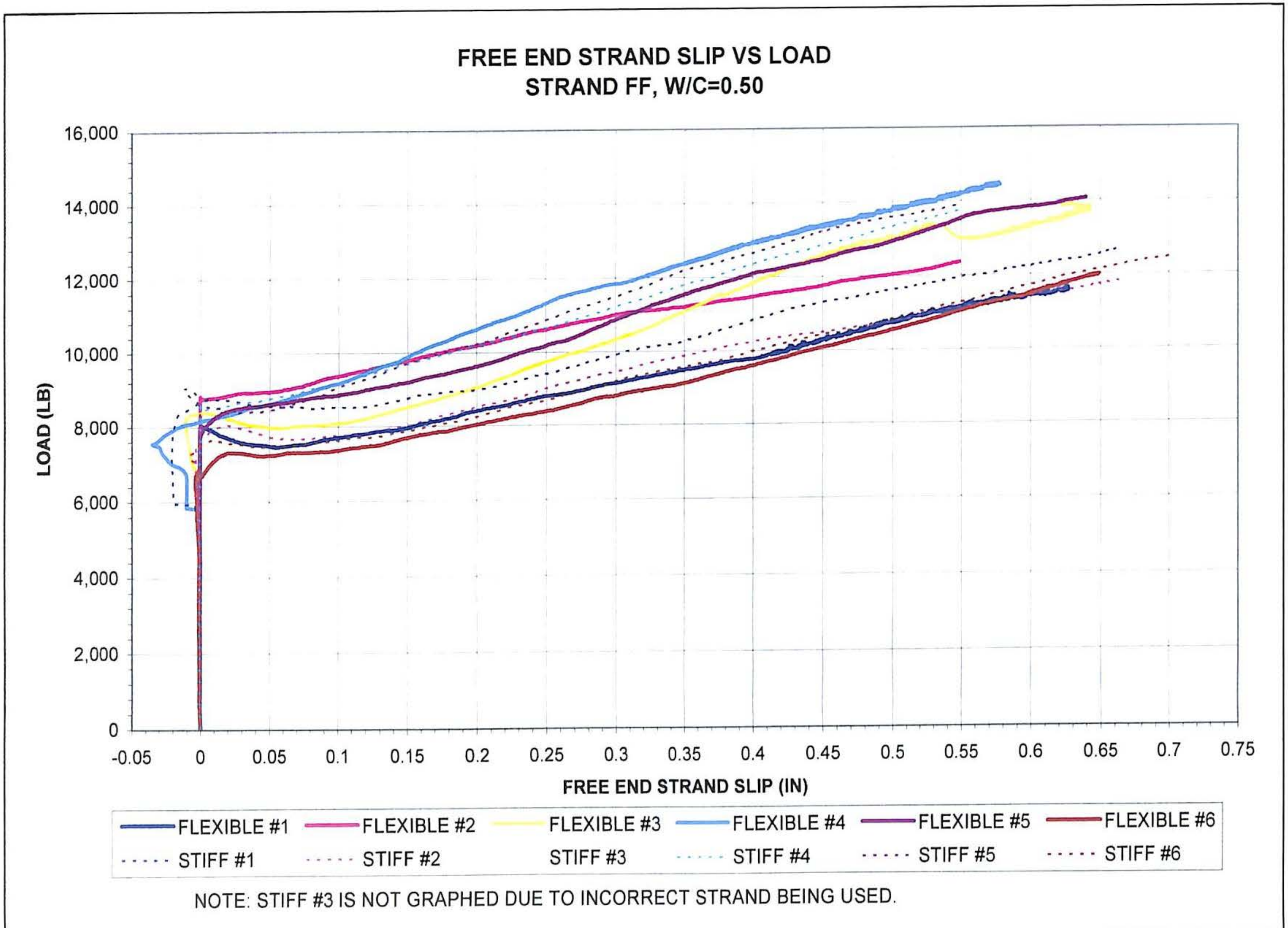


Figure 4.2. Slip Vs. Load Curves for Strand "FF" with W/C = 0.50.

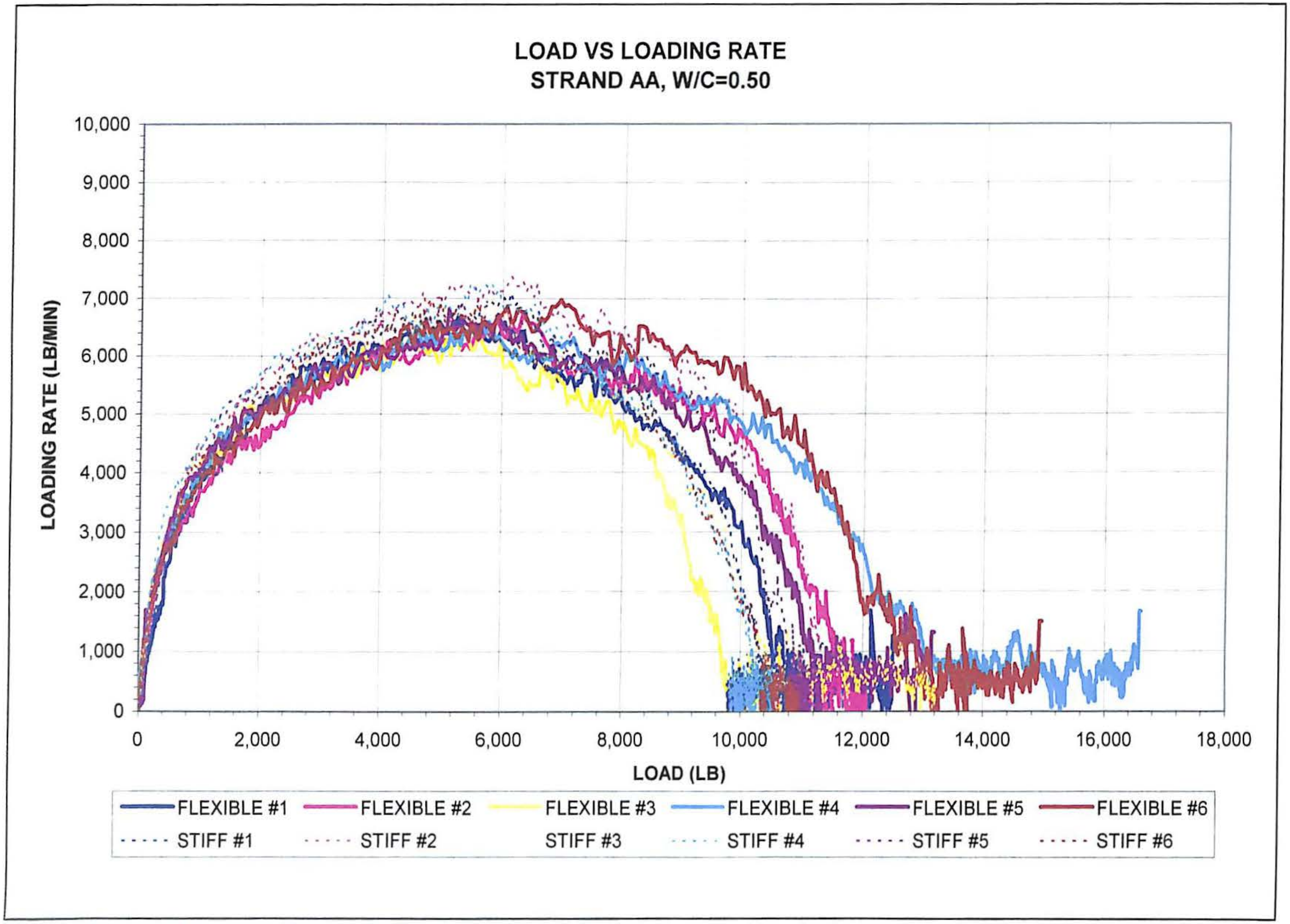


Figure 4.3. Load Vs. Loading Rate Curves for Strand "AA" with W/C = 0.50.

Table 4.9. NASP Test Results for Strand AA with W/C=0.45

TEST SYSTEM	COMP. STRENGTH (PSI)			STRAND	LOADING RATE* (LB/MIN)	LOAD (LB) STRAND SLIP	
	SAMPLE	BEFORE	AFTER			0.01 IN	0.10 IN
FLEXIBLE	1	4919	5674	1	7,250	18,450	21,170
	2	4719	6095	2	6,970	12,290*	19,850
	3	5081	5943	3	7,250	15,580	MAXED
	4	5478	5966	4	7,190	17,280	21,360
				5	7,150	16,390	23,410
				6	7,030	16,960	23,420
	AVG	**5050**	**5920**	**AVG**	**7,140**	**16,930**	**21,840**
	ST. DEV.	**322**	**177**	**ST. DEV**	**116**	**1066**	**1549**
	C.O.V.	**6.38%**	**2.99%**	**C.O.V.**	**1.63%**	**6.30%**	**7.09%**
STIFF	1	4919	5674	7	7,930	15,290	21,290
	2	4719	6095	8	8,190	16,070	21,970
	3	5081	5943	9	7,940	15,770	20,910
	4	5478	5966	10	7,640	15,360	20,330
				11	7,820	14,380	21,880
				12	7,890	15,860	21,440
	AVG	**5050**	**5920**	**AVG**	**7,900**	**15,460**	**21,300**
	ST. DEV.	**322**	**177**	**ST. DEV**	**179**	**606**	**616**
	C.O.V.	**6.38%**	**2.99%**	**C.O.V.**	**2.27%**	**3.92%**	**2.89%**

* Data Point was discarded in average, standard deviation, and C.O.V. because it was an outlier.

Table 4.10. NASP Test Results for Strand AA with W/C=0.475

TEST SYSTEM	COMP. STRENGTH (PSI)			STRAND	LOADING RATE* (LB/MIN)	LOAD (LB) STRAND SLIP	
	SAMPLE	BEFORE	AFTER			0.01 IN	0.10 IN
FLEXIBLE	1	4571	5356	7	7,300	13,330	20,300
	2	4620	4998	8	7,110	13,470	19,670
	3	4644	5309	9	6,970	11,940	15,780*
	4	4599	4963	10	7,480	15,750	21,680
				11	7,240	14,620	19,440
				12	7,370	16,340	21,910
	AVG	**4610**	**5155**	**AVG**	**7,250**	**14,240**	**20,600**
	ST. DEV.	**31**	**205**	**ST. DEV**	**183**	**1,646**	**1,138**
	C.O.V.	**0.68%**	**3.97%**	**C.O.V.**	**2.53%**	**11.56%**	**5.53%**
STIFF	1	4571	5356	1	7,590	12,960	17,900
	2	4620	4998	2	7,600	12,990	17,900
	3	4644	5309	3	8,220	13,630	18,750
	4	4599	4963	4	8,050	15,550	19,570
				5	7,720	11,820	18,820
				6	7,520	11,980	18,290
	AVG	**4610**	**5155**	**AVG**	**7,780**	**13,160**	**18,540**
	ST. DEV.	**31**	**205**	**ST. DEV**	**285**	**1,356**	**643**
	C.O.V.	**0.68%**	**3.97%**	**C.O.V.**	**3.66%**	**10.30%**	**3.47%**

* Data Point was discarded in average, standard deviation, and C.O.V. because it was an outlier.

Table 4.11. NASP Test Results for Strand AA with W/C=0.50.

TEST SYSTEM	COMP. STRENGTH (PSI)			STRAND	LOADING RATE* (LB/MIN)	LOAD (LB) STRAND SLIP	
	SAMPLE	BEFORE	AFTER			0.01 IN	0.10 IN
FLEXIBLE	1	3688	3480	7	6,310	9,020	10,280
	2	3266	3497	8	6,190	9,650	11,750
	3	3733	3794	9	6,130	8,490	9,910
	4	3676	4107	10	6,190	8,750	13,120
				11	6,300	9,140	11,400
				12	6,350	10,470	13,040
	AVG	**3590**	**3720**	**AVG**	**6,250**	**9,250**	**11,580**
	ST. DEV.	**218**	**296**	**ST. DEV**	**87**	**713**	**1,345**
	C.O.V.	**6.07%**	**7.95%**	**C.O.V.**	**1.39%**	**7.70%**	**11.61%**
STIFF	1	3688	3480	1	6,550	9,000	10,370
	2	3266	3497	2	6,930	10,070	11,520
	3	3733	3794	3	6,680	8,500	10,620
	4	3676	4107	4	6,850	8,780	10,250
				5	6,650	9,220	11,020
				6	6,660	8,190	10,550
	AVG	**3590**	**3720**	**AVG**	**6,720**	**8,960**	**10,720**
	ST. DEV.	**218**	**296**	**ST. DEV**	**141**	**654**	**472**
	C.O.V.	**6.07%**	**7.95%**	**C.O.V.**	**2.10%**	**7.30%**	**4.40%**

Table 4.12. NASP Test Results for Strand FF with W/C=0.40.

TEST SYSTEM	COMP. STRENGTH (PSI)			STRAND	LOADING RATE* (LB/MIN)	LOAD (LB) STRAND SLIP	
	SAMPLE	BEFORE	AFTER			0.01 IN	0.10 IN
FLEXIBLE	1	6051	6868	1	6,950	10,790	8,740
	2	6249	6764	2	6,830	9,650	7,790
	3	6390	6506	3	6,570	8,710	8,740
	4	6041	6872	4	6,790	9,620	9,400
				5	7,010	7,920	8,250
				6	7,010	8,720	8,370
	AVG	**6185**	**6750**	**AVG**	**6,860**	**9,240**	**8,550**
	ST. DEV.	**168**	**172**	**ST. DEV**	**169**	**1,001**	**547**
	C.O.V.	**2.72%**	**2.55%**	**C.O.V.**	**2.47%**	**10.84%**	**6.40%**
STIFF	1	6051	6868	7	7,610	10,040	8,570
	2	6249	6764	8	7,450	7,940	8,320
	3	6390	6506	9	7,620	10,690	9,070
	4	6041	6872	10	7,710	11,150	9,040
				11	7,170	7,110	7,990
				12	7,630	8,240	7,810
	AVG	**6185**	**6750**	**AVG**	**7,530**	**9,200**	**8,470**
	ST. DEV.	**168**	**172**	**ST. DEV**	**196**	**1,650**	**526**
	C.O.V.	**2.72%**	**2.55%**	**C.O.V.**	**2.61%**	**17.94%**	**6.21%**

Table 4.13. NASP Test Results for Strand FF with W/C=0.45.

TEST SYSTEM	COMP. STRENGTH (PSI)			STRAND	LOADING RATE* (LB/MIN)	LOAD (LB) STRAND SLIP	
	SAMPLE	BEFORE	AFTER			0.01 IN	0.10 IN
FLEXIBLE	1	5126	5471	7	6,630	7,190	7,250
	2	5120	5122	8	6,630	8,180	7,470
	3	5124	5514	9	6,530	7,740	7,230
	4	4663	5120	10	6,740	8,860	9,190*
				11	6,760	8,790	7,050
				12	6,310	7,520	7,730
	AVG	**5010**	**5305**	**AVG**	**6,600**	**8,050**	**7,350**
	ST. DEV.	**230**	**215**	**ST. DEV**	**165**	**684**	**261**
	C.O.V.	**4.60%**	**4.06%**	**C.O.V.**	**2.50%**	**8.50%**	**3.56%**
STIFF	1	5126	5471	1	7,160	7,250	6,680
	2	5120	5122	2	6,980	8,320	7,850
	3	5124	5514	3	7,350	9,300	7,960
	4	4663	5120	4	7,040	11,430	10,100*
				5	7,110	8,940	7,340
				6	7,100	7,580	7,400
	AVG	**5010**	**5305**	**AVG**	**7,120**	**8,800**	**7,450**
	ST. DEV.	**230**	**215**	**ST. DEV**	**127**	**1,504**	**507**
	C.O.V.	**4.60%**	**4.06%**	**C.O.V.**	**1.79%**	**17.08%**	**6.81%**
* Data Point was discarded in average, standard deviation, and C.O.V. because it was an outlier.							

Table 4.14. NASP Test Results for Strand FF with W/C=0.50.

TEST SYSTEM	COMP. STRENGTH (PSI)			STRAND	LOADING RATE* (LB/MIN)	LOAD (LB) STRAND SLIP	
	SAMPLE	BEFORE	AFTER			0.01 IN	0.10 IN
FLEXIBLE	1	4288	4569	1	6,140	7,880	7,700
	2	4266	4179	2	6,090	8,800	9,370
	3	3765	4253	3	6,310	8,350	8,090
	4	3853	4473	4	5,690	8,240	9,170
				5	6,260	8,240	8,860
				6	5,690	7,090	7,360
	AVG	**4045**	**4370**	**AVG**	**6,030**	**8,100**	**8,430**
	ST. DEV.	**273**	**183**	**ST. DEV**	**275**	**576**	**826**
	C.O.V.	**6.75%**	**4.19%**	**C.O.V.**	**4.56%**	**7.11%**	**9.80%**
STIFF	1	4288	4569	7	6,500	8,760	8,530
	2	4266	4179	8	6,670	7,970	7,760
	3	3765	4253	9	INCORRECT STRAND		
	4	3853	4473	10	5,940	8,510	9,180
				11	6,830	8,550	9,060
				12	6,300	7,660	7,640
	AVG	**4045**	**4370**	**AVG**	**6,450**	**8,290**	**8,430**
	ST. DEV.	**273**	**183**	**ST. DEV**	**346**	**457**	**715**
	C.O.V.	**6.75%**	**4.19%**	**C.O.V.**	**5.36%**	**5.52%**	**8.47%**

Chapter 5

DISCUSSION OF RESULTS

5.1 INTRODUCTION

The discussion of the results from the testing program will be separated into the following categories:

- Concrete batching.
- Mortar batching.
- NASP testing

5.2 CONCRETE BATCHING

The concrete was batched in order to achieve the goal of reaching three desired compressive strength combinations with a workable mixture that had a slump of 6 to 8 in. One combination was a 6,000 psi one day strength and a 10,000 psi 28 or 56 day strength. The next combination was a 8,000 psi one day strength and a 14,000 psi 28 or 56 day strength, and the last combination was a 10,000 psi one day strength and a 18,000 psi 28 or 56 day strength. The target mixtures will be referred to by their one day strength.

5.2.1 Admixtures

Admixtures had significant effects on the fresh and hardened properties of the concrete. The high range water reducer admixtures used were Glenium 3200 HES, Glenium 3030 NS, and Rheobuild 1000. The other chemical admixtures used were a mid range water reducer, Polyheed 997, and a water reducing set retarder, Pozzolith 100 XR. The mineral admixtures used were silica fume and fly ash.

5.2.1.1 Chemical Admixtures

The chemical admixtures were used to increase the slump of the fresh concrete and to change the rate of setting.

The high range admixtures had significant effects on the fresh and hardened properties of the concrete. Glenium 3200 HES resulted in batches that were difficult to handle since the admixture was causing the concrete to set up before all cylinders were cast. As seen by comparing the data within the Glenium 3200 HES data set, increasing the high range water reducer increased the one day strength until the slump was high enough to allow segregation of aggregates. Due to the early setting using Glenium 3200 HES, two other high range water reducers were analyzed.

The two other water reducers, Glenuim 3030 NS and Rheobuild 1000, were used to determine the best suited admixture for the batching. The two 0.36 water to cement ratio mixtures had similar workability, but the Glenium 3030 NS resulted in 20 percent higher one day compressive strengths than the Rheobuild 1000. The compressive strengths can be found in Table 4.3. Glenium 3030 NS is also an admixture used in the precast plant that will cast the beams utilizing the trial batching. Due to the higher strengths and the use at the precast plant, Glenium 3030 NS was chosen for the remainder of the batches.

The high range admixture affected the strengths of concrete. By comparing the 8-28 mixtures utilizing Glenium 3200HES with the G3030-8-28-1 mixture, the strength difference between the two Glenium admixtures is not significant. The compressive strengths can be found on Table 4.1 and 4.5. The Glenium 3030 NS mixture had compressive strengths that fell within the range recorded for the Glenium 3200HES mixtures. However, by comparing the 0.36 water to cement ratio mixtures, Rheobuild 1000 resulted in lower compressive strength concrete than the Glenium 3030 NS.

Based on the results, Glenium 3030 NS is recommended for the prestressed beams to be made in subsequent research.

5.2.1.2 Mineral Admixtures

The mineral admixtures, silica fume and fly ash, were used to increase compressive strengths of the concrete.

Based on the results from G3030-8-32-1 and G3030-8-32-5 as shown in Figure 5.1, fly ash had a negative effect on the early compressive strength of concrete and a positive effect on the 28 and 56 day compressive strengths of the concrete. At day one, the compressive strength of the concrete was 14 percent lower with 10 percent fly ash replacement than with no cementious replacement. At 28 and 56 days, the compressive strength of the concrete was 6 and 2 percent, respectfully, greater with 10 percent fly ash replacement than with no cementious replacement. The values were given in Chapter 4 and can be seen graphically in Figure 5.1.

Based on the results, the effects of silica fume addition were not observed until 28 day testing. The compressive strength at one day for 10 percent silica fume replacement batches was on average 2 percent lower than the batch without cementious replacement. However, at 28 and 56 days, the compressive strength in the 10 percent silica fume replacement was 21 and 17 percent, respectfully, greater than the batch with no cementious replacement. After the mixtures were complete, a new shipment of silica

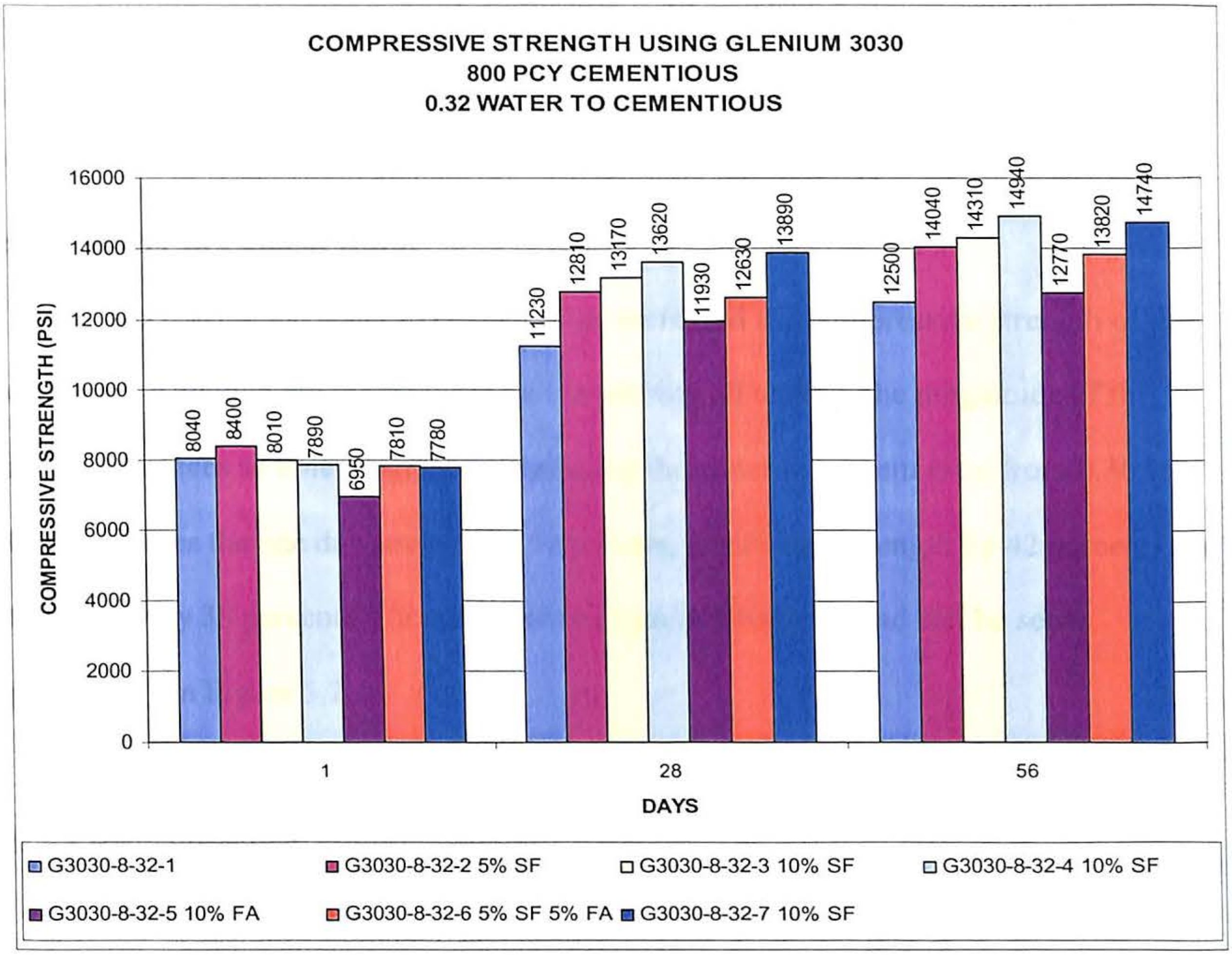


Figure 5.1. Graphical Glenium 3030 NS Results for 0.32 W/Cm.

fume was received. After the mixtures were complete, a new shipment of silica fume was received. The new silica fume, which was the same product as used in the batching, contained noticeably smaller particles. It is believed that the silica fume used may have been activated by moisture prior to its use. So, with the new silica fume, it is possible that the 28 day strength will be greater than 14 ksi. The values were given in Chapter 4 and can be seen graphically in Figure 5.1.

The combination of silica fume and fly ash replacement yielded about the same compressive strength at one day as with no cementious replacement; however, at 28 and 56 days, the compressive strength of the silica fume and fly ash replacement batch was 6 and 10 percent, respectfully, greater than with no cementious replacement. The values were given in Chapter 4 and can be seen graphically in Figure 5.1.

5.2.2 Water to Cement Ratio

Decreasing the water to cement ratio, increased the compressive strength of the concrete. Although the increase can be seen during all testing, the magnitude of the increase reduces as time progresses. Reducing the water to cement ratio from 0.36 to 0.28 increases the one day strength by 52 percent, the 28 day strength by 42 percent, and the 56 day by 38 percent. The values were given in Chapter 4 and can be seen graphically in Figure 5.2.

5.2.3 Recommended Mixture Designs

Although all criteria were not achieved with this batching sequence, some recommendations can be made. The mixture proportions and properties for the mixtures referenced can be found in Appendix D.

The 6,000 psi one day and 10,000 psi 28 or 56 day mixture can be achieved with the mixture design from G3030-8-36-1. The one day compressive strength was 6220 psi and the 56 day compressive strength was 10,110 psi. This mixture can be used without changes to make the 6 ksi mixture.

The 8,000 psi one day and 14,000 psi 28 or 56 day mixture can be achieved with the silica fume replacement mixture G3030-8-32-7. The one day strength was 7780 psi and the 56 day compressive strength was 14,470. This mixture may result in high 28 day strength with the new silica fume. If this is the case, then G3030-32-2 should be used. In

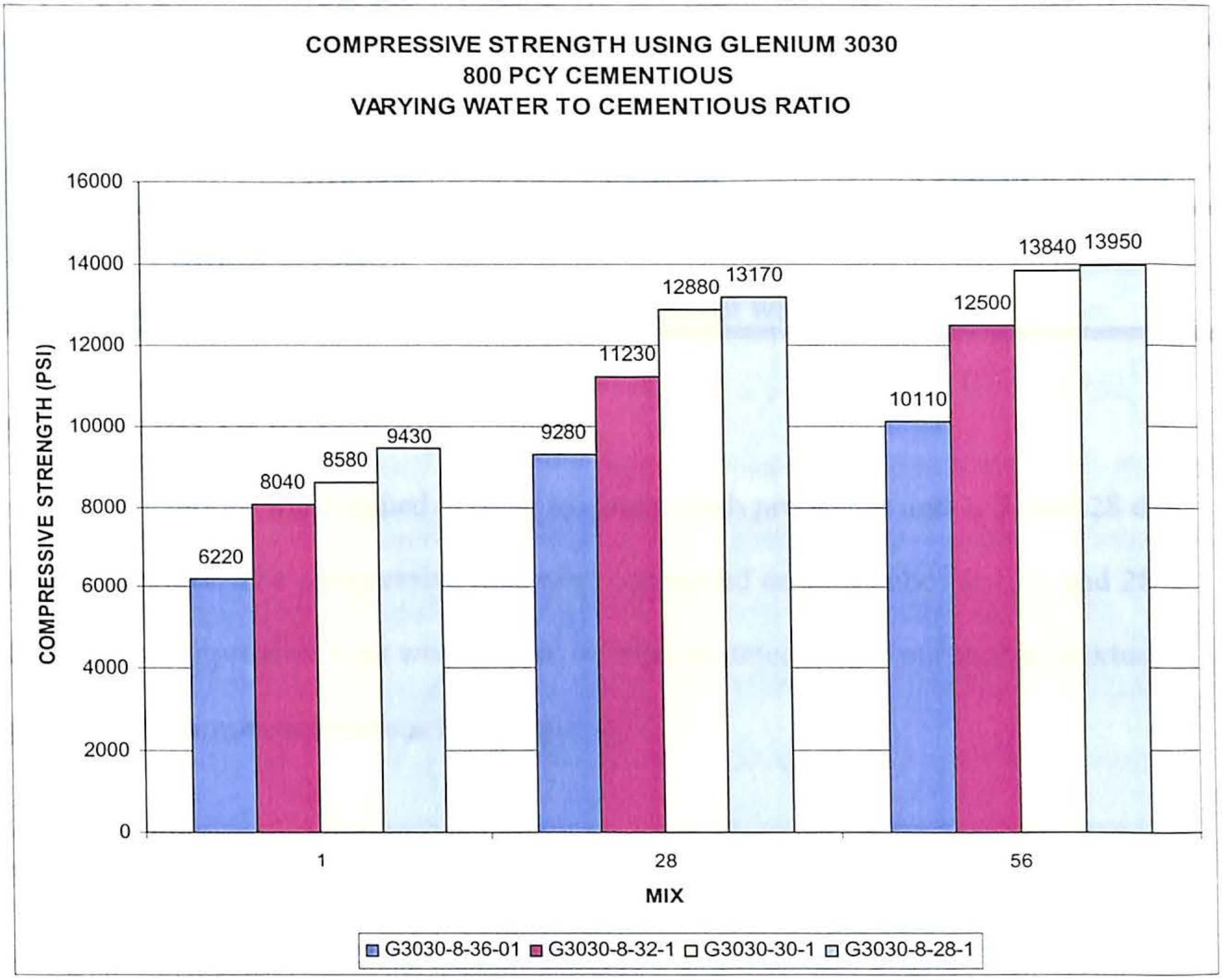


Figure 5.2. Graphical Glenium 3030 NS Results for Varying W/Cm.

both mixtures the high range water reducer needs to be slightly increased. Some additional trial batching may be required to ensure the correct mixture design is used.

The 10,000 psi one day and 18,000 psi 28 or 56 day mixture can likely be achieved with silica fume replacement using G3030-8-28-1. The one day strength was 9430 psi and the 56 day strength was 13,950 psi. Given the increase seen with silica fume, the 56 day compressive strength will be at least 16,000 psi, but more likely over 18,000 psi with new silica fume. The high range water reducer will need to be increased to increase the slump.

Table 5.1 gives the recommended mixture designs.

Table 5.1. Recommended Mixture Designs

	f_{ci}'	f_c'	Mixture
Class 1	6,000	10,000	G3030-8-36-1
Class 2	8,000	14,000	G3030-8-32-7*
Class 3	10,000	18,000	G3030-8-28-1**
Notes: * Increase HRWR quantity ** Increase HRWR quantity and replace 10% cement with silica fume			

5.3 MORTAR BATCHING

The mortar was batched in order to obtain fresh properties and 1, 7, and 28 day strength results. The compressive tests were conducted on four cubes at 1, 7, and 28 days. The compressive tests were needed in order to determine if our mortar mixture gave similar results as previous NASP testing.

5.3.1 Repeatability of Mortar

The first batching with the old aggregate was conducted in order to achieve similar results as batches previously used for NASP Tests and to increase repeatability of results by practice.

The flow of the original aggregate varied significantly, ranging from 80.0 to 132.5. However, the repeatability was better with the new aggregate. The 0.50 water to cement ratio mixtures were only about 3% different. The measured flows of the new aggregate mortars also varied as expected with greater flows for greater water to cement ratios.

The unit weight and air content were relatively consistent. The unit weight only varied by about 2 percent for all the mortar batches. The air content varied, but given the variability in the test, the variations are acceptable.

The compressive strengths of the mortar batches were also relatively consistent. After the first couple of original aggregate batches, the compressive strengths of the original aggregate mortar batches were consistent. The one day strengths for NASP-C through NASP-E varied by 4 percent. The new aggregate mortar mixtures were not repeated multiple times for each water to cement ratio; however, the compressive strength did increase with decreasing water to cement ratios as should be expected. Figure 5.3 shows the water to cement ratios and the compressive strengths of the mortar trial batches and the NASP batching. The regression analysis shows a good correlation of the results with an r^2 value of 0.86. An r^2 value of 1.0 would be a "perfect" correlation. One factor affecting the correlation was the moisture content of the fine aggregate. In the trial batching, small amounts were batched, so the moisture content throughout the aggregate was relatively consistant. However, the NASP batches were approximately twenty times the size of the trial batches, and with the larger quantities used, the moisture content did vary substantially. The variable moisture content in the fine aggregate affects the actual water to cement ratio. Since the correlation is based on water to cement ratio, the moisture content of fine aggregate affects it.

5.3.2 Water to Cement Ratio

The water to cement ratio affected the mortar mixtures as expected. Increased water to cement ratios increased the flow and decreased the compressive strength. These trends are shown in Figures 5.3 and 5.4.

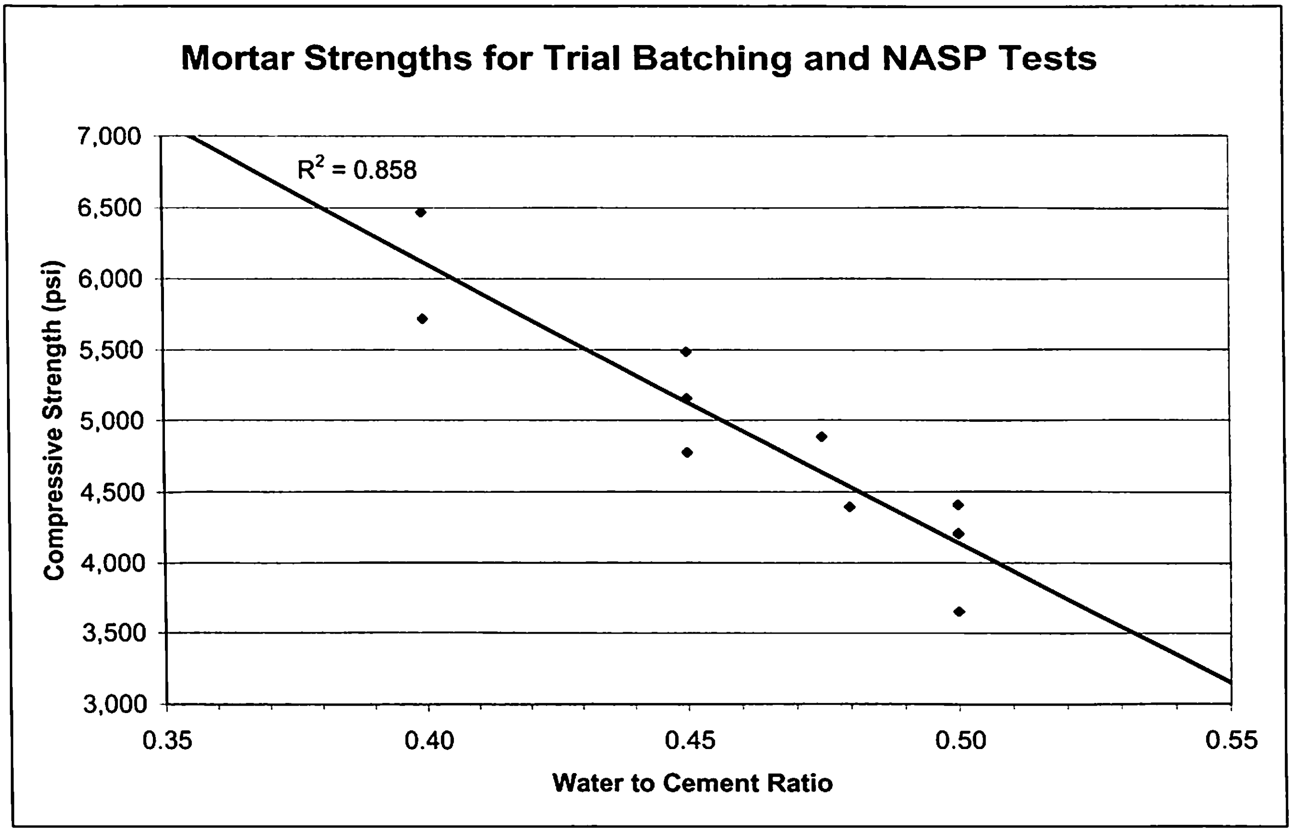


Figure 5.3. Mortar Strengths for Trial Batching and NASP Tests.

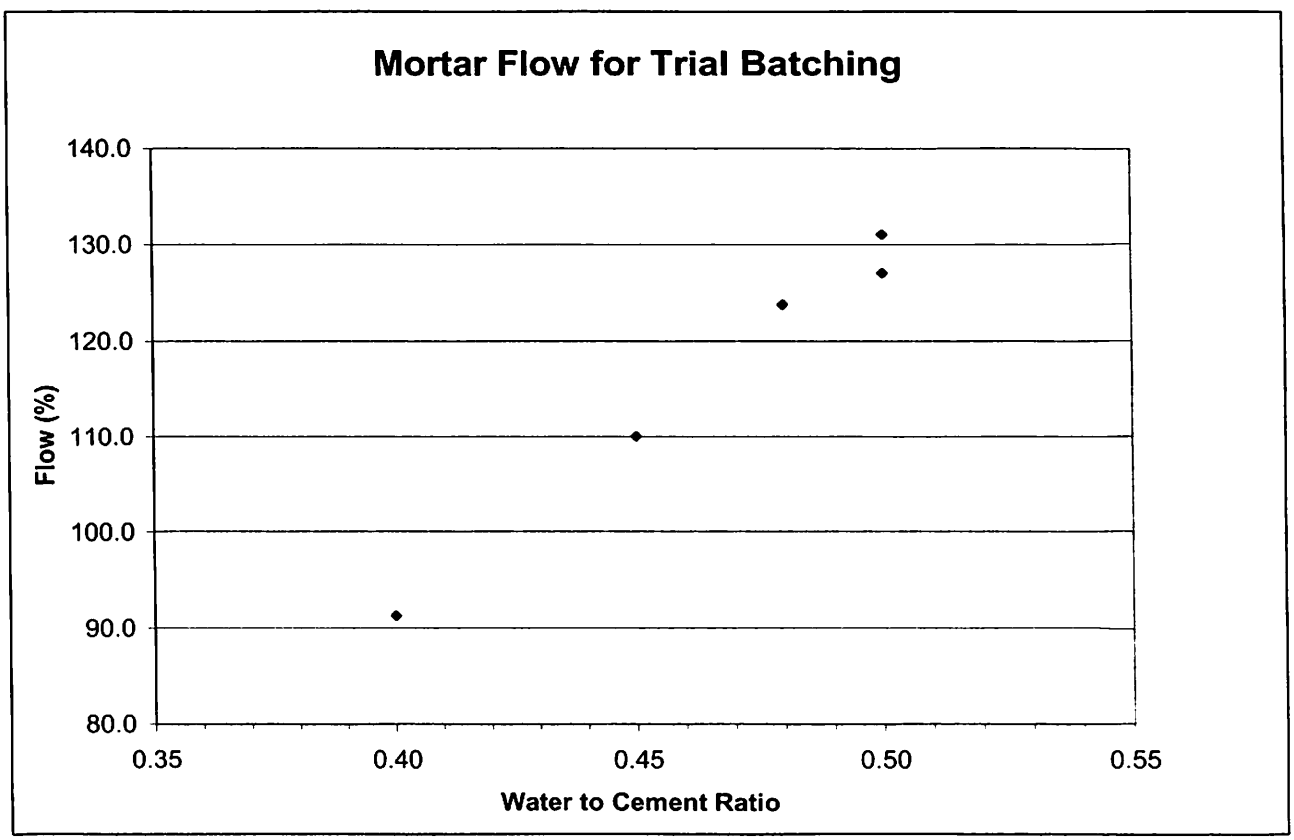


Figure 5.4. Mortar Flow for Trial Batching.

5.3.3 Recommended Mixture Designs

The compressive strength range of 4000 to 6000 psi was achieved with the mortar batching program. The 0.40, 0.45, and 0.50 mixtures with the 2:1 fine aggregate to cement ratios should be used for the NASP testing.

5.4 NASP TESTING

After initial batching, NASP bond tests were conducted on two strand samples with three water to cement ratios for each and two testing frames for each for a total of twelve tests.

5.4.1 Batching Conditions

During batching, the flow, unit weight, concrete temperature, air temperature, and air relative humidity were recorded. Based on the observed consistency, flow and mortar strength the data suggest that the Strand "AA" mixture intended for a 0.50 water to cement ratio had a greater water content than expected. The mortar cube compressive strengths also suggest a water content greater than expected for the mixture since the compressive strengths were much less than the Strand "FF" companion batch and the trial batches. The remainder of the batches had similar properties as the trial and companion batches.

5.4.2 Test Results

The results for NASP testing include mortar strengths, load rates, force at 0.01 in. free end strand slip, and force at 0.10 in. free end strand slip. The air temperature was also recorded during curing. The mortar strength testing was conducted during the first two NASP specimen load tests and during the last two NASP specimen load tests. The load rates and forces at free end strand slips were calculated from the data collected.

For data comparisons, more emphasis will be made for the 0.10 in. free end strand slip than the 0.01 in. free end strand slip. As a result of testing by Russell and Paulsgrove (1999a and 1999b) and Brown (2003), the 0.10 in. free end strand slip was determined to be more reliable than the 0.01 in. free end strand slip due to lower coefficients of variation in the data and greater repeatability.

5.4.2.1 Test Setup

The testing program was designed to determine if the testing frame setup affected the results of the test. Although the "flexible" system with rods exhibited slightly higher NASP bond force values than the "stiff" system with channels for Strand "AA", the difference for 0.10 in. free end strand slip was 8 percent or less. For Strand "FF", the testing frame setup did not appear to affect the NASP bond forces for 0.10 in. free end strand slip. In one test the average values were identical, in one test the "stiff" frame resulted in larger values than the "flexible" frame, and in one test the "flexible" frame resulted in larger values than the "stiff" system. The effect of the testing setup can also be seen with Brown's (2003) test setup which was more flexible than the "flexible" test setup of this research. Strand "AA" NASP bond force results for 0.10 in. free end strand slip reported in Brown (2003) were slightly lower than the average of results from the 0.45 and 0.475 water to cement ratio mixtures for the "stiff" and "flexible" setups. The average was used since the compressive strength of the mortar in Brown's work fell between the compressive strengths of the 0.45 and 0.475 mixture strengths. Strand "FF" NASP bond force results for 0.10 in. free end strand slip reported in Brown (2003) were 25 percent lower than the results from the 0.50 water to cement ratio mixture for the "stiff" and "flexible" setups. Brown's data was compared to the 0.50 water to cement ratio mixture since the compressive strengths were similar.

In order to see if the data for the "flexible" and "stiff" testing frames can be considered one data set, the 95% confidence intervals for the NASP bond force averages for each data set were computed. The 95% confidence interval gives the interval where the probability is 95% that the true average is within this interval. This can give an idea as to whether the data is the same data set by checking to see if the average of the "flexible" frame fits into the confidence interval for the "stiff" frame and vice versa. As can be seen in Tables 5.2 and 5.3, not all data fit into the other testing frame's 95% confidence interval. So, it can be concluded that the "flexible" and "stiff" systems do give different NASP bond forces.

Although the test setup stiffness did not have a significant affect on the NASP bond forces, it should be noted that with every mixture, the coefficient of variation was smaller for the "stiff" system.

Table 5.2. Confidence Interval Check for Data Overlap for Strand "AA".

W/C	Slip (in.)	Flexible			Stiff			One Data Set?
		AVG.	95% MAX	95% MIN	AVG.	95% MAX	95% MIN	
0.45	0.01	16.16	17.86	14.46	15.46	15.94	14.97	NO
	0.10	21.84	23.08	20.60	21.30	21.80	20.81	NO
0.475	0.01	14.24	15.56	12.92	13.16	14.24	12.07	YES
	0.10	19.80	21.57	18.02	18.54	19.05	18.02	NO
0.50	0.01	9.25	9.82	8.68	8.96	9.48	8.44	YES
	0.10	11.58	12.66	10.51	10.72	11.10	10.34	NO

Table 5.3. Confidence Interval Check for Data Overlap for Strand "FF".

W/C	Slip (in.)	Flexible			Stiff			One Data Set?
		AVG.	95% MAX	95% MIN	AVG.	95% MAX	95% MIN	
0.45	0.01	9.24	10.04	8.43	9.20	10.51	7.88	YES
	0.10	8.55	8.99	8.11	8.47	8.89	8.05	YES
0.475	0.01	8.05	8.59	7.50	8.80	10.01	7.60	NO
	0.10	7.65	8.28	7.02	7.89	8.83	6.95	YES
0.50	0.01	8.10	8.56	7.64	8.29	8.66	7.92	YES
	0.10	8.43	9.09	7.76	8.43	9.01	7.86	YES

5.4.2.2 NASP Bond Force Related to Compressive Strength

The testing program was designed to determine the relationship between the NASP bond forces and the compressive strength of the mortar. Two strand were tested, Strand "AA" and Strand "FF". It should be noted that in previous tests, Strand "AA" was considered a "good performer", and Strand "FF" was considered a "marginal performer". The two strands do not show similar trends.

The NASP bond forces at 0.01 in. and 0.10 in. free end strand slip varied approximately linearly with the compressive strength of the specimens for Strand "AA". The bond forces also varied approximately linearly with the square root of the compressive strength. Regression analysis demonstrated that the two relationships have similar accuracy.

Although a trend was obvious in Strand "AA", Strand "FF" did not have similar results. Regression analysis resulted in weak correlation of data for Strand "FF". The regression analysis graphs for mortar compressive strength and NASP bond force at 0.10 in. of free end strand slip are shown in Figures 5.5 and 5.6. Tables 5.4, 5.5 5.6, and 5.7 give the results for the regression analysis.

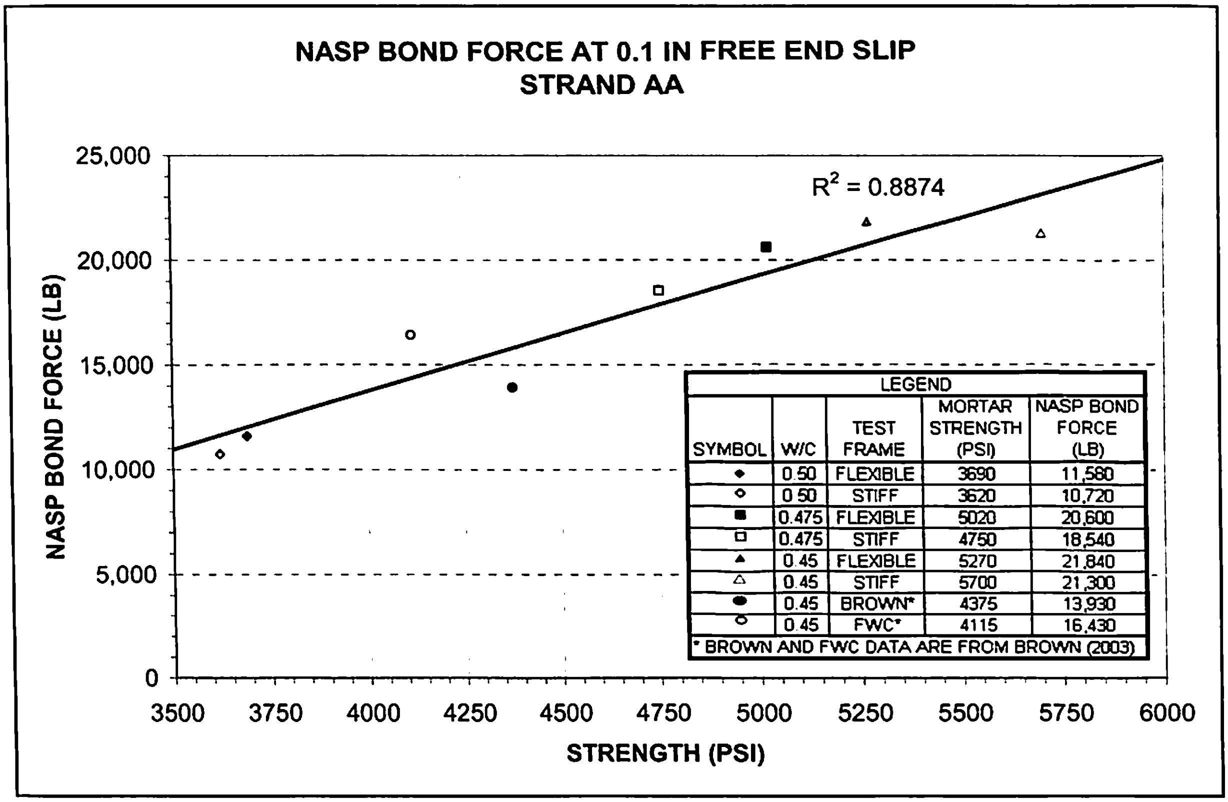


Figure 5.5. Regression Analysis of Mortar Strength and Bond Force for Strand "AA".

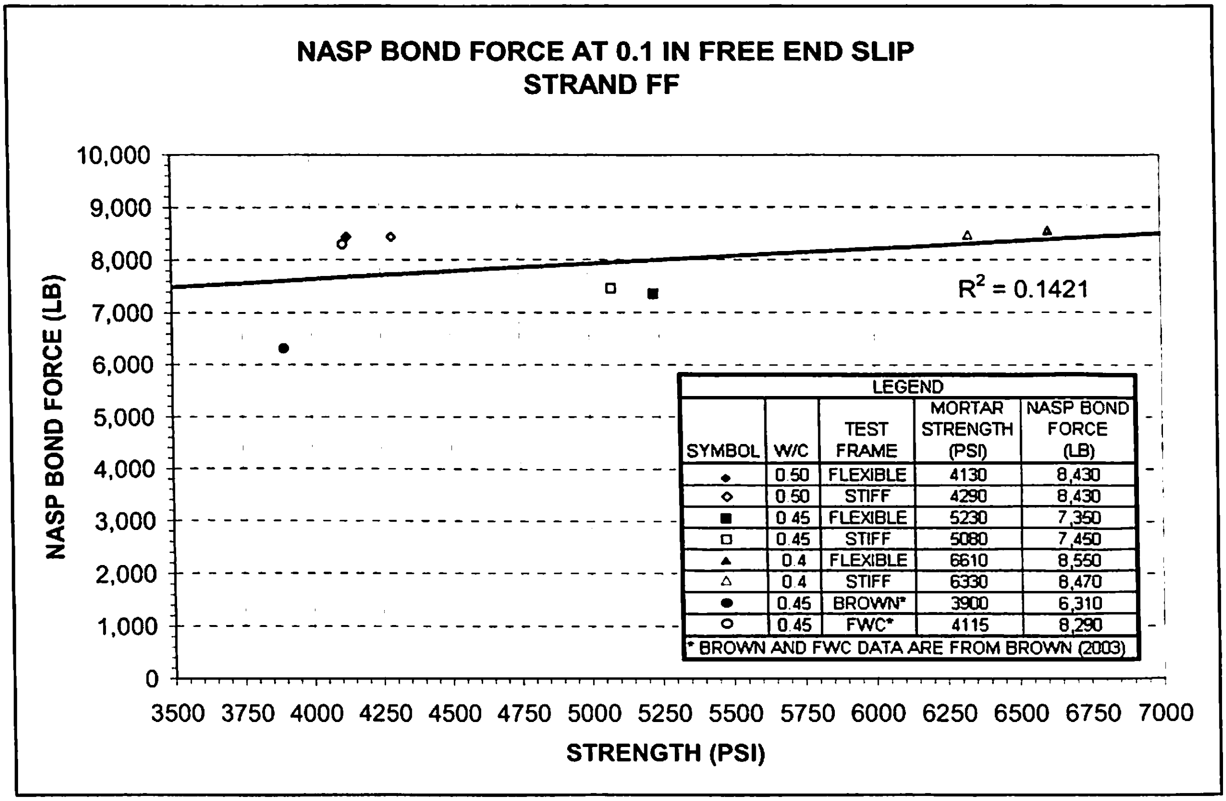


Figure 5.6. Regression Analysis of Mortar Strength and Bond Force for Strand "FF".

Table 5.4. Regression Analysis for Strand "AA" With Separate Values for Each Test Setup.

Data	f_c' Vs. Bond Force	$\sqrt{f_c'}$ Vs. Bond Force	Free End Strand Slip (in.)	R^2
OSU, Brown, and FWC	X		0.01	0.847
	X		0.10	0.887
OSU	X		0.01	0.910
	X		0.10	0.941
OSU, Brown, and FWC		X	0.01	0.847
		X	0.10	0.900
OSU		X	0.01	0.917
		X	0.10	0.952

Table 5.5. Regression Analysis for Strand "FF" With Separate Values for Each Test Setup.

Data	f_c' Vs. Bond Force	$\sqrt{f_c'}$ Vs. Bond Force	Free End Strand Slip (in.)	R^2
OSU, Brown, and FWC	X		0.01	0.571
	X		0.10	0.142
OSU	X		0.01	0.727
	X		0.10	0.025
OSU, Brown, and FWC		X	0.01	0.573
		X	0.10	0.139
OSU		X	0.01	0.712
		X	0.10	0.013

Table 5.6. Regression Analysis for Strand "AA" With Separate Trends for OSU Test Setups.

Data	f_c' Vs. Bond Force	$\sqrt{f_c'}$ Vs. Bond Force	Free End Strand Slip (in.)	R^2
OSU "Flexible"	X		0.01	0.959
	X		0.10	0.999
OSU "Stiff"	X		0.01	0.986
	X		0.10	0.953

Table 5.7. Regression Analysis for Strand "FF" With Separate Trends for OSU Test Setups.

Data	f_c' Vs. Bond Force	$\sqrt{f_c'}$ Vs. Bond Force	Free End Strand Slip (in.)	R^2
OSU "Flexible"	X		0.01	0.774
	X		0.10	0.024
OSU "Stiff"	X		0.01	0.961
	X		0.10	0.027

Based on the regression analysis, for the specified range of the NASP Test, 3500 to 5000 psi, the NASP bond force for Strand "AA" varied linearly with the compressive strength of the mortar. The regression analysis demonstrates that correlation between NASP bond forces and either the compressive strength of the mortar or the square root of the compressive strength of the mortar does not exhibit a need to use the square root of the compressive strength as a factor in the NASP bond forces. Since the correlation is virtually the same for the compressive strength and the square root of that value, the compressive strength value can be assumed to cause a linear variation in the NASP bond force.

Although the data for Strand "AA" suggested a strong trend between compressive strength and NASP bond forces, the same was not true for Strand "FF". The differences may be due to a difference in bond behavior. The results shown graphically in Appendix D reflect an increase in the dip in the force experienced by the strand with an increase in compressive strength. The results also show negative slip occurring. Popping from the specimen was heard when the negative slip occurred. This behavior may also explain why the 0.01 in. free end strand slip NASP bond force was less variable and exhibited a similar trend to Strand "AA".

Based on the regression analysis, the correlation of data was greater when the previous studies were omitted. This suggests that something other than the compressive strength of mortar is affecting the NASP bond forces. The scatter of data may be attributed to materials, but without further testing, this can not be identified as a source of variance. The scatter may also be attributed to strand conditions. Although efforts were made to keep strand in a moisture free environment, the strand did sit for a long time between the earlier testing and current testing. Also, the lab procedures may not have been identical for the strand used in the early tests.

The regression analysis shows a greater correlation when the testing frames are considered separately for Strand "AA". This suggests that the data for the two testing systems are in fact two separate sets of data. Based on the higher correlation when the testing frame results are separated, it can be concluded that the testing frame stiffness does have an affect on the NASP bond forces. The regression analysis does not indicate that one setup consistently results in higher correlation.

5.4.2.3 Loading Rate

The loading rate of the strand was affected by the testing frame setup. The loading rate of the "stiff" system was 7 to 11 percent greater than the "flexible" system. As noted previously, the "flexible" system resulted in slightly larger values for NASP bond forces in Strand "AA", while the trend varied in Strand "FF".

Regression analysis of the loading rate versus NASP bond force for the current research as well as the previous research by Brown and FWC was performed. The testing setup at FWC was stiffer than the "stiff" system tested in this program, and since the loading rate and NASP bond force at 0.10 in. free end strand slip is shown in Figures 5.7 and 5.8. The regression analysis results are given in Table 5.8.

Based on regression analysis, the correlation between NASP bond forces and loading rate is relatively small when all data is considered. Although the correlation does increase when looking at OSU data only, the data does not support a strong relationship between the loading rate and NASP bond force.

Like the mortar compressive strength regression analysis, the correlation of data was greater when the previous studies were omitted. This suggests that something other than the compressive strength of mortar and loading rate is affecting the NASP bond forces. The scatter of data may be attributed to materials, but without further testing, this can not be identified as a source of variance. The scatter may also be attributed to strand conditions. Although efforts were made to keep strand in a moisture free environment, the strand did sit for a long time between the earlier testing and current testing. Also, the lab procedures may not have been identical for the strand used in the early tests.

In order to determine the role of the loading rate on the NASP bond force, the loading rate needs to be investigated while using the same batch of mortar. By using the same batch of mortar, variables, except the natural variance in strand condition, will be eliminated from the testing.

Table 5.8. R^2 for Loading Rate Vs. NASP Bond Force.

Strand	OSU, Brown, and FWC	OSU	OSU "Flexible"	OSU "Stiff"
AA	0.398	0.604	0.921	0.974
FF	0.421	0.008	0.059	0.048

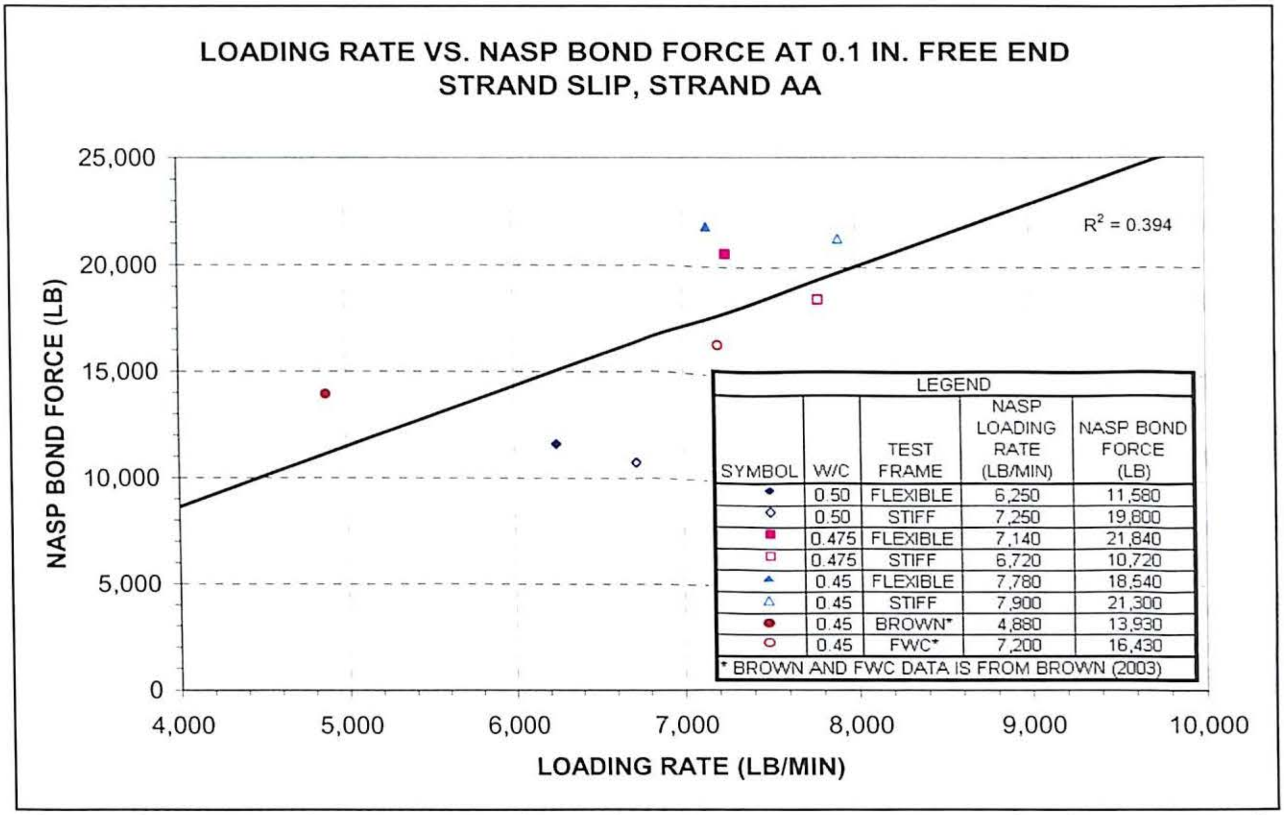


Figure 5.7. Regression Analysis of Loading Rate and Bond Force for Strand "AA".

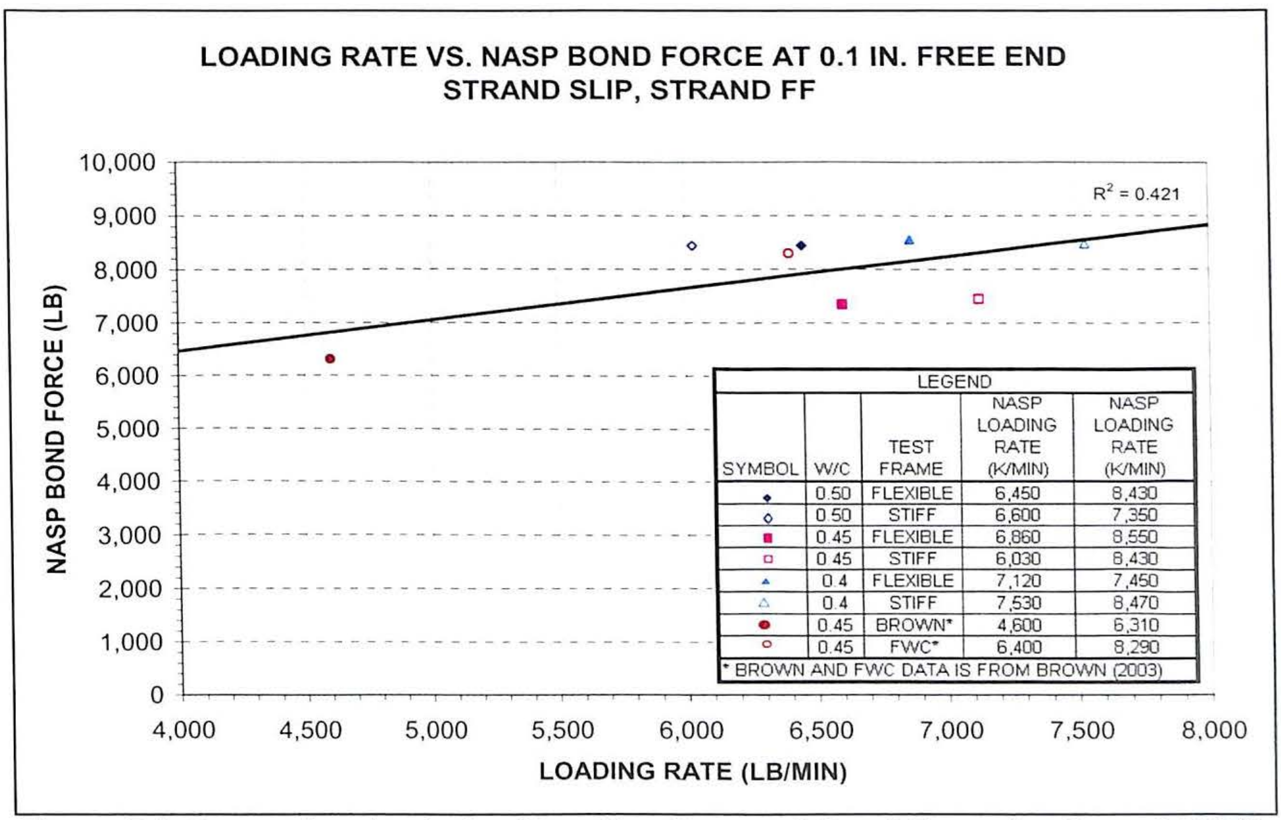


Figure 5.8. Regression Analysis of Loading Rate and Bond Force for Strand "FF".

5.4.3 Recommendations

Based on the testing conducted, some recommendations can be made. Based on the results of the testing, the test frame stiffness does have a significant effect on the results. Since the "stiff" testing frame generally gave slightly smaller coefficient of variation values for NASP bond forces and is less likely to be damaged by incorrect commands in the testing program, the "stiff" system is recommended for future NASP testing. Based on testing, for "good performing" strands, the compressive strength of the mortar has a significant affect on the NASP bond forces and should be accounted for in minimum bond force recommendations.

Further investigation is recommended into factors affecting the NASP bond test. Some studies that may be pursued are different materials sources, varying loading rate while maintaining constant compressive strengths, and additional strand studies.

Chapter 6

SUMMARY AND CONCLUSIONS

6.1 SUMMARY

In order to determine concrete batches for future use and in order to test for variables affecting the results of the NASP Strand Pull-out Test, the following was conducted:

- Concrete batching.
- Mortar batching.
- NASP Testing

6.1.1 Concrete Batching

The concrete was batched in order to achieve the goal of reaching three desired compressive strength combinations with a workable mixture that had a slump of 6 to 8 in. One combination was a 6,000 psi one day strength and a 10,000 psi 28 or 56 day strength. The next combination was a 8,000 psi one day strength and a 14,000 psi 28 or 56 day strength, and the last combination was a 10,000 psi one day strength and a 18,000 psi 28 or 56 day strength.

Admixtures and water to cement ratios had significant effects on the fresh and hardened properties of the concrete. The batching revealed the following:

- Glenium 3200 HES resulted unworkable concrete due to early set times.
- Glenuim 3030 NS resulted in higher compressive strengths than Rheobuild 1000.

- Fly ash replacement had a negative effect on the early compressive strength of concrete and a positive effect on the 28 and 56 day compressive strengths of the concrete.
- Silica fume replacement had no impact until the 28 and 56 day strengths.
- Fly ash in combination with silica fume replacement had no impact on the one day strength and resulted in strengths between the fly ash and silica fume results for 28 and 56 days.
- A reduction in water to cement ratio increases the compressive strength of concrete; however, the magnitude of the increase decreases with time.

6.1.2 Mortar Batching

The mortar was batched in order to obtain fresh properties and 1, 7, and 28 day strength results. The compressive tests were conducted on four cubes at 1, 7, and 28 days.

The batches were made to increase the repeatability of the properties by practice and determine mixture proportions for the NASP testing. The batching revealed the following:

- The compressive strengths for the 0.45 water to cement ratio mortar were similar to those by Russell and Paulsgrove.
- Like concrete, the compressive strengths of the specimens increased with decreasing water to cement ratios.
- In order to obtain one day strengths ranging from approximately 4000 psi to 6000 psi, 0.40, 0.45, and 0.50 water to cement ratios should be used.

6.1.3 NASP Testing

After initial batching, NASP bond tests were conducted. The bond tests were tested according to the NASP test procedures by Russell and Paulsgrove (1999b).

The testing was conducted to determine the effect of the testing frame stiffness and varying mortar strengths on the NASP bond force results. The testing revealed the following:

- The NASP bond force for 0.10 in. free end strand slip varied by less than 8 percent for the two testing frames.
- The NASP bond force coefficient of variation for the "stiff" test frame was less than that for the "flexible" frame.
- The NASP bond forces at 0.01 in. and 0.10 in. free end strand slip varied linearly with the compressive strength of the specimens for Strand "AA".
- Strand "FF" did not exhibit a defined trend for NASP bond forces versus compressive strengths.
- The loading rate of the "flexible" system was 7 to 11 percent greater than the "stiff" system.
- Neither strand exhibited strong correlation between loading rate and NASP bond force.
- Based on the regression analysis of the new data and that by Brown (2003), something other than the compressive strength of mortar and loading rate is affecting the NASP bond forces.
- In the tests conducted, the loading rate appears to be a function of the compressive strength.

6.2 CONCLUSIONS

Based the concrete batching, mortar batching, and NASP Pull-Out Tests, the following can be concluded:

- To achieve 6,000 psi one day and 10,000 psi 28 or 56 day compressive strengths, concrete mixture G3030-8-36-1 with a 0.36 water to cement ratio and 800 pcy of cement using Glenium 3030 can be used.
- To achieve 8,000 psi one day and 14,000 psi 28 or 56 day compressive strengths, concrete mixture G3030-8-32-7 with a 0.32 water to cement ratio and 800 pcy of cement using Glenium 3030 and 10 percent silica fume replacement is the most likely candidate to be used with an increase in the Glenium 3030 dosage rate.
- To achieve 10,000 psi one day and 18,000 psi 28 or 56 day compressive strengths, concrete mixture G3030-8-28-1 with a 0.28 water to cement ratio and 800 pcy of cement using Glenium 3030 is the most likely candidate to be used with an increase in the Glenium 3030 dosage rate and a 10 percent silica fume replacement.
- The "stiff" testing frame is recommended for future NASP testing.
- Studies varying material sources, varying loading rate while maintaining constant compressive strengths, and testing additional strands should be conducted.

REFERENCES

AASHTO LRFD Bridge Design Specifications (1998, Update 2002), American Association of State Highway and Transportation Officials.

ACI Committee 318 (2002), "Building Code Requirements for Structural Concrete (ACI 318-02) and Commentary (318R-02)," American Concrete Institute, Farmington Hills, Michigan, 443 pp.

Anderson, A. R. and Anderson, R.G. (1976). "An Assuance Criterion for Flexural Bond in Pretensioned Hollow Core Units." *ACI Journal*, Vol. 72, No. 8, August, pp. 457-464.

Brearly, L. M. and Johnston, D. W. (1990). "Pull-Out Bond Tests of Epoxy-Coated Prestressing Strand." *Journal of Structural Engineering*, Vol. 116, No. 8, August, pp. 2236-2252.

Brown, M. D. (2003) "Evaluation of Test Methods in Assessing Bond Quality of Prestressing Strand," Master's Thesis. University of Oklahoma, Norman, Oklahoma.

Cousins, T. E., Badeaux, M. H., and Moustafa, S. (1992) "Proposed Test for Determining Bond Characteristics of Prestressing Strand," *PCI Journal*, Vol. 37, No. 1, January-February, pp. 66-73.

Cousins, T. E., Johnston, D. W., and Zia, P. (1990) "Transfer Length of Epoxy-Coated Prestressing Strand," *ACI Materials Journal*, Vol. 87, No. 3, May-June, pp. 193-203.

Cousins, T. E., Johnston, D. W., and Zia, P. (1990) "Development Length of Epoxy-Coated Prestressing Strand," *ACI Materials Journal*, Vol. 87, No. 4, July-August, pp. 309-318.

Cousins, T. E., Stallings, J. M., and Simmons, M. B. (1994) "Reduced Strand Spacing in Pretensioned, Prestressed Members," *ACI Structural Journal*, Vol. 91, No. 3, May-June, pp. 277-286.

Deatherage, J. H., Burdette, E. G., and Chew, C. K. (1994) "Development Length and Lateral Spacing Requirements of Prestressing Strand for Prestressed Concrete Bridge Girders," *PCI Journal*, Vol. 39, No. 1, January-February, pp. 70-83.

Freyne, S. F. (2003) "High Performance Concrete in Oklahoma Designed for Precast/Prestressed Bridge Beams," Doctoral Dissertation. University of Oklahoma, Norman, Oklahoma.

Hale, W. M. (2002) Investigating the Need for and Effect of Air Entrainment in High Performance Concrete," Doctoral Dissertation. University of Oklahoma, Norman, Oklahoma

Hanson, N. W. and Karr, P. H. (1959) "Flexural Bond Tests on Pretensioned Prestressed Beams," *ACI Journal*, Vol. 55, No. 7, January, pp. 783-802.

Hyett, A. J., Dube, S., and Bawden, W. F. (1994) "Laboratory Bond Sytrength Testing of 0.6" 7-Wire Strand from 7 Different Manufacturers." Final Report. Department of Mining Engineering, Queen's University, Kingston, Ontario, November.

Janney, Jack R., (1963) "Report on Stress Transfer Length Studies on 270 k Prestressing Strand, *PCI Journal*, Vol. 8, No. 1, February, pp. 41-45.

Kaar, P. H., LaFraugh, R. W., and Mass, M. A. (1963) "Influence of Concrete Strength on Strand Transfer Length," *PCI Journal*, Vol. 8, No. 5, October, pp. 47-67.

Kaar, P. H. and Magura, D. D. (1965) "Effect of Strand Blanketing on Performance of Pretensioned Girders," *PCI Journal*, Vol. 10, No. 6, December, pp. 20-34.

Kahn, L. F., Dill, J. C., and Reutlinger, C. G. (2002) "Transfer and Development Length of 15-mm Strand in High Performance Concrete Girders," *Journal of Structural Engineering*, Vol. 128, No. 7, July, pp. 913-921.

Logan, D. R. (1996). Discussion of "Acceptance Criteria for Bond Quality of Strand for Pretensioned Prestressed Concrete Applications," *PCI Journal*, Vol. 41, No. 2, March-April, pp. 112-116.

Logan, D. R. (1997). "Acceptance Criteria for Bond Quality of Strand for Pretensioned Prestressed Concrete Applications," *PCI Journal*, Vol. 42, No. 2, March-April, pp. 52-90.

Mitchell, D., Cook, W. D., Khan, A. A., and Tham, T. (1993) "Influence of High Strength Concrete on Transfer and Development Length of Pretensioning Strand," *PCI Journal*, Vol. 38, No. 3, May-June, pp. 52-66.

Peterman, R. J., Ramirez, J. A., and Olek, J. (2000) "Influence of Flexure-Shear Cracking on Strand Development Length in Prestressed Concrete Members," *PCI Journal*, Vol. 45, No. 5, September-October, pp. 76-94.

Rabbat, B.G., Kaar, P. H., Russell, and Bruce (1979) "Fatigue Tests of Pretensioned Girders with Blanketed and Draped Strands," *PCI Journal*, Vol. 24, No. 4, July-August, pp. 89-114.

Rose, D. R. and Russell, B. W. (1997). "Investigation of Standardized Tests to Measure the Bond Performance of Prestressing Strand," *PCI Journal*, Vol. 42, No. 4, July-August, pp. 56-80.

Russell, B. W. and Burns, N. H. (1993). "Static and Fatigue Behavior of Pretensioned Bridge Girders Made with High Strength Concrete," *PCI Journal*, Vol. 38, No. 3, May-June, pp. 118-128.

Russell, B. W., Burns, N. H., and ZumBrunnen, L. G. (1994). "Predicting Bond Behavior of Prestressed Concrete Beams Containing Debonded Strands," *PCI Journal*, Vol. 39, No. 5, September-October, pp. 60-77.

Russell, B. W. and Burns, N. H. (1994). "Fatigue Tests on Pretsressed Concrete Beams Containing Debonded Strands," *PCI Journal*, Vol. 39, No. 6, November-December, pp. 70-88.

Russell, B. W. and Burns, N. H. (1996). "Measured Transfer Lengths of 0.5 and 0.6 in. Strands in Pretensioned Concrete," *PCI Journal*, Vol. 41, No. 5, September-October, pp. 44-65.

Russell, B. W. and Burns, N. H. (1997). "Measured Transfer Lengths on Pretensioned Concrete Elements," *Journal of Structural Engineering*, Vol. 123, No. 5, May, pp. 541-549.

Russell, B. W. and Paulsgrove, G. A. (1999). "NASP Strand Bond Testing, Round One, Pull-Out Tests and Friction Bond Tests of Untensioned Strand," Final Report 99-03, Fears Structural Engineering Laboratory, University of Oklahoma at Norman, July.

Russell, B. W. and Paulsgrove, G. A. (1999). "NASP Strand Bond Testing, Round Two, Assessing Repeatability and Reproducibility of the Moustafa Test, the PTI Test, and the NASP Bond Test," Final Report 99-04, Fears Structural Engineering Laboratory, University of Oklahoma at Norman, July.

Salmons, J. R. and McCrate, T. E. (1977). "Bond Characteristics of Untensioned Prestressing Strand." *PCI Journal*, Vol. 22, No. 1, January-February, pp. 52-65.

Shahawy, M. (2001) "A Critical Evaluation of the AASHTO Provisions for Strand Development Length of Prestressed Concrete Members," *PCI Journal*, Vol. 46, No. 4, July-August, pp. 94-117.

Shing, P. B., Frangopol, D. M., McMullen, M. L., Hutter, W., Cooke, D. E., and Leonard, M. A. (2000). "Strand Development and Transfer Length Tests on High Performance Concrete Box Girders," *PCI Journal*, Vol. 45, No. 5, September-October, pp. 96-109.

Steinberg, E., Beier, J. T., and Sargand, S. (2001) "Effects of Sudden Prestress Force Transfer in Pretensioned Concrete Beams," *PCI Journal*, Vol. 46, No. 1, January-February, pp. 64-75.

Tabatabai, H. and Dickson, T. J. (1995). "The History of the Prestressing Strand Development Length Equation," Federal Highway Administration Report FHWA-RD-93-076, Construction Technology Laboratories, Virginia, February.

Wan, B., Harries, K. A., and Petrou, M. F. (2002) "Transfer Length of the Strands in Prestressed Concrete Piles," *ACI Structural Journal*, Vol. 99, No. 5, September-October, pp. 577-581.

APPENDIX A

Table A.1.1. NASP Test Results Strand AA #1. (Brown 2003)

AA #1		
MTS STROKE (IN)	LVDT #2 (IN)	LOAD (K)
0.01318	0.00148	0.0244
0.0293	-0.00185	0.0488
0.04688	-0.00148	0.1709
0.06446	-0.00148	0.4395
0.08057	-0.00148	0.8789
0.09815	-0.00296	1.4404
0.11573	-0.00148	2.1729
0.13185	-0.00185	3.0273
0.14943	-0.00148	3.54
0.16701	-0.00074	4.2236
0.18312	-0.00259	5.0293
0.19924	-0.00037	5.8838
0.21682	-0.00074	6.7383
0.2344	0.00148	7.5195
0.25198	0.00445	8.252
0.26956	0.00815	8.96
0.28714	0.01297	9.6436
0.30325	0.01964	10.2295
0.31937	0.02816	10.7178
0.33695	0.04002	11.1328
0.3516	0.0478	11.377
0.36771	0.06113	11.5723
0.38383	0.07484	11.6943
0.40141	0.09152	11.8408
0.41899	0.10597	11.9873
0.4351	0.11967	12.1582
0.45268	0.13561	12.2559
0.47026	0.15413	12.3535
0.48638	0.16747	12.4268
0.50396	0.18377	12.4756
0.52154	0.20045	12.5244
0.53765	0.21564	12.5488
0.55523	0.23231	12.6709
0.57281	0.24713	12.8418
0.58893	0.26306	12.915
0.60504	0.27788	13.0615
0.62262	0.29344	13.1836
0.6402	0.30938	13.2324
0.65778	0.32605	13.2813
0.6739	0.34161	13.3789
0.68562	0.35199	13.4033
0.7032	0.37051	13.4033

Table A.1.2. NASP Test Results Strand AA #2. (Brown 2003)

AA #2		
MTS STROKE (IN)	LVDT #2 (IN)	LOAD (K)
0.01465	0.00074	0.0244
0.03076	0.00074	0.0488
0.04834	0.00074	0.2441
0.06592	0.00111	0.5859
0.08204	0.00111	1.0742
0.09962	-0.00222	1.6602
0.1172	0.00037	2.3438
0.13331	0.00185	3.0029
0.15089	0.00296	3.7354
0.16701	0.00148	4.5166
0.18459	0.00111	5.3467
0.20217	0.00222	6.1768
0.21975	0.00222	7.0557
0.23733	0.00556	7.8369
0.25344	0.00815	8.6426
0.26956	0.01186	9.3994
0.28714	0.01593	10.1563
0.30472	0.02112	10.8154
0.32083	0.02631	11.4502
0.33988	0.03298	12.0361
0.3516	0.04039	12.4268
0.36918	0.05076	12.793
0.38676	0.06188	13.1104
0.40287	0.0741	13.4033
0.42045	0.08744	13.5254
0.43803	0.103	13.6475
0.45415	0.11893	13.6719
0.47173	0.13561	13.5986
0.48931	0.15265	13.6963
0.50542	0.1671	13.7207
0.523	0.18266	13.7939
0.54058	0.19859	13.8672
0.5567	0.21304	13.9893
0.57281	0.22638	14.1602
0.59039	0.24194	14.3555
0.60651	0.25602	14.5508
0.62555	0.2701	14.7217
0.64313	0.28381	14.917
0.65925	0.29678	15.1367
0.67683	0.30975	15.3809
0.68708	0.31901	15.5029
0.70466	0.33198	15.6982

Table A.1.3. NASP Test Results Strand AA #3. (Brown 2003)

AA #3		
MTS STROKE (IN)	LVDT #2 (IN)	LOAD (K)
0.01318	0	0.0244
0.0293	0	0.0244
0.04688	0.00111	0.0732
0.06299	0	0.0488
0.08057	0.00037	0.0977
0.09815	0	0.2197
0.11427	0	0.4395
0.13185	0.00148	0.8789
0.14943	0.00037	1.4404
0.16701	0	2.0996
0.18312	0.00074	2.7832
0.2007	0.00037	3.5156
0.21828	-0.00111	4.2969
0.2344	0.00074	5.127
0.25198	0.00185	6.0059
0.26956	0.00074	6.8604
0.28567	0.00111	7.7393
0.30325	0.00222	8.5938
0.32083	0.00519	9.4238
0.33695	0.00852	10.2051
0.3516	0.01186	10.8398
0.36771	0.01593	11.5967
0.38529	0.02112	12.3291
0.40287	0.02742	12.9395
0.41899	0.0352	13.4766
0.43657	0.04483	13.916
0.45415	0.05521	14.2822
0.47026	0.06706	14.624
0.48784	0.08003	14.9902
0.50542	0.08929	15.332
0.52154	0.10115	15.6006
0.53912	0.11301	15.8691
0.55523	0.12634	16.1133
0.57281	0.13857	16.3818
0.58893	0.15376	16.5527
0.60651	0.16562	16.6992
0.62409	0.17859	16.9434
0.6402	0.1923	17.0898
0.65632	0.206	17.2852
0.67536	0.22194	17.2607
0.68708	0.23379	17.2607
0.7032	0.24898	17.3584

Table A.1.4. NASP Test Results Strand AA #4. (Brown 2003)

AA #4		
MTS STROKE (IN)	LVDT #2 (IN)	LOAD (K)
0.01465	-0.00037	0.0244
0.03076	-0.00037	0.0732
0.04834	-0.00371	0.2197
0.06592	-0.00074	0.4883
0.08204	0	0.9033
0.09962	-0.00037	1.416
0.1172	0	2.0508
0.13331	0	2.71
0.15089	-0.00037	3.4424
0.16847	0	4.1992
0.18605	0.00037	4.9805
0.20217	0.00074	5.8105
0.21975	0.00148	6.6406
0.23586	0.00185	7.4463
0.25344	0.00296	8.2764
0.27102	0.00593	9.0576
0.2886	0.00963	9.7656
0.30618	0.01482	10.4736
0.3223	0.02112	11.084
0.33841	0.02816	11.6455
0.35306	0.03483	12.0361
0.37064	0.0452	12.4512
0.38676	0.05558	12.793
0.40434	0.06743	13.0615
0.42045	0.08077	13.2568
0.43657	0.09448	13.4033
0.45561	0.10893	13.5742
0.47173	0.12338	13.7451
0.48931	0.13894	13.8672
0.50396	0.15339	13.916
0.523	0.16932	13.9404
0.54058	0.18451	13.9893
0.5567	0.20267	13.9404
0.57428	0.2186	13.9893
0.59186	0.23416	14.0869
0.60944	0.24898	14.1602
0.62555	0.26566	14.2578
0.64313	0.27937	14.3799
0.65925	0.29567	14.502
0.67683	0.31012	14.624
0.68855	0.32086	14.6484
0.70466	0.33605	14.7461

Table A.1.5. NASP Test Results Strand AA #5. (Brown 2003)

AA #5		
MTS STROKE (IN)	LVDT #2 (IN)	LOAD (K)
0.01318	-0.00037	0
0.0293	-0.00037	0.0244
0.04688	-0.00037	0.0244
0.06446	-0.00074	0.0977
0.08057	-0.00037	0.3418
0.09962	-0.00037	0.708
0.11573	-0.00074	1.2207
0.13331	0.00074	1.8066
0.14943	-0.00074	2.4658
0.16701	-0.00037	3.1982
0.18459	-0.00037	3.9307
0.2007	-0.00037	4.7119
0.21828	0	5.5176
0.23586	0.00074	6.3721
0.25198	0.00037	7.1777
0.26956	0.00074	8.0078
0.28714	0.00185	8.8379
0.30472	0.00445	9.5947
0.32083	0.00852	10.3027
0.33841	0.01371	11.0107
0.3516	0.01853	11.5479
0.36771	0.02482	12.1582
0.38676	0.0326	12.7197
0.40287	0.04187	13.1348
0.42045	0.05298	13.4766
0.43657	0.06521	13.7451
0.45415	0.07966	13.9404
0.47173	0.09263	14.1602
0.48784	0.10782	14.2334
0.50396	0.12301	14.3555
0.52154	0.13857	14.4775
0.53912	0.15302	14.6484
0.55523	0.16451	14.7949
0.57281	0.1834	14.917
0.59039	0.19896	14.9902
0.60651	0.21453	15.0879
0.62409	0.22935	15.2588
0.64167	0.23898	15.4785
0.65778	0.2575	15.6738
0.67536	0.27158	15.8203
0.68708	0.2827	15.8691
0.70466	0.29789	16.0156

Table A.1.6. NASP Test Results Strand AA #6. (Brown 2003)

AA #6		
MTS STROKE (IN)	LVDT #2 (IN)	LOAD (K)
0.01465	-0.00037	0
0.03076	-0.00037	0.0244
0.04834	-0.00037	0.0488
0.06592	-0.00037	0.2441
0.08204	-0.00037	0.5371
0.10108	0	1.001
0.1172	0	1.5625
0.13478	-0.00037	2.1973
0.15236	-0.00037	2.9053
0.16847	0	3.6377
0.18605	0	4.4189
0.20217	0	5.2246
0.21975	0.00037	6.0547
0.23586	0.00074	6.8604
0.25344	0.00296	7.6172
0.27102	0.0063	8.374
0.2886	0.01	9.1064
0.30472	0.01408	9.8389
0.32376	0.02112	10.5713
0.33841	0.02371	11.2549
0.35306	0.02927	11.7676
0.37064	0.03631	12.3535
0.38822	0.04483	12.915
0.40434	0.05261	13.4033
0.42045	0.06447	13.7695
0.43803	0.07595	14.1846
0.45561	0.08781	14.502
0.47319	0.10041	14.7705
0.48931	0.11264	15.0635
0.50689	0.12449	15.4053
0.523	0.13783	15.7227
0.54058	0.1508	15.9668
0.55816	0.16414	16.2354
0.57428	0.1771	16.5283
0.59186	0.1897	16.748
0.60944	0.20415	17.0166
0.62555	0.21712	17.2607
0.64313	0.22898	17.5293
0.65925	0.23861	17.7979
0.67683	0.25862	17.9932
0.68855	0.26566	18.0908
0.70613	0.28011	18.2861

Table A.1.7. NASP Test Results Strand FF #1. (Brown 2003)

FF #1		
MTS STROKE (IN)	LVDT #2 (IN)	LOAD (K)
0.01465	0	0.02441
0.03076	0.00037	0.02441
0.04834	0.00037	0.04883
0.06446	0	0.04883
0.08204	0	0.12207
0.09962	0	0.29297
0.1172	0.00037	0.61035
0.13331	0.00074	1.09863
0.15089	0.00333	1.68457
0.16847	0.00037	2.34375
0.18459	0	3.07617
0.20217	0	3.80859
0.21828	0.00037	4.61426
0.23586	0.00037	5.39551
0.25344	0	6.17676
0.26956	0	6.90918
0.28714	-0.00037	7.2998
0.30472	0.00926	6.61621
0.3223	0.02816	6.39648
0.33841	0.04854	6.00586
0.35306	0.06521	5.71289
0.36918	0.08336	5.56641
0.38676	0.10115	5.49316
0.40434	0.1193	5.37109
0.42045	0.13598	5.37109
0.43803	0.15302	5.37109
0.45415	0.16969	5.39551
0.47173	0.18785	5.27344
0.48931	0.20489	5.27344
0.50542	0.22231	5.24902
0.523	0.24009	5.27344
0.53912	0.25602	5.24902
0.5567	0.27344	5.27344
0.57428	0.29233	5.24902
0.58893	0.30789	5.24902
0.60797	0.32531	5.22461
0.62409	0.34346	5.24902
0.64167	0.36014	5.32227
0.65778	0.37792	5.32227
0.67536	0.39645	5.39551
0.68708	0.40682	5.44434
0.70466	0.42461	5.41992

Table A.1.8. NASP Test Results Strand FF #2. (Brown 2003)

FF #2		
MTS STROKE (IN)	LVDT #2 (IN)	LOAD (K)
0.01465	0	-0.02441
0.03223	0	0.02441
0.04834	-0.00111	0.04883
0.06446	0.00037	0.1709
0.08204	0	0.41504
0.09962	0.00037	0.87891
0.1172	0	1.46484
0.13478	0	2.12402
0.15089	-0.00037	2.85645
0.16847	-0.00037	3.6377
0.18605	0	4.41895
0.20363	0.00037	5.2002
0.21975	0.00074	5.95703
0.23733	0.00074	6.71387
0.25344	0.00333	7.34863
0.27102	0.02482	6.98242
0.28714	0.04743	6.5918
0.30472	0.06669	6.4209
0.3223	0.08411	6.4209
0.33988	0.09967	6.49414
0.35306	0.11523	6.46973
0.36918	0.1319	6.51855
0.38676	0.1508	6.49414
0.40287	0.16895	6.44531
0.42045	0.18674	6.39648
0.43803	0.20378	6.4209
0.45415	0.22194	6.37207
0.47173	0.24157	6.25
0.48931	0.25973	6.22559
0.50542	0.27714	6.27441
0.523	0.29381	6.32324
0.54058	0.31012	6.44531
0.5567	0.32605	6.56738
0.57428	0.34272	6.64063
0.59039	0.35865	6.68945
0.60797	0.37829	6.51855
0.62555	0.39719	6.39648
0.64167	0.4146	6.4209
0.65925	0.43127	6.44531
0.67536	0.44832	6.4209
0.68708	0.46277	6.25
0.70466	0.48018	6.20117

Table A.1.9. NASP Test Results Strand FF #3. (Brown 2003)

FF #3		
MTS STROKE (IN)	FREE END SLIP (IN)	LOAD (K)
0.01465	0.00037	0.02441
0.03076	-0.00185	0
0.04834	-0.00037	0.02441
0.06592	-0.00333	0.02441
0.08204	-0.00037	0.04883
0.09962	-0.00037	0.07324
0.1172	0	0.09766
0.13331	0	0.14648
0.15089	0.00111	0.24414
0.16701	0	0.46387
0.18459	-0.00037	0.83008
0.20217	-0.00037	1.36719
0.21828	-0.00037	1.97754
0.23586	0	2.70996
0.25344	0.00037	3.49121
0.27102	0.00037	4.27246
0.2886	0.00074	5.07813
0.30472	0.00037	5.85938
0.3223	0.00074	6.61621
0.33988	0.00259	7.32422
0.35306	0.01	7.49512
0.37064	0.03001	7.20215
0.38676	0.05113	6.95801
0.40287	0.07003	6.7627
0.42045	0.08892	6.64063
0.43803	0.10671	6.54297
0.45561	0.12523	6.44531
0.47173	0.14154	6.49414
0.48784	0.16006	6.39648
0.50542	0.17822	6.32324
0.523	0.19785	6.27441
0.54058	0.21416	6.27441
0.5567	0.2349	6.0791
0.57428	0.25047	6.10352
0.59186	0.26714	6.15234
0.60797	0.28418	6.22559
0.62555	0.30048	6.25
0.64167	0.31827	6.27441
0.65778	0.33309	6.29883
0.67536	0.35013	6.37207
0.68708	0.36162	6.39648
0.70466	0.37755	6.49414

Table A.1.10. NASP Test Results Strand FF #4. (Brown 2003)

FF #4		
MTS STROKE (IN)	FREE END SLIP (IN)	LOAD (K)
0.01318	-0.00074	0.02441
0.0293	0	0.02441
0.04688	0	0.04883
0.06446	0	0.04883
0.08057	0	0.09766
0.09815	-0.00074	0.24414
0.11573	0.00037	0.5127
0.13185	-0.00148	0.97656
0.14943	0.00037	1.53809
0.16701	-0.00037	2.14844
0.18312	0	2.88086
0.2007	0.00037	3.66211
0.21828	0.00037	4.46777
0.2344	0.00037	5.27344
0.25198	0.00037	6.0791
0.26956	0.00037	6.83594
0.28567	0.0063	7.25098
0.30325	0.03001	6.83594
0.32083	0.05002	6.66504
0.33841	0.0678	6.64063
0.3516	0.08262	6.54297
0.36918	0.10189	6.39648
0.38529	0.11967	6.39648
0.40287	0.13635	6.4209
0.41899	0.15339	6.4209
0.43657	0.17192	6.37207
0.45268	0.18674	6.37207
0.47026	0.20415	6.49414
0.48784	0.22045	6.56738
0.50542	0.23602	6.64063
0.52154	0.25306	6.66504
0.53912	0.26973	6.66504
0.55523	0.2864	6.64063
0.57281	0.30419	6.5918
0.59039	0.32086	6.5918
0.60651	0.33642	6.64063
0.62409	0.35273	6.68945
0.64167	0.36792	6.73828
0.65778	0.38237	6.81152
0.67536	0.39867	6.90918
0.68708	0.40978	6.90918
0.7032	0.42386	7.00684

Table A.1.11. NASP Test Results Strand FF #5. (Brown 2003)

FF #5		
MTS STROKE (IN)	FREE END SLIP (IN)	LOAD (K)
0.01318	0.00074	0
0.03076	-0.00037	0
0.04834	-0.00037	0.02441
0.06446	-0.00037	0.04883
0.08204	-0.00037	0.14648
0.09962	-0.00037	0.3418
0.1172	-0.00037	0.65918
0.13331	-0.00037	1.12305
0.15089	-0.00037	1.70898
0.16847	-0.00037	2.39258
0.18459	0	3.125
0.20217	-0.00185	3.88184
0.21828	0.00074	4.66309
0.23586	0.00074	5.44434
0.25344	0.00074	6.22559
0.26956	0.00111	6.95801
0.28714	0.00333	7.61719
0.30472	0.02371	7.27539
0.32083	0.04705	6.93359
0.33841	0.06706	6.71387
0.3516	0.08299	6.64063
0.36918	0.10189	6.54297
0.38529	0.12153	6.4209
0.40287	0.14265	6.15234
0.42045	0.16117	6.0791
0.43657	0.17896	5.95703
0.45415	0.20045	5.9082
0.47173	0.21823	5.85938
0.48784	0.23602	5.9082
0.50542	0.25232	5.93262
0.52154	0.27195	5.88379
0.53912	0.28937	5.93262
0.5567	0.30715	5.88379
0.57281	0.3242	5.95703
0.58893	0.34161	5.98145
0.60797	0.35828	6.00586
0.62409	0.37718	6.00586
0.64167	0.39311	6.05469
0.65778	0.40978	6.0791
0.67536	0.42683	6.15234
0.68708	0.43906	6.12793
0.70466	0.4561	6.17676

Table A.1.12. NASP Test Results Strand FF #6. (Brown 2003)

FF #6		
MTS STROKE (IN)	FREE END SLIP (IN)	LOAD (K)
0.01465	-0.00111	0.02441
0.03076	0	0.04883
0.04834	0	0.04883
0.06592	-0.00037	0.07324
0.08204	0.00185	0.1709
0.09962	-0.00037	0.43945
0.1172	0.00037	0.80566
0.13331	0	1.26953
0.15089	0.00371	1.80664
0.16847	0.00037	2.44141
0.18459	0	3.125
0.20217	0.00037	3.85742
0.21975	0.00074	4.61426
0.23733	0.00111	5.37109
0.25344	0.00111	6.10352
0.26956	0.00148	6.83594
0.28714	0.00704	7.2998
0.30472	0.02668	7.08008
0.32083	0.04817	6.7627
0.33841	0.06632	6.5918
0.3516	0.08262	6.4209
0.36918	0.10078	6.32324
0.38676	0.11856	6.29883
0.40287	0.13894	6.17676
0.42045	0.15636	6.12793
0.43803	0.17488	5.95703
0.45415	0.19341	5.88379
0.47173	0.21082	5.85938
0.48784	0.22712	5.9082
0.50542	0.24417	5.95703
0.523	0.26158	6.00586
0.53912	0.27677	6.00586
0.5567	0.29381	6.05469
0.57281	0.31234	6.10352
0.59039	0.32938	6.10352
0.60651	0.34754	6.15234
0.62409	0.36421	6.15234
0.64167	0.38126	6.15234
0.65925	0.39793	6.22559
0.67536	0.41571	6.20117
0.68708	0.42794	6.17676
0.70466	0.44387	6.22559

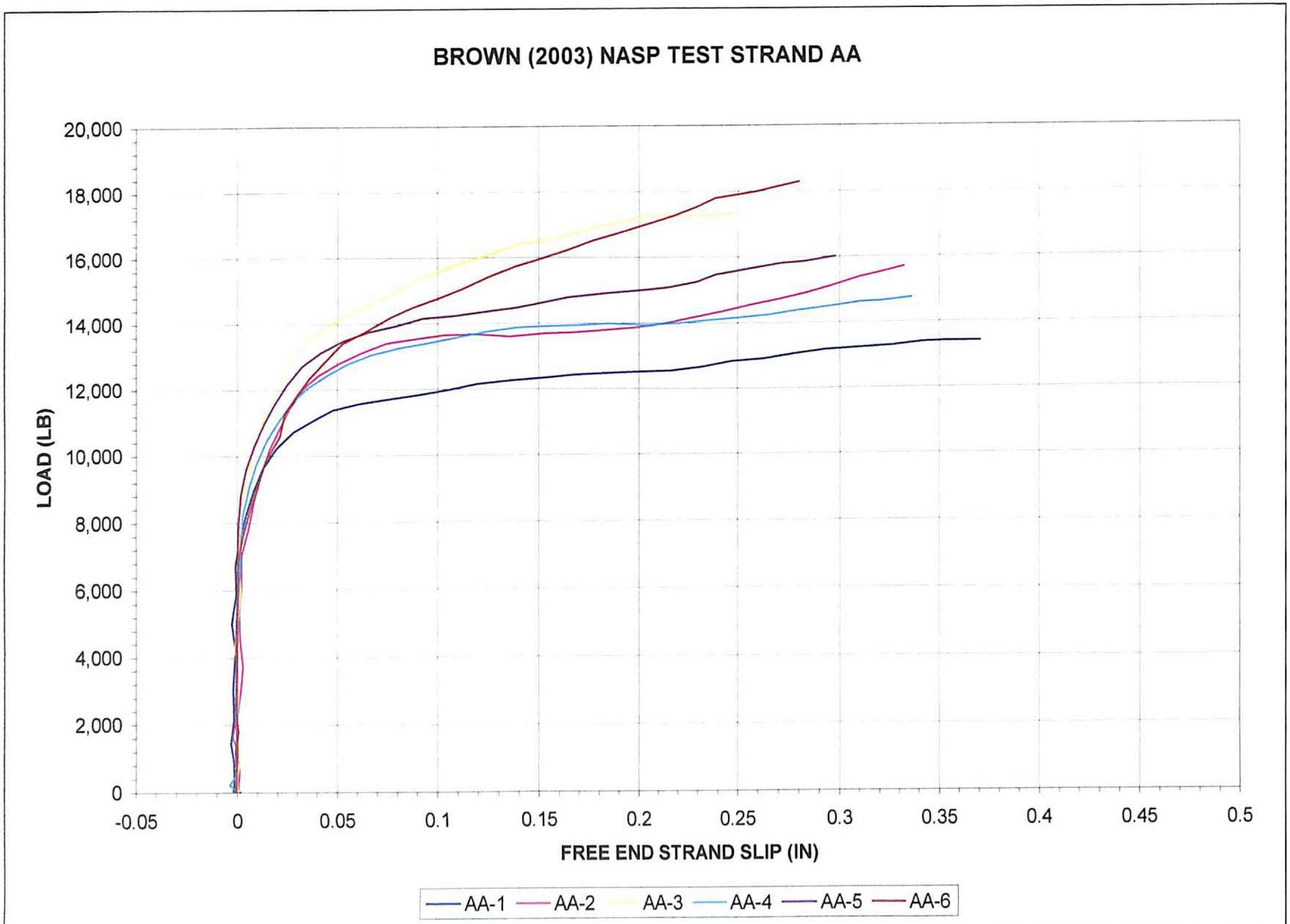


Figure A.1.1. NASP Test Results Strand AA. (Brown 2003)

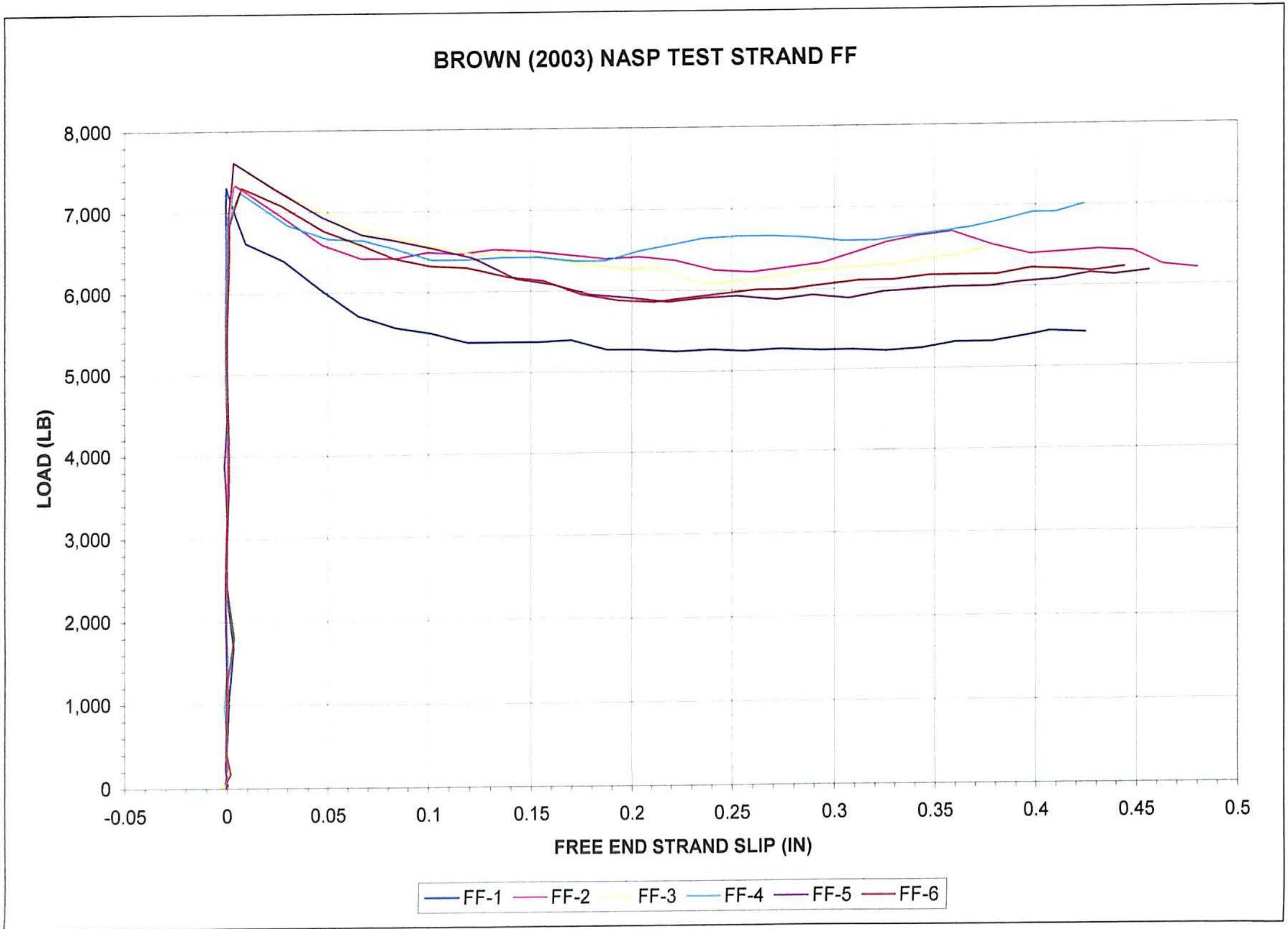


Figure A.1.2. NASP Test Results Strand FF. (Brown 2003)

APPENDIX B

NASP STRAND BOND TEST (DRAFT)

Standard Test Method to Assess the Bond of 0.5 in. (12.7 mm) Seven Wire Strand with Cementitious Materials

1. Scope

1.1 This test method provides a means to assess the ability of 0.5 in. (12.7 mm) seven wire strand to bond with concrete and other cementitious products. The method tests the bondability of strands that are made and intended for use as prestressing strands that conform to ASTM A 416.

1.2 This test does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the use of this test method to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.

2. Reference Documents

2.1 ASTM A 416
2.2 ASTM C 33
2.3 ASTM C 150
2.4 ASTM C 192

3. Summary of the Test Method

Test specimens are prepared by casting a single, 0.5 in. (12.7 mm) seven wire strand into a cylinder of concrete mortar with a bonded length of 16 in. (400 mm). The constituents and proportions for the concrete mortar mixture are prescribed. The concrete in the specimen is cured for approximately one day under controlled conditions. The specimen is tested at one day of age by pulling the strand through the mortar at a prescribed rate of loading. The pull-out force is recorded at 0.10 in. (2.5 mm) of total slip. A single NASP Bond Test shall consist of 6 or more individual pull-out tests. The strand for the NASP Bond Test shall be taken from the same lot or reel of strand.

4. Preparation of Test Specimens

4.1 Strand Specimens. The strand shall conform to ASTM A 416 and shall be intended for use in pretensioned or post-tensioned applications. Strand specimens for a single NASP Strand Bond Test shall be taken from the same lot or the same reel of prestressing strand. A minimum of six strand specimens are required for a single NASP Strand Bond Test.

4.2 Concrete Mortar Mixture Constituents and Proportions. The concrete mortar mixture shall consist of sand, cement and water mixed thoroughly in the following proportions: 2 parts sand, 1 part cement and 0.45 parts water. The sand shall conform to ASTM C 33 requirements for Fine Aggregate. The batch weight for sand shall be computed using the aggregate's unit weight at saturated surface dry (SSD) conditions. In computing weights for mixture proportions, the moisture content within the sand shall be accurately sampled and measured. The mixture proportions shall be corrected for the moisture content measured in the sand prior to mixing. Batch materials shall be handled in conformance with ASTM C 192. The cement shall conform to ASTM C 150 requirements for Type III cement. The water shall be potable and suitable for making concrete.

4.3 Mixing. The concrete mortar and the test specimens shall be made in conformance with ASTM C 192. Measurements of slump and air content are not required.

4.4 Curing. The concrete mortar and test specimens shall be cured in conformance with ASTM C 192. The concrete mortar shall be cured at 73 ± 3EF (23 ± 2EC) from the time of molding until the moment of test. Storage during the curing period shall be in a vibration-free environment.

4.5 Mortar Strength. Concrete mortar strength shall be evaluated in conformance with ASTM C 109 using 2 in. (51 mm) mortar cubes, except that the mixture proportions for the mortar are given in Section 4.1 and flow measurement shall not be required. The average mortar cube strength at the time of the NASP Bond Test shall not be less than 3500 psi (500 kPa). Mortar cube strength shall not exceed 5000 psi (700 kPa) at the time of the NASP test.

4.6 Test specimens shall be made by casting one single strand concentrically in concrete mortar within a 5 in. (125 mm) diameter steel casing as described in Fig. B.1. The length of the steel tube shall be 18 in. as shown. The bonded length of the strand shall be 16 in., with a 2 in. long bond breaker as shown in the figure. The steel casing shall have sufficient rigidity to prevent radial cracking in the specimen during testing. The test specimen shall be cast with the longitudinal axis of the strand and the steel casing in the vertical position. Test specimens shall be mechanically consolidated by vibration in conformance with ASTM C 192.

5. Test Procedure.

5.1 Timing of the Test. The NASP Bond Test shall be conducted 24 ± 2 hrs. from the time of casting the specimens.

5.2 Instrumentation and measurement. The pull-out force shall be measured by a calibrated load measuring device, either electronically or hydraulically, or in combination of hydraulics and electronics. Pull-out force shall be measured to the nearest 10 lb increments. The relative movement of the strand to the hardened concrete mortar shall be measured. This measurement is typically called the "free-end slip" and shall be measured to 0.01 in. The slip shall be measured by a calibrated device.

5.3 Strand shall be pulled from the concrete by reacting against the transverse steel plate. The loading shall be controlled by strand displacement measured at the point where the load is applied to the strand. The displacement rate shall be 0.1 in. per minute (2.5 mm per minute).

5.4 The strand shall be loaded at a distance approximately 6 in. from the end of the specimen.

5.5 The pull-out force shall be recorded when the opposite end of the strand, or the "free end" achieves a total displacement of 0.10 in. relative to the hardened concrete mortar.

5.6 If the hardened concrete mortar exhibits cracking in two or more of the six individual tests, then all results of NASP Strand Bond Test shall be discarded and new specimens prepared for a new NASP Strand Bond Test.

6. Reporting.

6.1 Sample Size. A single NASP Strand Bond Test shall consist of a minimum of six (6) individual tests conducted on single strand specimens.

6.2 For each individual test, report the pull-out force that corresponds to a relative displacement of 0.10 in. between the strand and the hardened concrete mortar.

6.3 For the NASP Bond Test, compute the average pull-out force from the individual tests and report the value as the average value for the NASP Bond Test. If one of the specimens exhibited radial cracking during testing, disregard the pull-out value of that specimen when reporting results. If two or more of the specimens exhibit radial cracking, the entire results should be disregarded and the NASP Bond Test performed again in its entirety.

7. Acceptance

7.1 The strand shall be accepted for pretensioned and post-tensioned prestressed applications when the average value of the NASP Strand Bond Test is not less than _________ lbs and no individual test result is less than __________lbs.

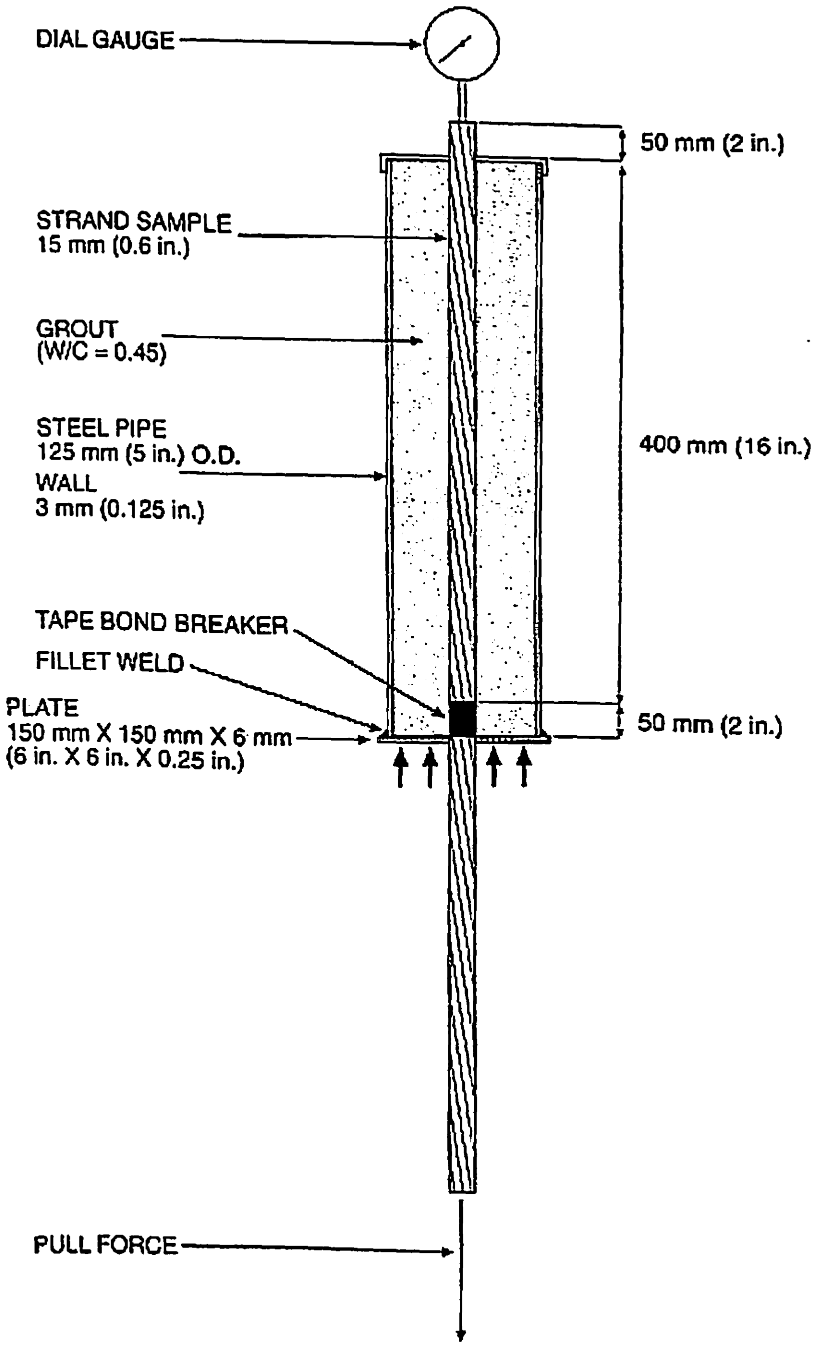


Figure 10.1 Strand Bond Capacity Test Arrangement

Figure B.1. NASP Test Setup

APPENDIX C

Table C.1.1. Sieve Analysis for Original Coarse Aggregate.

Sieve Size	Percent Retained	Percent Coarser	Percent Passing
1 in.	0 %	0 %	100 %
3/4 in.	17.9 %	17.9 %	82.1 %
1/2 in.	29.5 %	47.4 %	52.6 %
3/8 in.	41.4 %	88.8 %	11.2 %
No. 4	11.0 %	99.8 %	0.2%
Pan	0.2 %	100 %	0%

Table C.1.2. Sieve Analysis for Dolese Coarse Aggregate.

Sieve Size	Percent Retained	Percent Coarser	Percent Passing
1 in.			
3/4 in.			
1/2 in.	0 %	0 %	100 %
3/8 in.	1.2 %	1.2 %	98.8 %
No. 4	82.6 %	83.8 %	16.2 %
Pan	16.2 %	100 %	0 %

Table C.1.3. Sieve Analysis for Original Fine Aggregate.

Sieve Size	Percent Retained	Percent Coarser	Percent Passing
No. 4	0.7 %	0.7 %	99.3 %
No. 8	3.9 %	4.6 %	95.4 %
No. 16	16.7 %	21.3 %	78.7 %
No. 30	32.5 %	53.7 %	46.3 %
No. 50	31.1 %	84.9 %	15.1 %
No. 100	13.1 %	98.0 %	2.0 %
No. 200	1.0 %	99.0 %	1.0 %
Pan	1.0 %	100 %	0 %

Table C.1.4. Sieve Analysis for Dolese Fine Aggregate.

Sieve Size	Percent Retained	Percent Coarser	Percent Passing
No. 4	0.7 %	0.7 %	99.3 %
No. 8	3.9 %	4.6 %	95.4 %
No. 16	16.7 %	21.3 %	78.7 %
No. 30	32.5 %	53.7 %	46.3 %
No. 50	31.1 %	84.9 %	15.1 %
No. 100	13.1 %	98.0 %	2.0 %
No. 200	1.0 %	99.0 %	1.0 %
Pan	1.0 %	100 %	0 %

Chemical Analysis of Type III Cement.

Lafarge North America, River - Region

LABORATORY TEST REPORT FOR TYPE IIILA PORTLAND CEMENT

Lafarge North America
2609 North 145th East Ave.
Tulsa, Oklahoma 74116
918-437-3902

Shipped: ________

Production Date: 06-Feb-03

Silo: ____ 1

Car: ________

Mill
Anal # 020603
CERT # TIII-03002T

Quantity: ________

Chemical Tests		Specification	Physical Tests		Specification
			SPECIFIC GRAVITY: 3.15		
SiO_2:	20.38	20.0 Min.	COMPRESSIVE STRENGTHS - (psi)		
Al_2O_3:	4.49	6.0 Max.	1 DAY:	3650	
Fe_2O_3:	2.57	6.0 Max.	3 DAY:	5200	
CaO:	64.29		7 DAY:	6143	
MgO:	2.41	6.0 Max.	28 DAY:		
SO_3:	3.01	3.5 Max.	SETTING TIME (Vicat) - (mins)		
LOI:	0.24	3.0 Max	INITIAL:	120	60 Min.
Na_2O:	0.22				
K_2O:	0.38		FINAL:	195	600 Max.
Na_2O eq.:	0.47		FALSE SET:	69.4 %	50 Min.
Ins. Res.:	0.32	0.75 Max	BLAINE:	5910	2800 Min.
C_3S:	64		% 325 MESH	98.23 %	
C_2S:	10		% EXPANSION:		0.80 Max.
C_3A:	8		% AIR:	10.2	12 Max.
C_4AF:	8				

LAFARGE NORHT AMERICA, Cements are guaranteed to comply with the current ASTM Specifications C150; FEDERAL Specifications SS-C 1960/3B; and AASHTO Specification M85.

MILL TEST REPORT DATE: 02/12/2003

Subscribed and sworn

before me this _____day of

__________________19__

____

Lafarge North America

David McNitt
Chemist

APPENDIX D

Table D.1.1. Concrete Batches with Original Aggregate.

			8-28-1	8-28-2	8-28-3	8-28-4	8-28-5	8-28-6	6.5-40-1	6.5-40-2	6.5-40-3	6.5-36-1
Mix Proportions	Cement (PCY)		888.8	795.4	795.4	795.4	800	800	650	650	650	650
	Fly Ash (PCY)		-	-	-	-	-	-	-	-	-	-
	Silica Fume (PCY)		-	-	-	-	-	-	-	-	-	-
	Coarse Agg. (PCY)		1608.0	1795.7	1795.7	1795.7	1806.1	1806.1	1784.6	1784.6	1784.6	1784.6
	Fine Agg. (PCY)		1400.5	1252.5	1252.5	1252.5	1259.8	1259.8	1311.0	1311.0	1311.0	1378.4
	Water (PCY)		253	222.7	222.7	222.7	224	224	260	260	260	234
	Polyheed 997 (fl. oz/cwt)		-	-	-	-	3	-	3	3	3	21
	Glenium 3200HES (fl. oz/cwt)		22.5	15	12.5	10	10	12.5	6	3	4.5	-
	Glenium 3030NS (fl. oz/cwt)		-	-	-	-	-	-	-	-	-	-
	Rheobuild 1000 (fl. oz/cwt)		-	-	-	-	-	-	-	-	-	-
	Pozzolith 100 XR (fl. oz/cwt)		-	-	-	-	-	3	-	-	-	-
	w/cm		0.285	0.280	0.280	0.280	0.28	0.28	0.40	0.40	0.40	0.36
Fresh Properties	Air Temperature (°F)		-	73	70	79	76	84	70	71	78	82
	Relative Air Humidity (%)		-	88.5	92	79	75	70.5	85	92	79	82
	Concrete Temperature (°F)		80	84	82	86	87	90	80	78	-	87.5
	Slump (in.)		11.75	10	9.5	1.75	1.75	9.5	10.25	3.5	8.75	0.75
	Unit Weight (pcy)		155	155	155	155	154	154	149	149	147	149
	Air Content (%)		1.2	3.5	1.8	3.0	3.0	1.9	3.0	3.2	3.9	3.6
Hardened Properties	Compressive Strength in psi (C.O.V.)	1 Day	9080 (0.5%)	10,180 (0.8%)	10,090 (1.2%)	9430 (5.1%)	9250 (0.6%)	9540 (1.1%)	5780 (1.8%)	4960 (1.9%)	5130 (2.2%)	5930 (3.4%)
		3 Day	11,110 (1.2%)	10,530 (0.4%)	11,090 (1.9%)	11,010 (1.2%)	10,070 (1.1%)	11,200 (1.0%)	7050 (3.7%)	5830 (1.0%)	5860 (1.6%)	5430 (25.1%)
		7 Day	11,400 (0.2%)	12,220 (2.5%)	11,190 (3.1%)	11,070 (0.8%)	10,620 (2.3%)	11,760 (0.8%)	7440 (1.2%)	6360 (2.1%)	6580 (1.4%)	5300 (18.5%)
		28 Day	13,330 (4.1%)	13,930 (2.5%)	13,150 (4.1%)	11,930 (3.6%)	11,710 (2.3%)	13,800 (0.3%)	8430 (4.2%)	7570 (1.4%)	7510 (2.4%)	6240 (12.9%)
		56 Day	14,200 (4.9%)	14,000 (5.5%)	12,830 (3.8%)	12,300 (-)	12,150 (5.0%)	14,810 (6.4%)	9220 (2.3%)	7990 (0.6%)	8460 (3.4%)	5600 (29.8%)
	Tensile Strength in psi	1 Day	-	805	630	605	700	735	485	343	490	205
		28 Day	691	773	483	-	605	941	640	510	490	615

Table D.1.2. Concrete Batches with New Aggregate.

			R-	G3030-	G3030-8-32-							G3030-8-	
			8-36-1	8-36-1	1	2	3	4	5	6	7	30-1	28-1
Mix Proportions	Cement (PCY)		800	800	800	760	720	720	720	720	720	800	800
	Fly Ash (PCY)		-	-	-	-	-	80	-	40	-	-	-
	Silica Fume (PCY)		-	-	-	40	80	-	80	40	80	-	-
	Coarse Agg. (PCY)		1800	1800	1800	1800	1800	1800	1800	1800	1800	1800	1800
	Fine Agg. (PCY)		1144	1144	1228	1228	1228	1228	1228	1228	1228	1270	1312
	Water (PCY)		288	288	256	256	256	256	256	256	256	240	224
	Polyheed 997 (fl. oz/cwt)		-	-	-	-	-	-	-	-	-	-	-
	Glenium 3200HES (fl. oz/cwt)		-	-	-	-	-	-	-	-	-	-	-
	Glenium 3030NS (fl. oz/cwt)		-	7.5	12.5	14.2	15	18.5	15	13.5	19	20	22.5
	Rheobuild 1000 (fl. oz/cwt)		13	-	-	-	-	-	-	-	-	-	-
	Pozzolith 100 XR (fl. oz/cwt)		-	-	3	3	3	3	3	3	3	3	3
	w/cm		0.36	0.36	0.32	0.32	0.32	0.32	0.32	0.32	0.32	0.30	0.28
Fresh Properties	Air Temperature (°F)		85	81	79	-	77	70	72	73	75	75	82
	Relative Air Humidity (%)		67	67.5	48	-	58	-	78	66	60	62	51
	Concrete Temperature (°F)		91	89	92	94	90	85	84	83	86	88	95
	Slump (in.)		8.25	9	7.5	4.5	3.5	5	9.75	5.25	3.5	7.5	3.5
	Unit Weight (pcy)		147	147	150	150	150	150	152	151	151	151	152
	Air Content (%)		4.0	3.5	-	3.6	3.3	3.2	2.3	2.3	3.3	2.6	3.5
Hardened Properties	Compressive Strength in psi (C.O.V.)	1 Day	5180 (1.5%)	6220 (0.6%)	8040 (1.3%)	8400 (0.8%)	8010 (0.4%)	7890 (1.4%)	6950 (3.2%)	7810 (3.3%)	7780 (0.7%)	8580 (0.8%)	9430 (0.7%)
		3 Day	6470 (1.2%)	7510 (1.5%)	9430 (2.8%)	9750 (1.2%)	9750 (7.5%)	9550 (1.2%)	8940 (5.0%)	9260 (2.3%)	9860 (1.6%)	10,040 (1.5%)	10,860 (2.2%)
		7 Day	7400 (2.0%)	8320 (3.9%)	- (-)	11,000 (1.8%)	10,600 (2.1%)	11,180 (1.7%)	10,060 (1.5%)	10,130 (2.2%)	11,180 (1.7%)	11,320 (0.8%)	11,900 (2.2%)
		28 Day	8780 (2.6%)	9280 (2.1%)	11,230 (3.7%)	12,810 (3.0%)	13,170 (2.0%)	13,620 (3.6%)	11,930 (1.8%)	12,630 (2.3%)	13,890 (0.6%)	12,880 (2.0%)	13,170 (2.6%)
		56 Day	9120 (1.6%)	10,110 (-)	12,500 (0.7%)	14,040 (3.2%)	14,310 (0.6%)	14,940 (1.5%)	12,770 (5.4%)	13,820 (1.1%)	14,740 (3.2%)	13,840 (1.6%)	13,950 (2.8%)
	Tensile Strength in psi	1 Day	400	390	550	515	625	-	-	570	465	490	205
		28 Day	570	740	700	730	610	660	595	700	655	490	615

Table D.2.1. Trial Mortar Batches.

		NASP-										
		A	B	C	D	E	F	AA	BB	CC	DD	EE
Mix Proportions	Cement (PCY)	1069.6	1069.6	1072.4	1072.4	1072.4	1072.4	1072.4	1072.4	1072.4	1046.4	1117.2
	Fine Agg. (PCY)	2150.1	2150.1	2144.8	2144.8	2144.8	2144.8	2144.8	2131.8	2003.7	2092.7	2234.4
	Water (PCY)	481.3	481.3	482.6	482.6	482.6	482.6	482.6	514.7	536.2	523.2	445.9
	w/cem	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.48	0.50	0.50	0.40
Fresh Properties	Flow (%)	106.3	116.5	132.5	121.5	91.5	80.0	110.0	123.8	131.0	127.0	91.3
	Unit Weight (pcy)	138.0	139.0	137.6	138.3	139.0	138.8	136.3	135.8	136.1	136.6	-
	Air Content (%)	1.4	0.6	1.6	1.1	0.6	0.7	2.5	2.9	2.7	2.3	-
Hardened Properties	1 Day Strength in psi (C.O.V.)	4040 (7.3%)	3190 (21.2%)	4720 (3.4%)	4905 (3.8%)	4850 (3.9%)	- (-)	4775 (3.1%)	4395 (1.6%)	4405 (2.9%)	4195 (2.1%)	5720 (4.0%)
	7 Day Strength in psi (C.O.V.)	6795 (3.4%)	6795 (3.2%)	7165 (5.2%)	7160 (4.0%)	7300 (2.4%)	7160 (8.0%)	7295 (4.0%)	7305 (1.6%)	6860 (2.3%)	6770 (4.9%)	8085 (4.7%)
	28 Day Strength in psi (C.O.V.)	7755 (3.7%)	7500 (2.4%)	7040 (6.0%)	7980 (4.9%)	7820 (11.1%)	8215 (1.5%)	8100 (3.8%)	7670 (1.6%)	7565 (2.3%)	7695 (4.4%)	9245 (3.0%)

NOTE: Single letter mixes used original aggregate and double letter mixes used new aggregate

Table D.3.1. NASP Testing Mortar Batches.

		Strand "AA"			Strand "FF"		
		0.45	0.475	0.50	0.40	0.45	0.50
Mix Proportions	Cement (PCY)	1072.38	1055.25	1046.35	1117.20	1072.38	1046.35
	Fine Agg. (PCY)	2144.76	2110.50	2092.69	2234.40	2144.76	2092.69
	Water (PCY)	482.57	501.24	523.17	446.90	482.57	523.17
	w/cem	0.45	0.475	0.50	0.40	0.45	0.50
Fresh Properties	Flow (%)	98	121	121	80	117.5	125
	Unit Weight (pcy)	138.4	140.1	137.8	140.0	139.5	137.7
	Air Temp. (°F)	72	72	70	69	72	59
	Air Rel. Humidity (%)	61	72	53	30	35	36
	Concrete Temp. (°F)	82	82	72	74	78	68
Hardened Properties	Beginning of Testing Strength in psi (C.O.V.)	5050 (6.38%)	4610 (0.68%)	3590 (6.07%)	6185 (2.72%)	5010 (4.60%)	4045 (6.75%)
	End of Testing Strength in psi (C.O.V.)	5920 (2.99%)	5155 (3.97%)	3720 (7.95%)	6750 (2.55%)	5305 (4.06%)	4370 (4.19%)

APPENDIX E

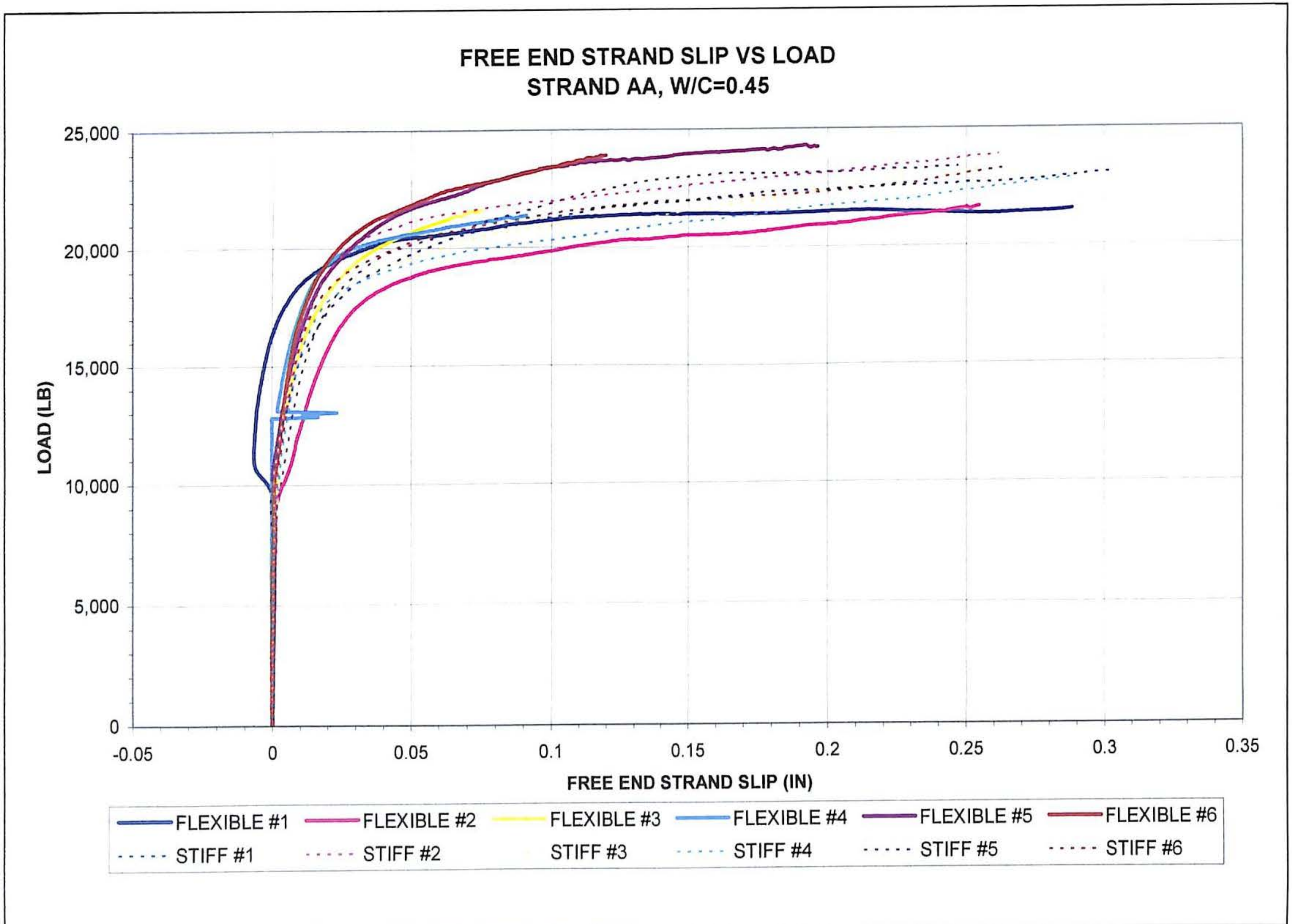


Figure E.1.1. NASP Test Results Strand "AA" with W/C = 0.45.

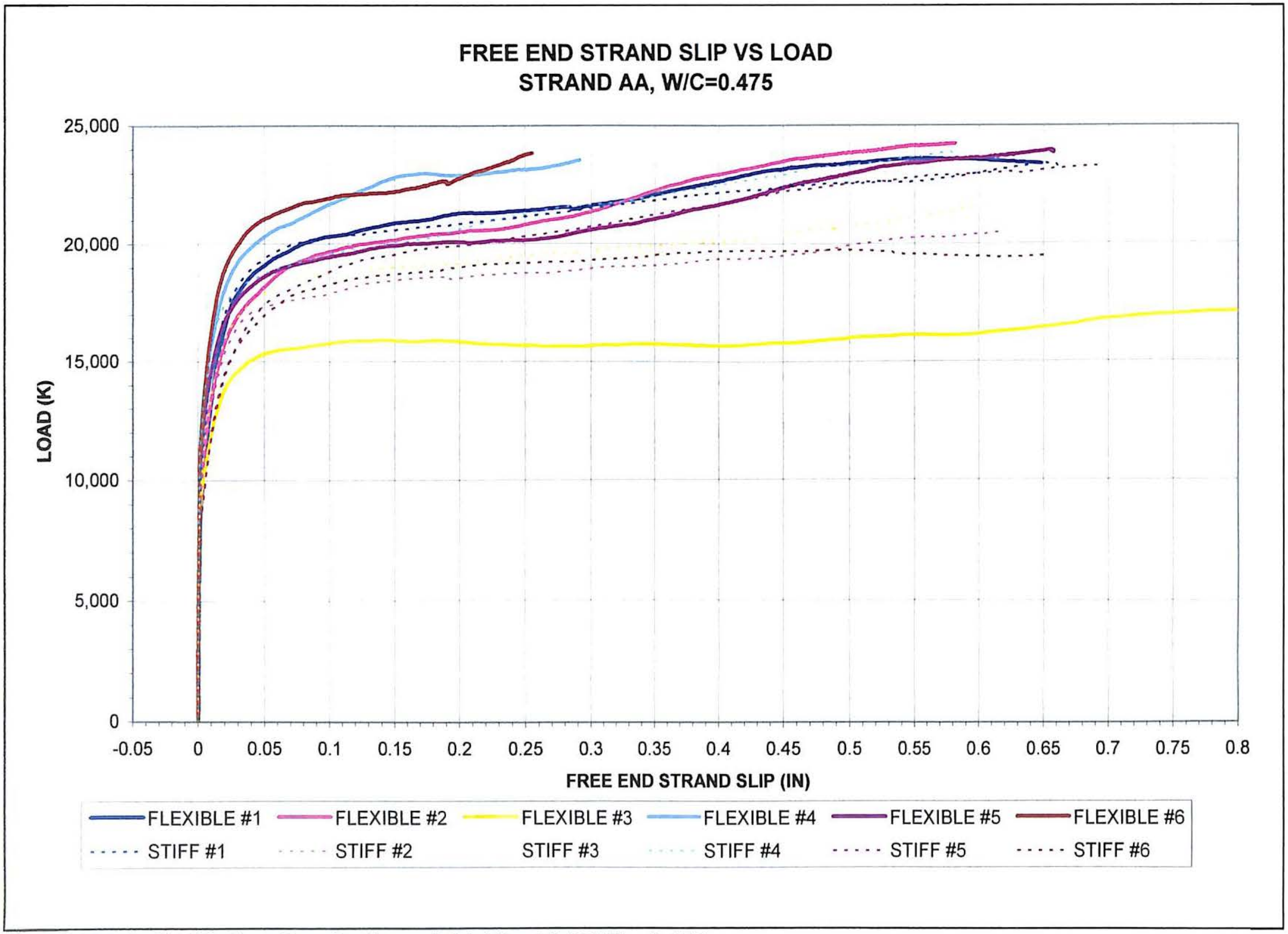


Figure E.1.2. NASP Test Results Strand "AA" with W/C = 0.475.

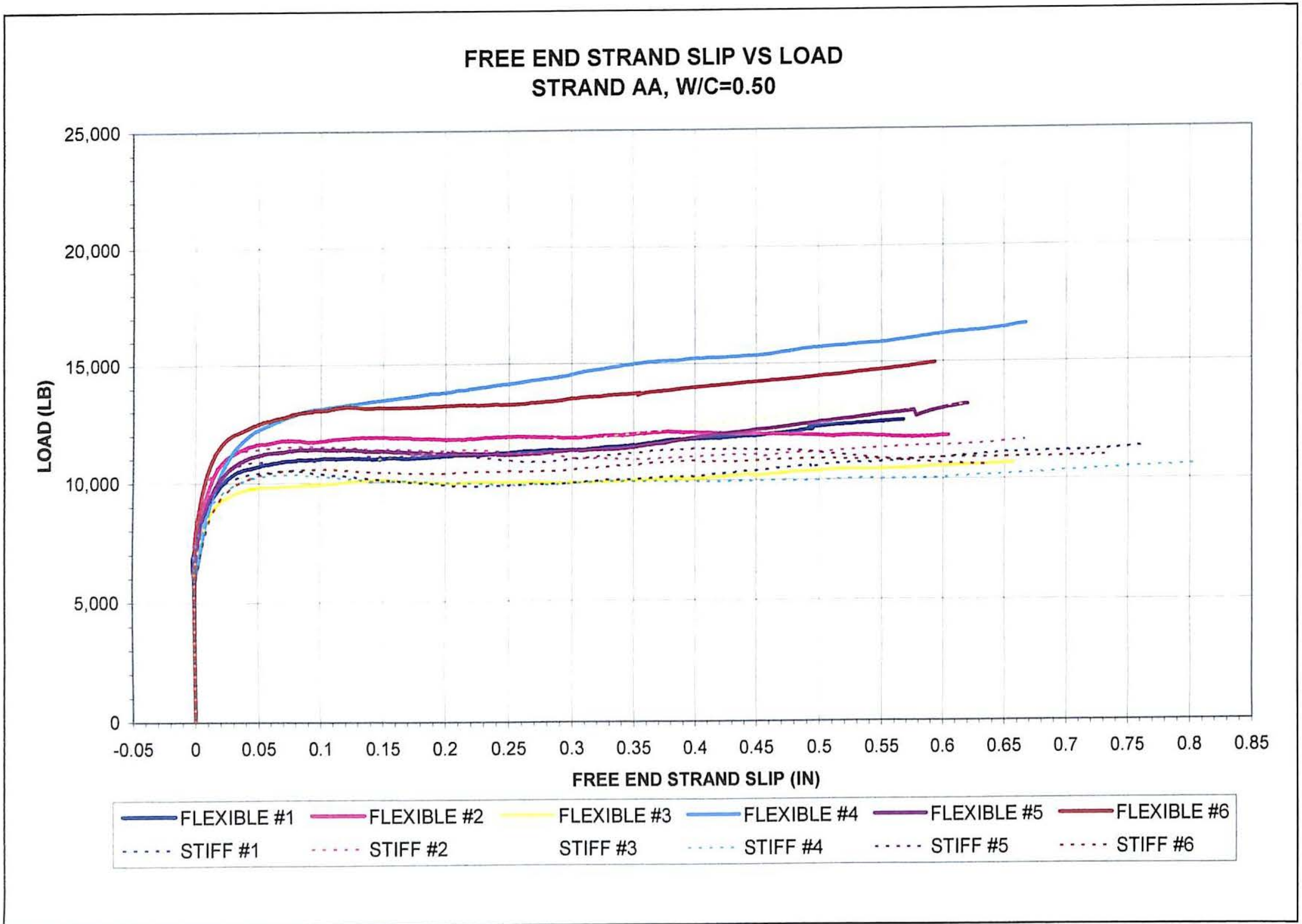


Figure E.1.3. NASP Test Results Strand "AA" with W/C = 0.50.

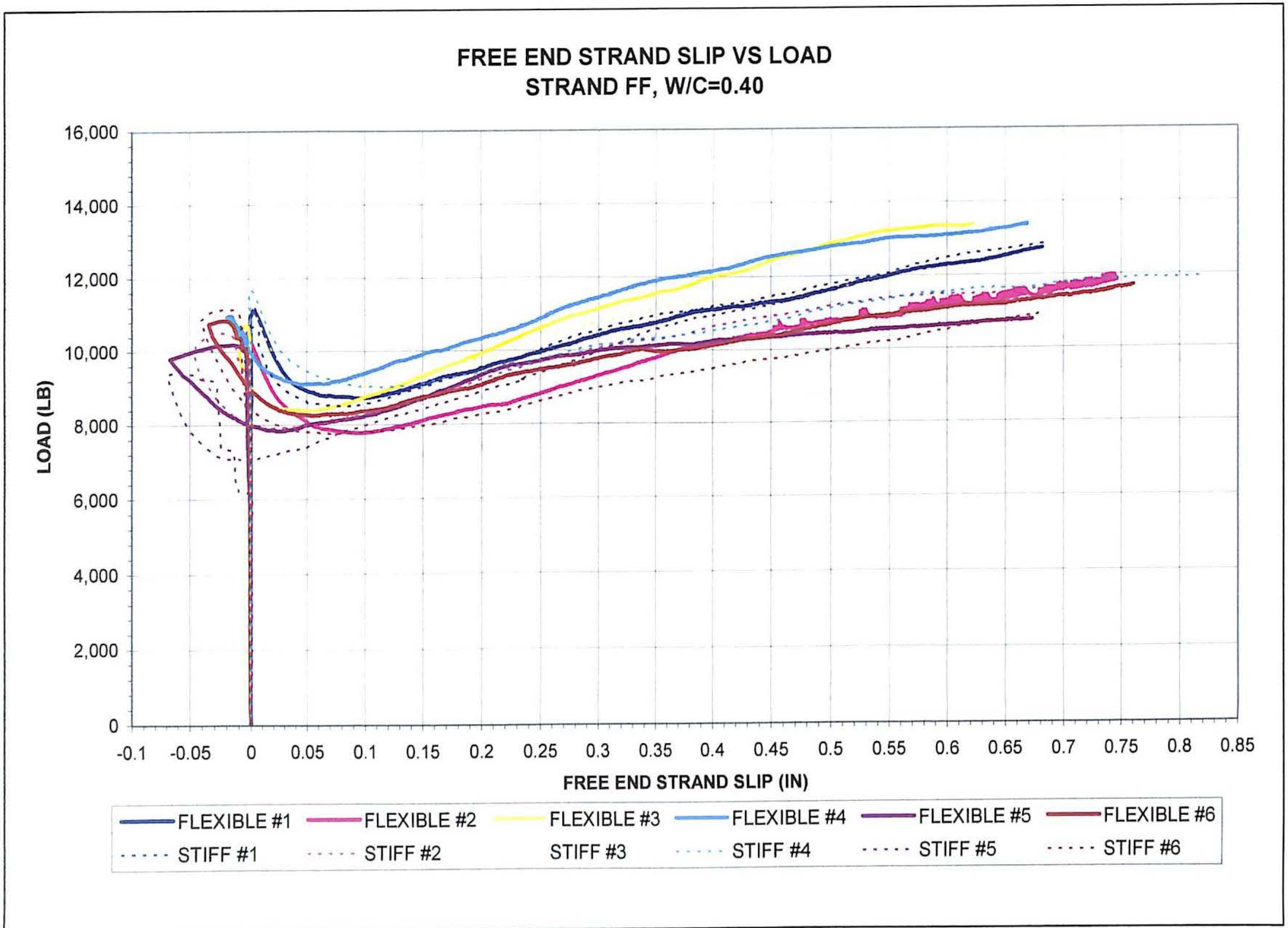


Figure E.1.4. NASP Test Results Strand "FF" with W/C = 0.40.

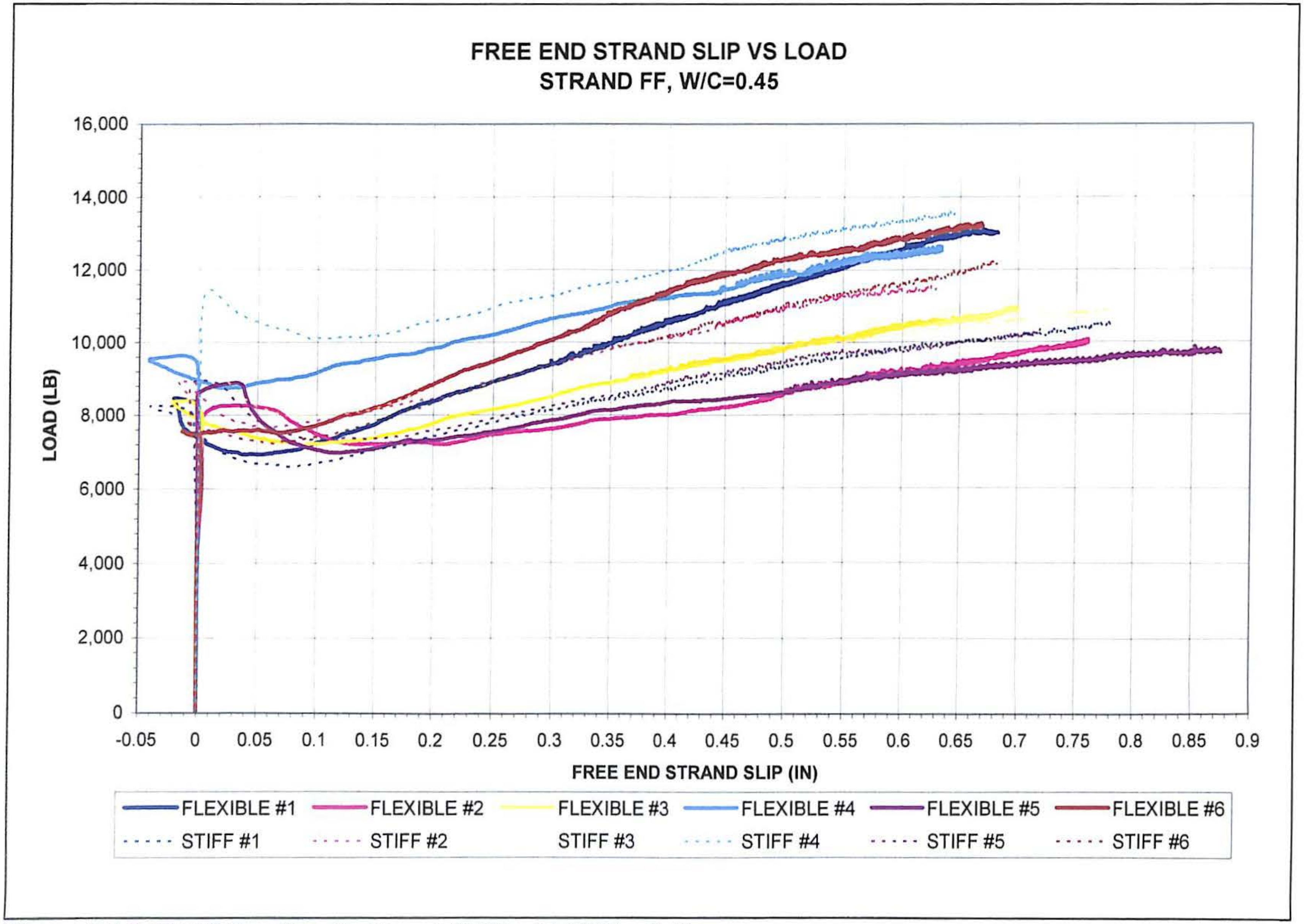


Figure E.1.5. NASP Test Results Strand "FF" with W/C = 0.45.

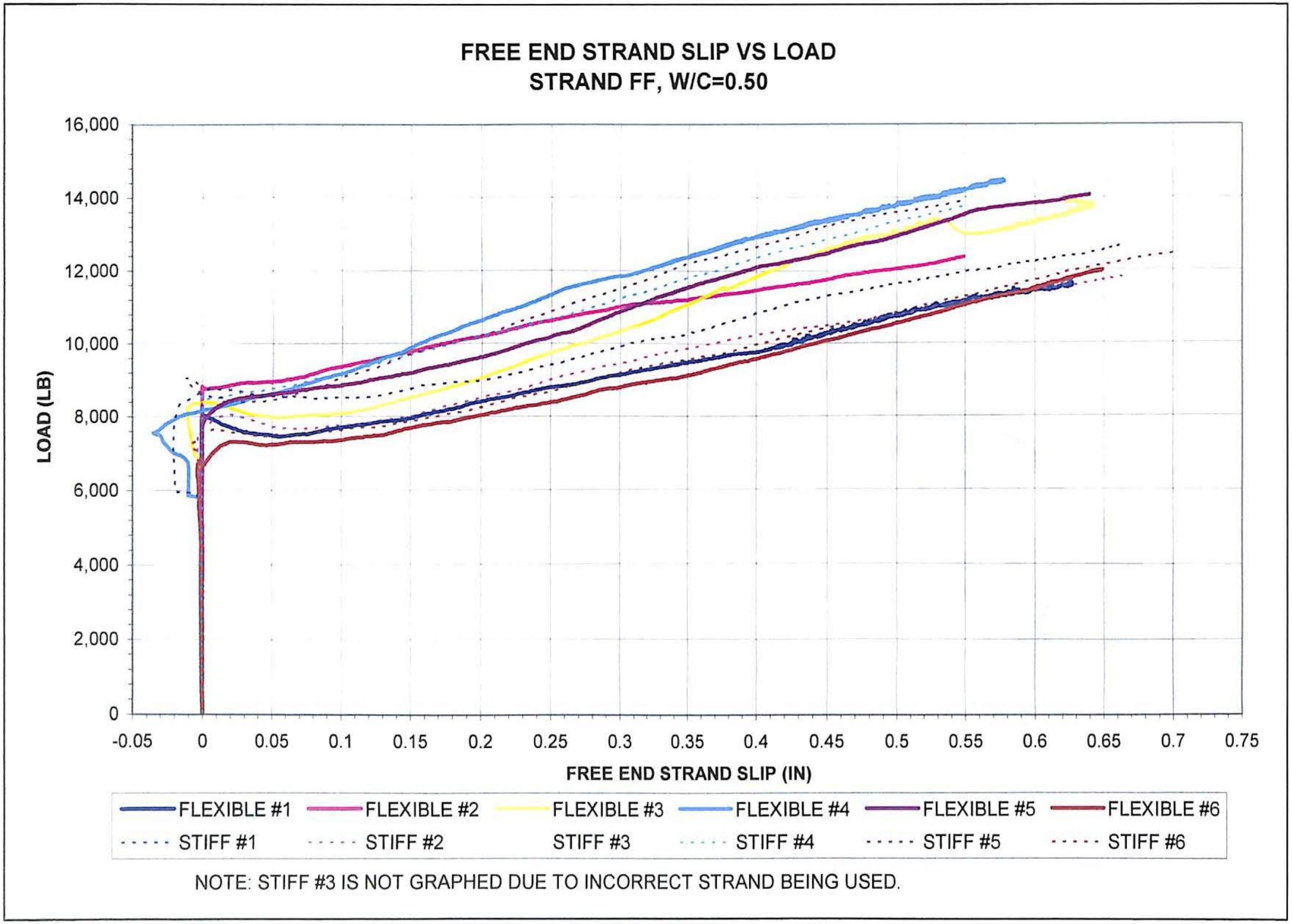


Figure E.1.6. NASP Test Results Strand "AA" with W/C = 0.50.

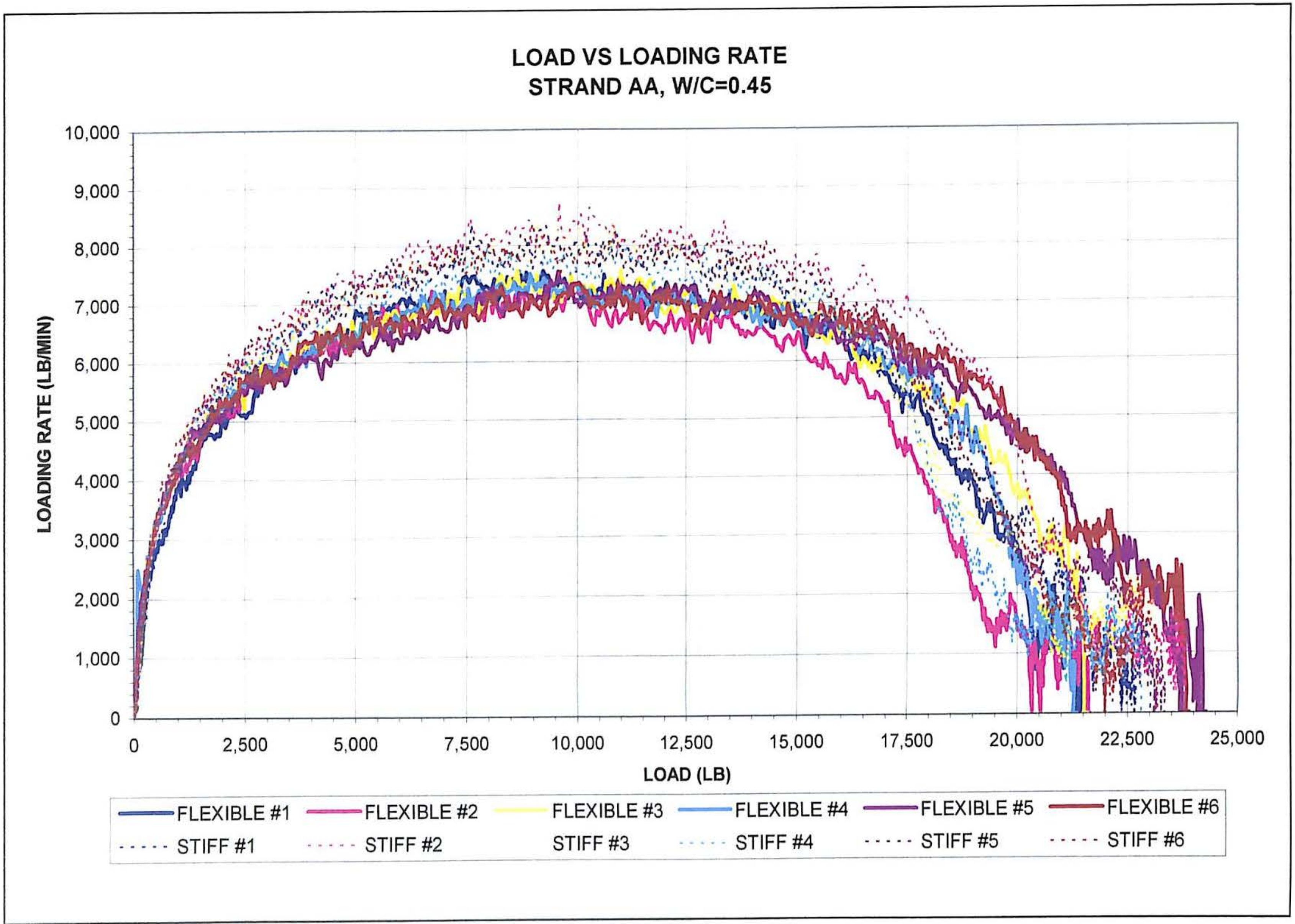


Figure E.2.1. NASP Loading Rate Strand "AA" with W/C = 0.45.

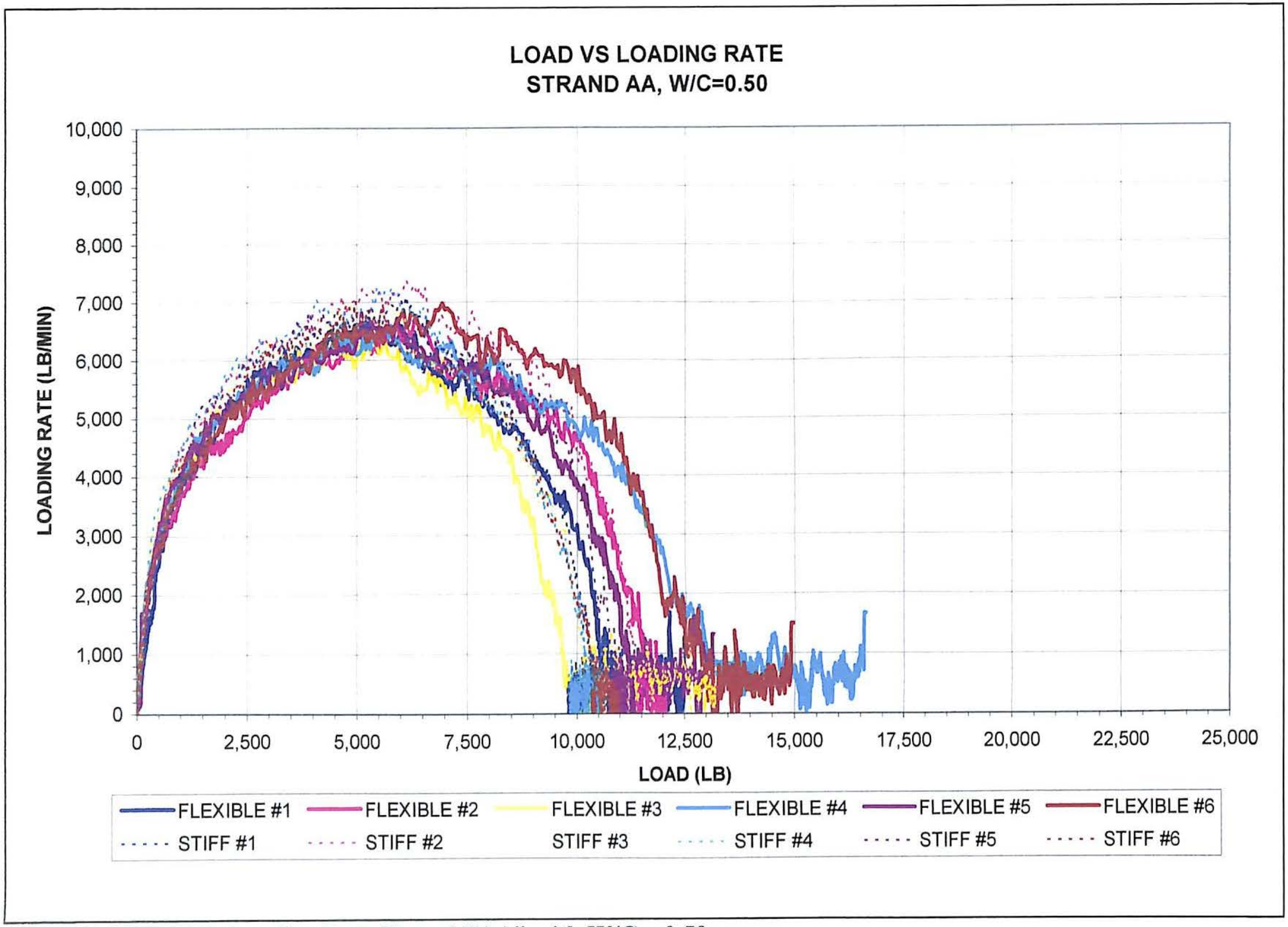


Figure E.2.3. NASP Loading Rate Strand "AA" with W/C = 0.50.

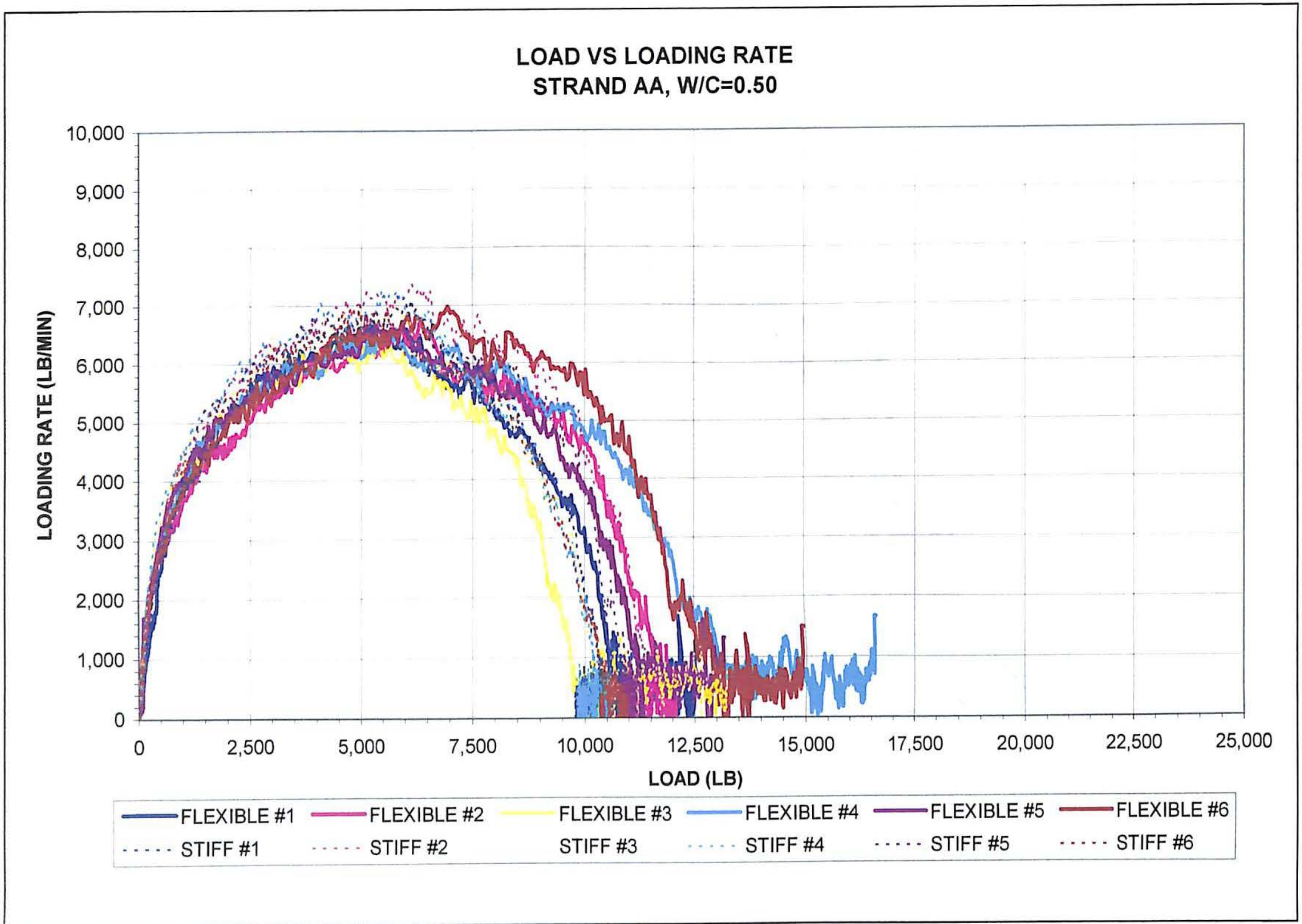


Figure E.2.3. NASP Loading Rate Strand "AA" with W/C = 0.50.

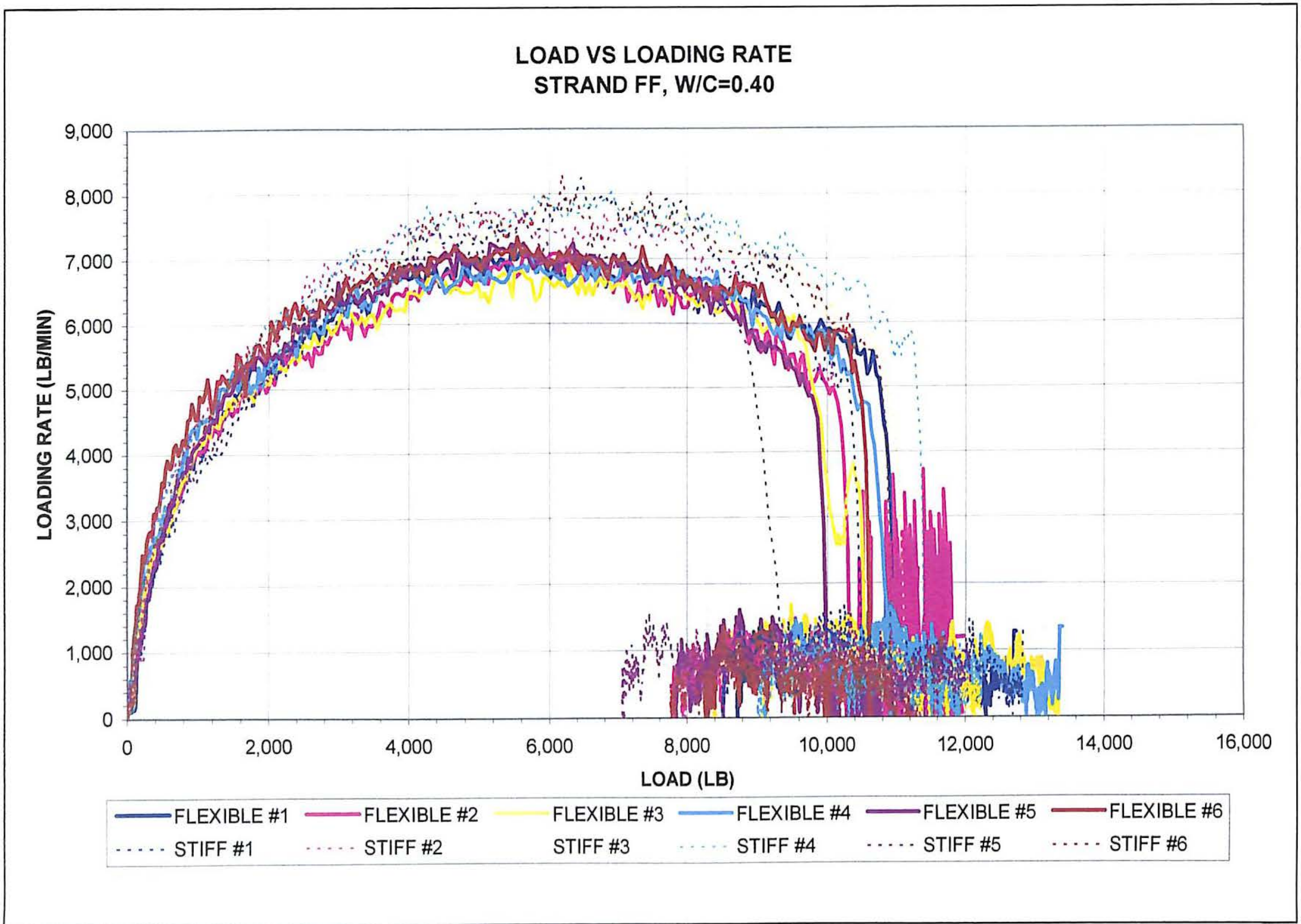


Figure E.2.4. NASP Loading Rate Strand "FF" with W/C = 0.40.

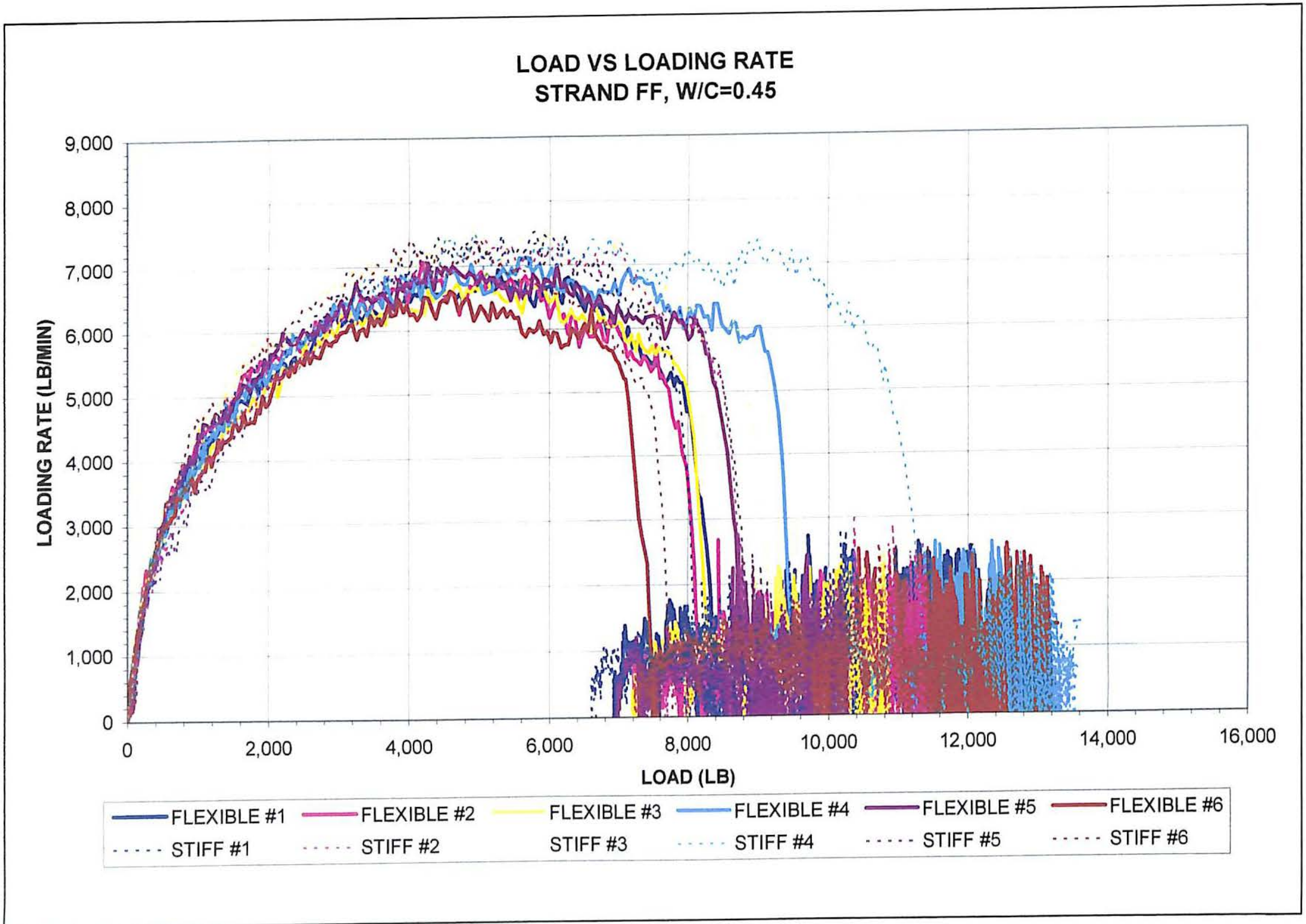


Figure E.2.5. NASP Loading Rate Strand "FF" with W/C = 0.45.

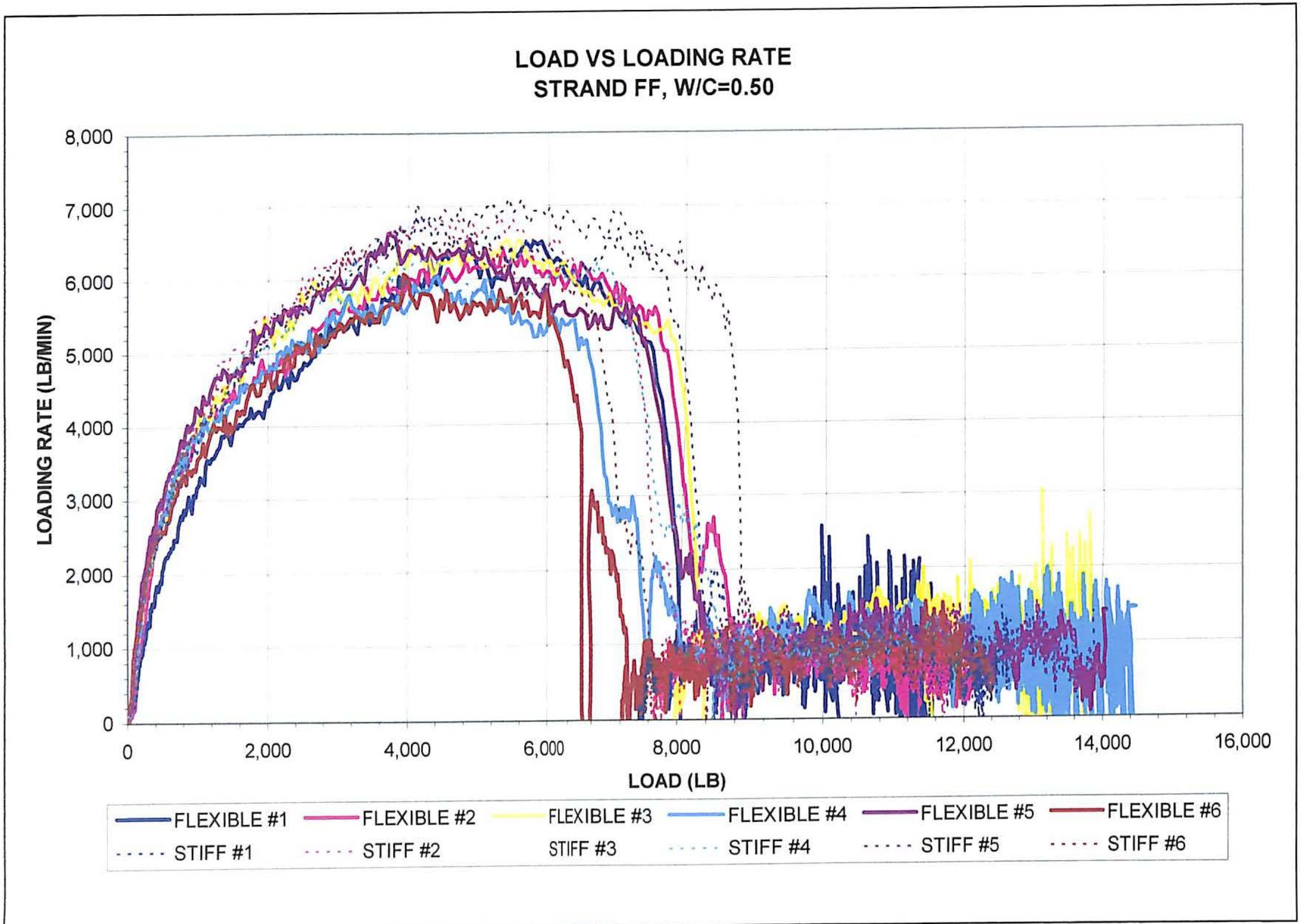


Figure E.2.6. NASP Loading Rate Strand "AA" with W/C = 0.50.

APPENDIX F

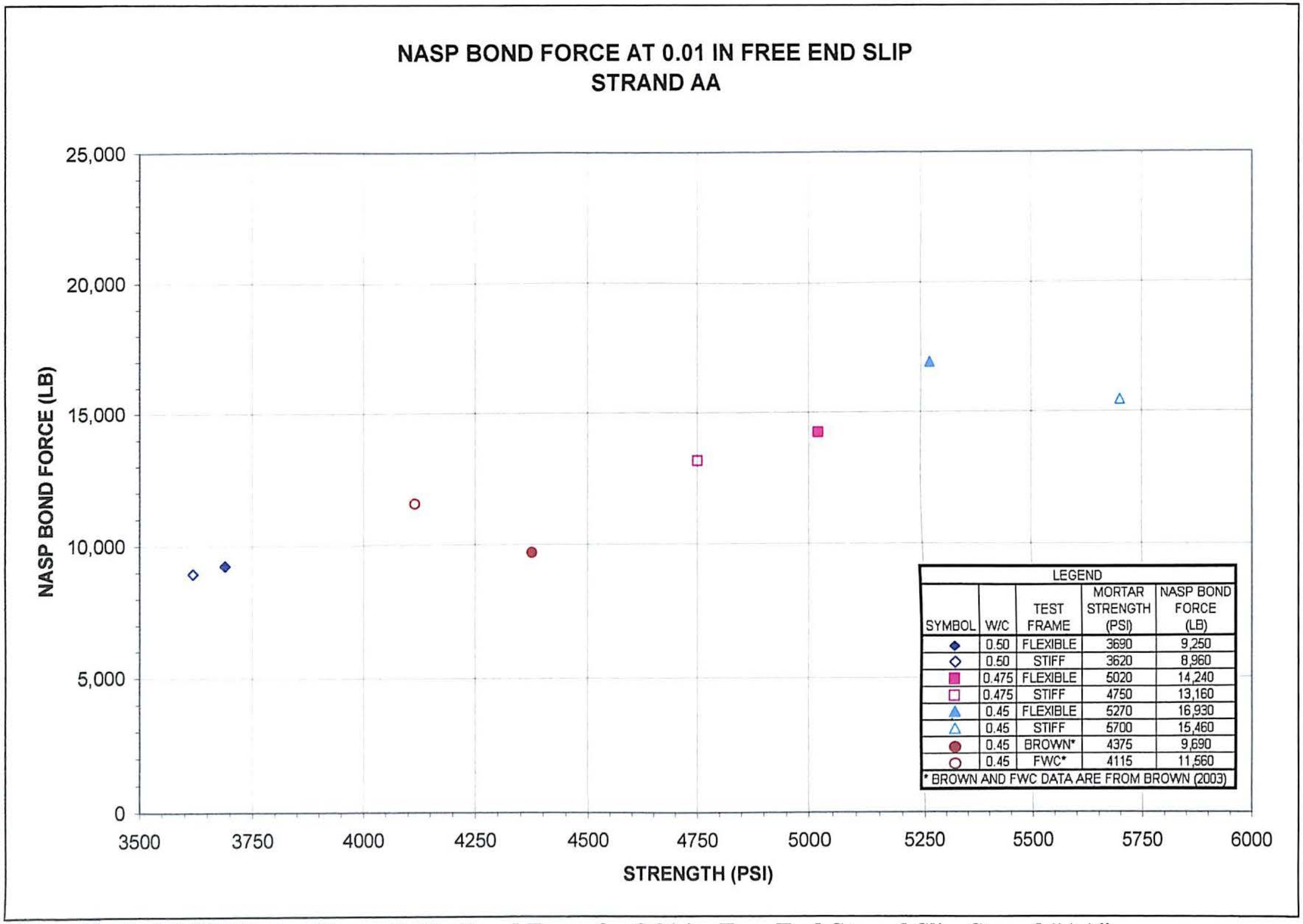


Figure F.1.1. NASP Mortar Strength Vs. Bond Force for 0.01 in. Free End Strand Slip, Strand "AA".

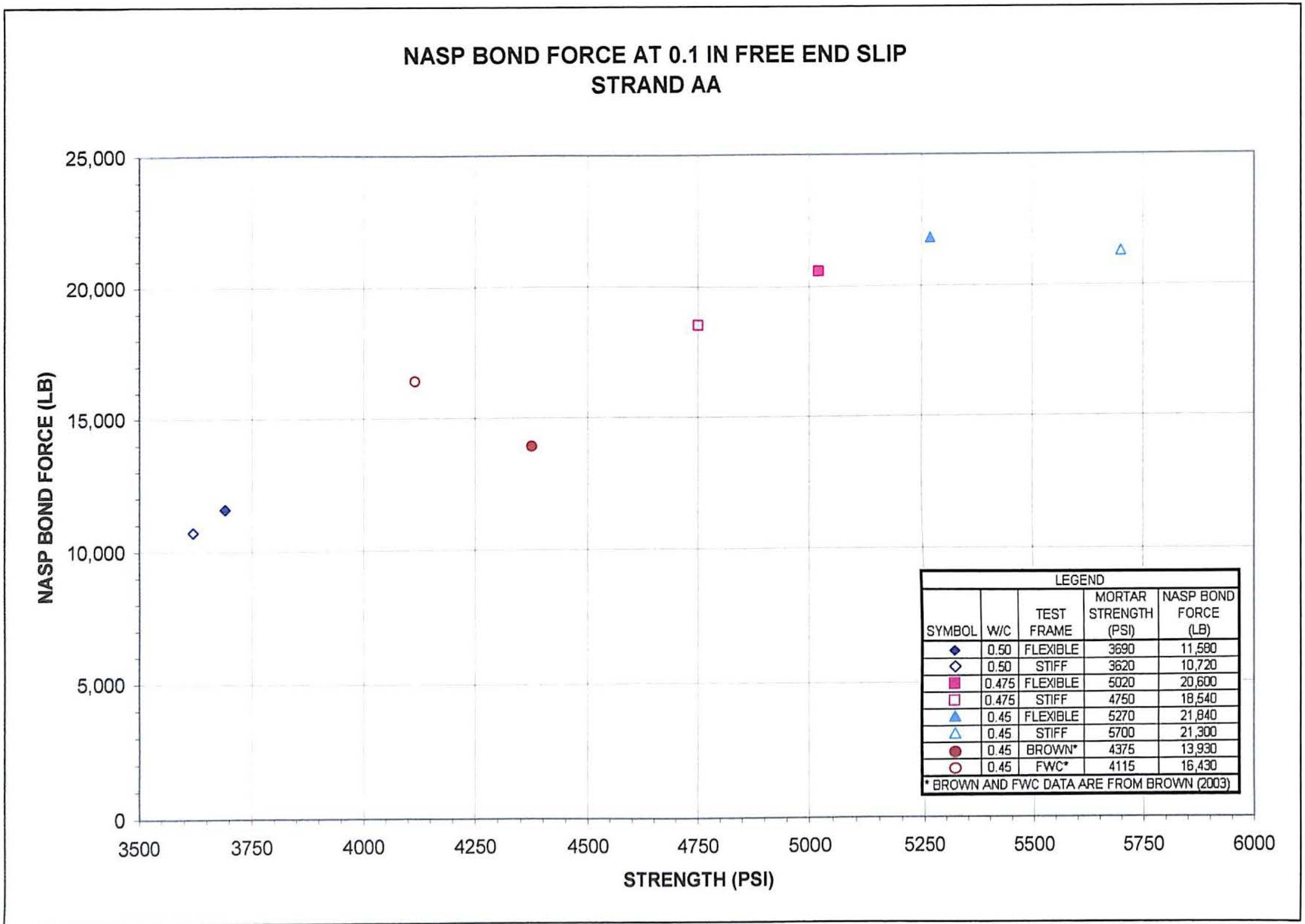


Figure F.1.2. NASP Mortar Strength Vs. Bond Force for 0.1 in. Free End Strand Slip, Strand "AA".

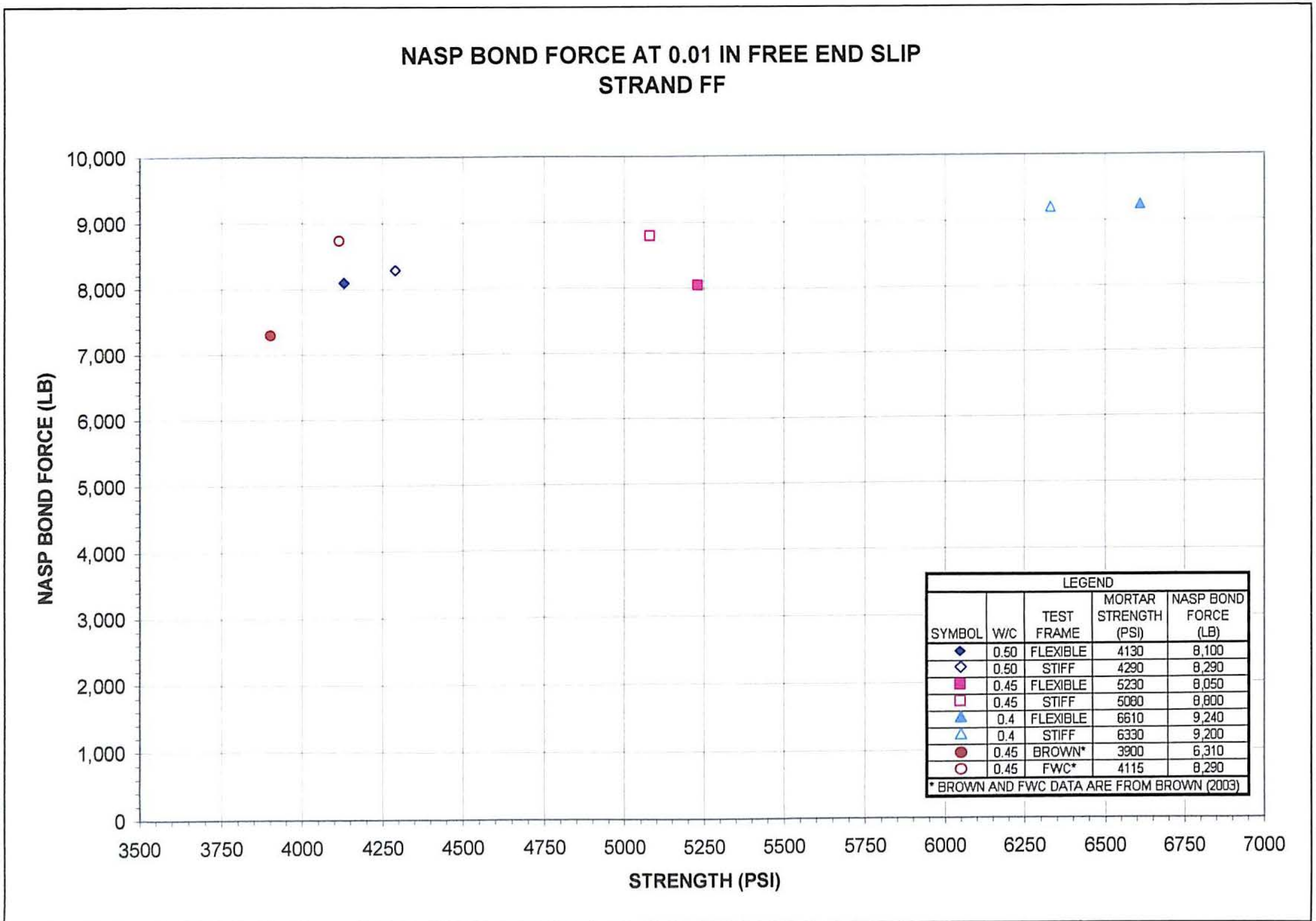


Figure F.1.3. NASP Mortar Strength Vs. Bond Force for 0.01 in. Free End Strand Slip, Strand "FF".

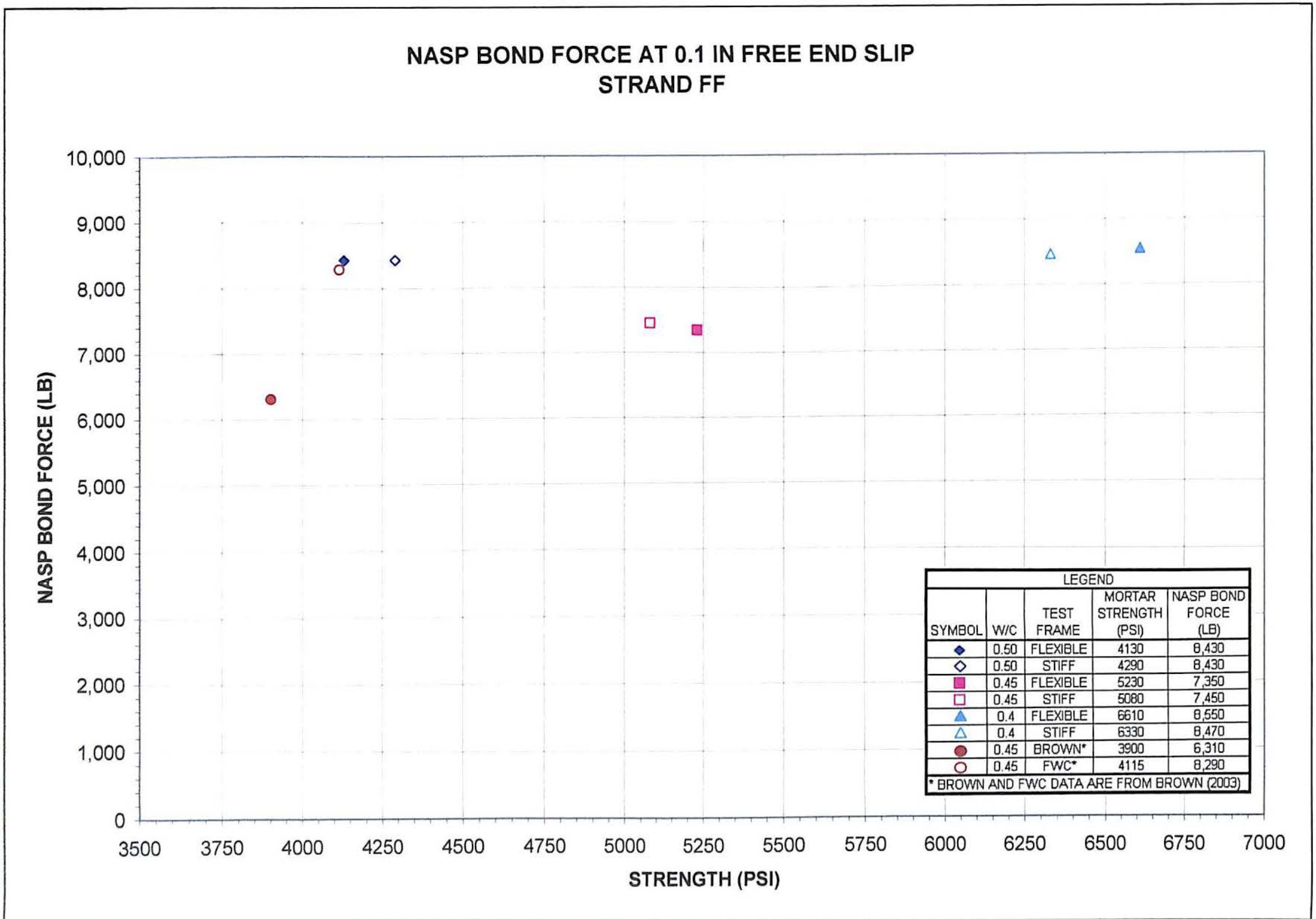
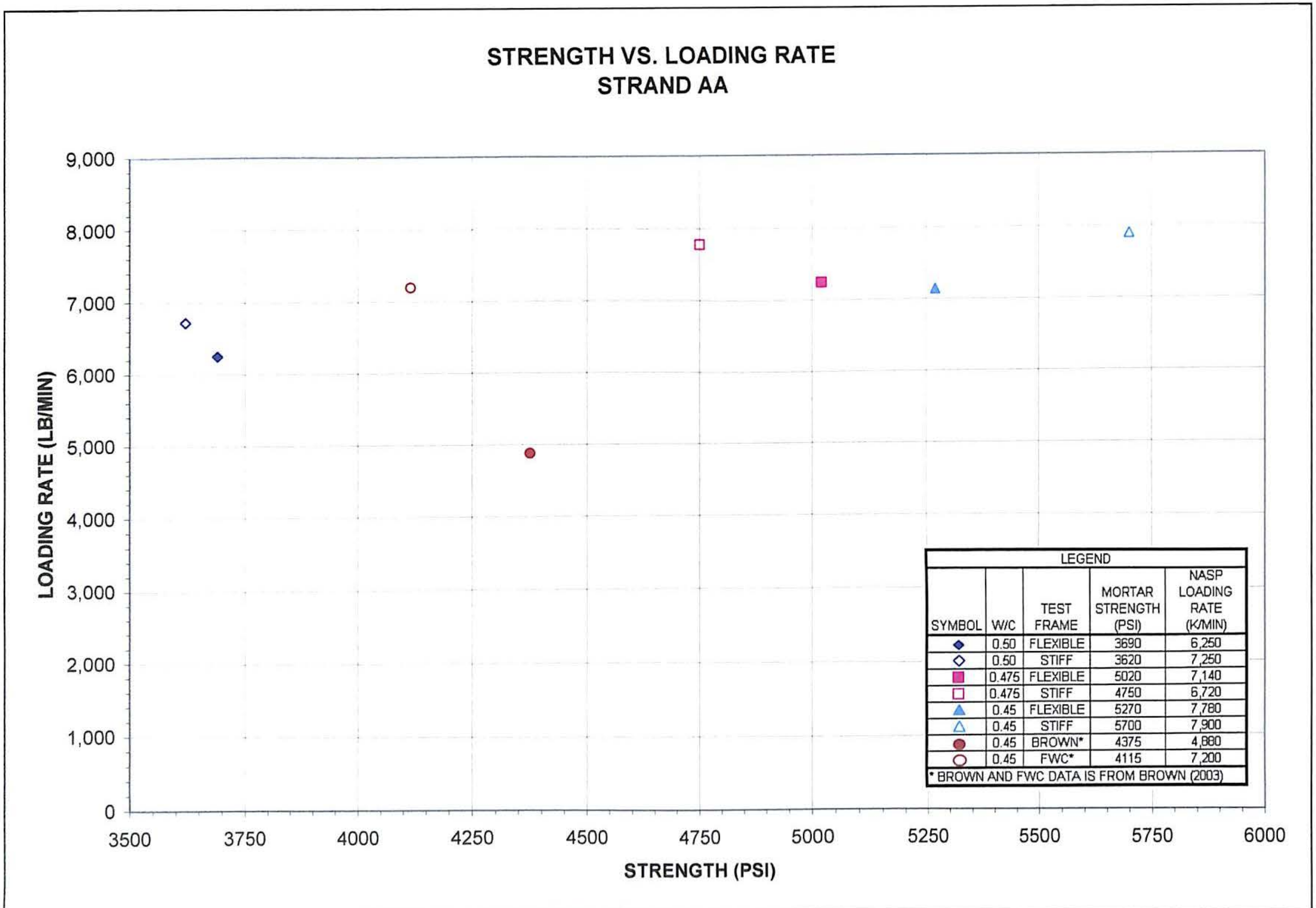


Figure F.1.4. NASP Mortar Strength Vs. Bond Force for 0.1 in. Free End Strand Slip, Strand "FF".



SYMBOL	W/C	TEST FRAME	MORTAR STRENGTH (PSI)	NASP LOADING RATE (K/MIN)
◆	0.50	FLEXIBLE	3690	6,250
◇	0.50	STIFF	3620	7,250
■	0.475	FLEXIBLE	5020	7,140
□	0.475	STIFF	4750	6,720
▲	0.45	FLEXIBLE	5270	7,780
△	0.45	STIFF	5700	7,900
●	0.45	BROWN*	4375	4,880
○	0.45	FWC*	4115	7,200

* BROWN AND FWC DATA IS FROM BROWN (2003)

Figure F.2.1. NASP Mortar Strength Vs. Loading Rate, Strand "AA".

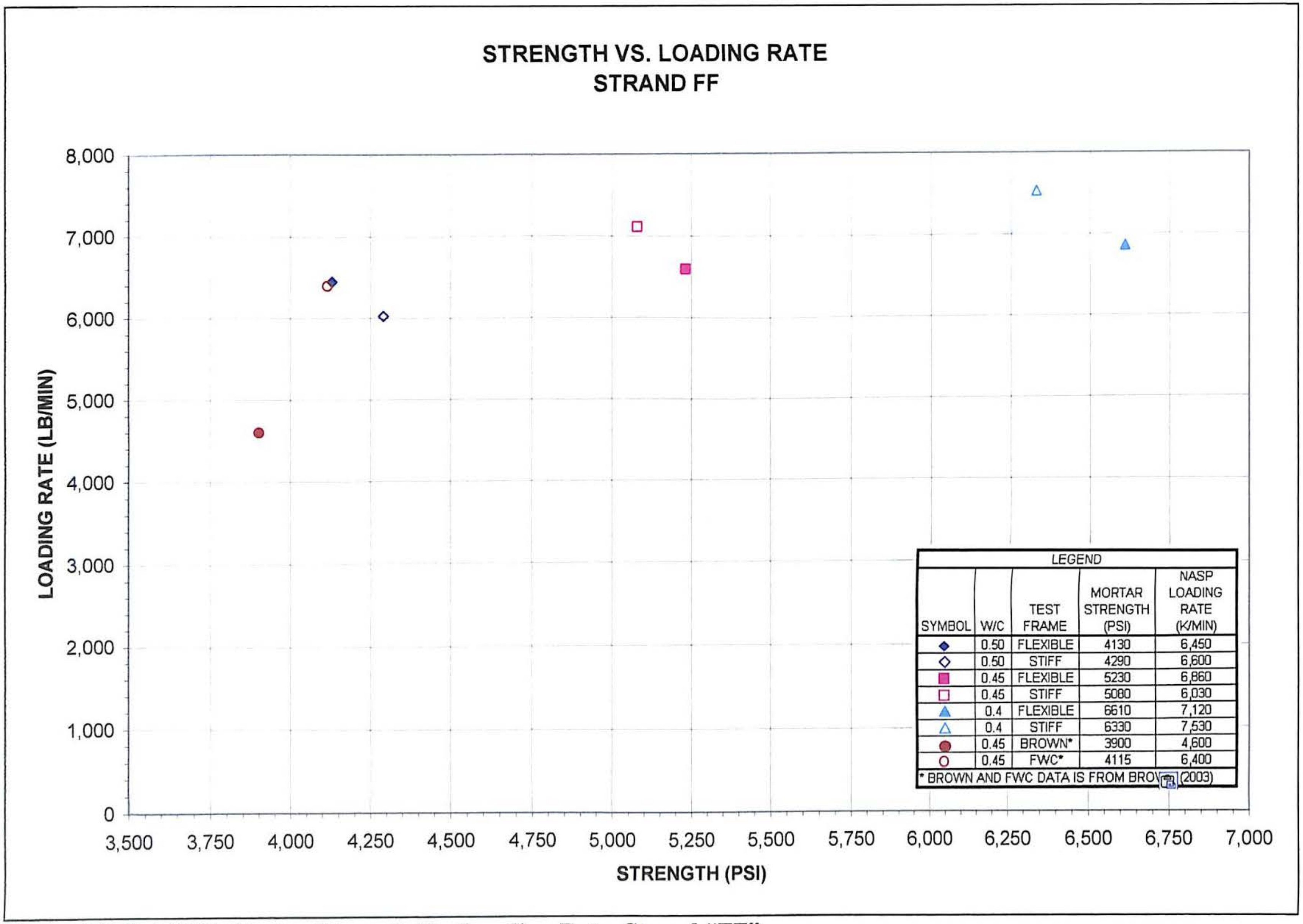


Figure F.2.2. NASP Mortar Strength Vs. Loading Rate, Strand "FF".

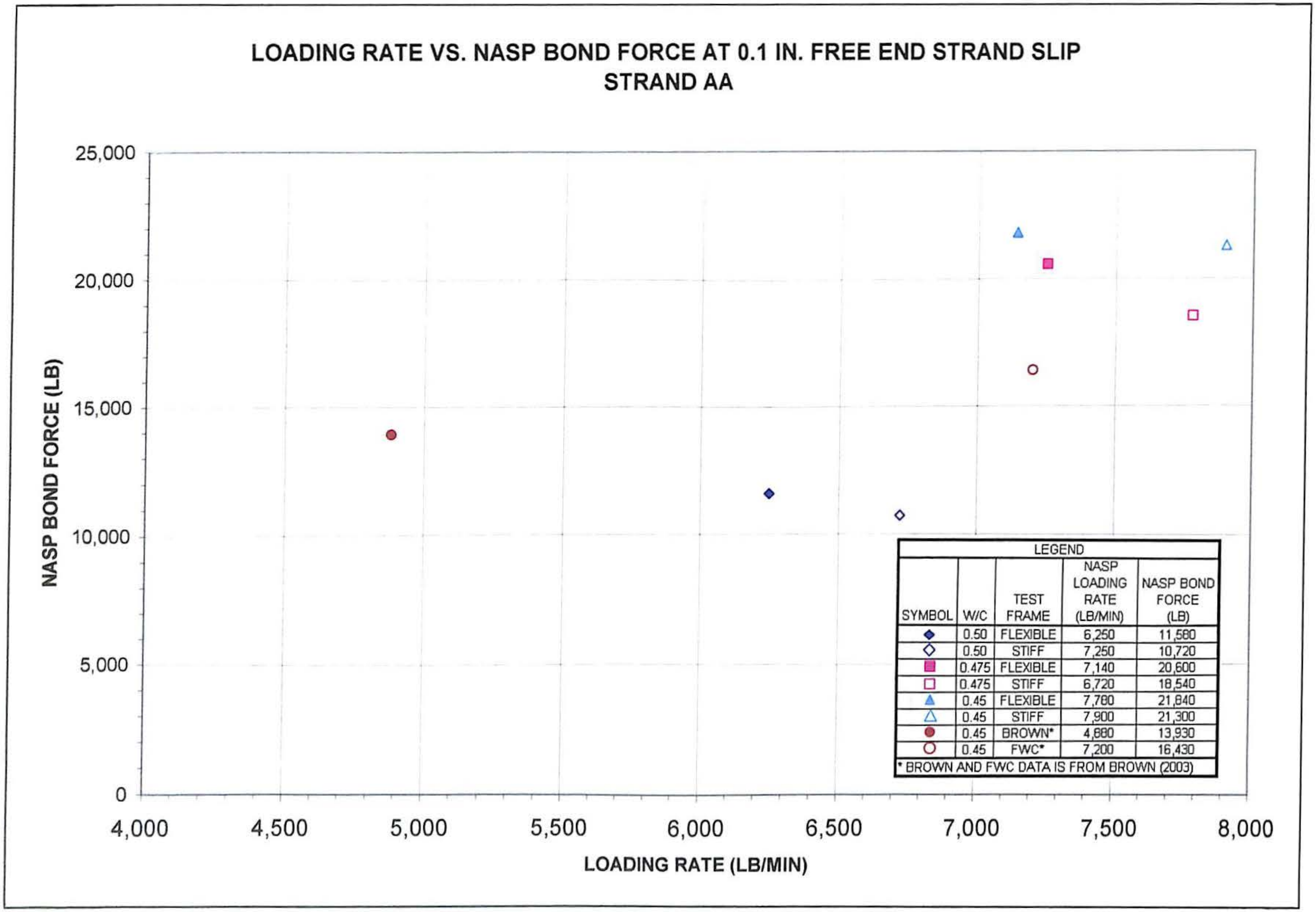


Figure F.3.1. NASP Loading Rate Vs. Bond Force at 0.1 in. of Free End Strand Slip, Strand "AA".

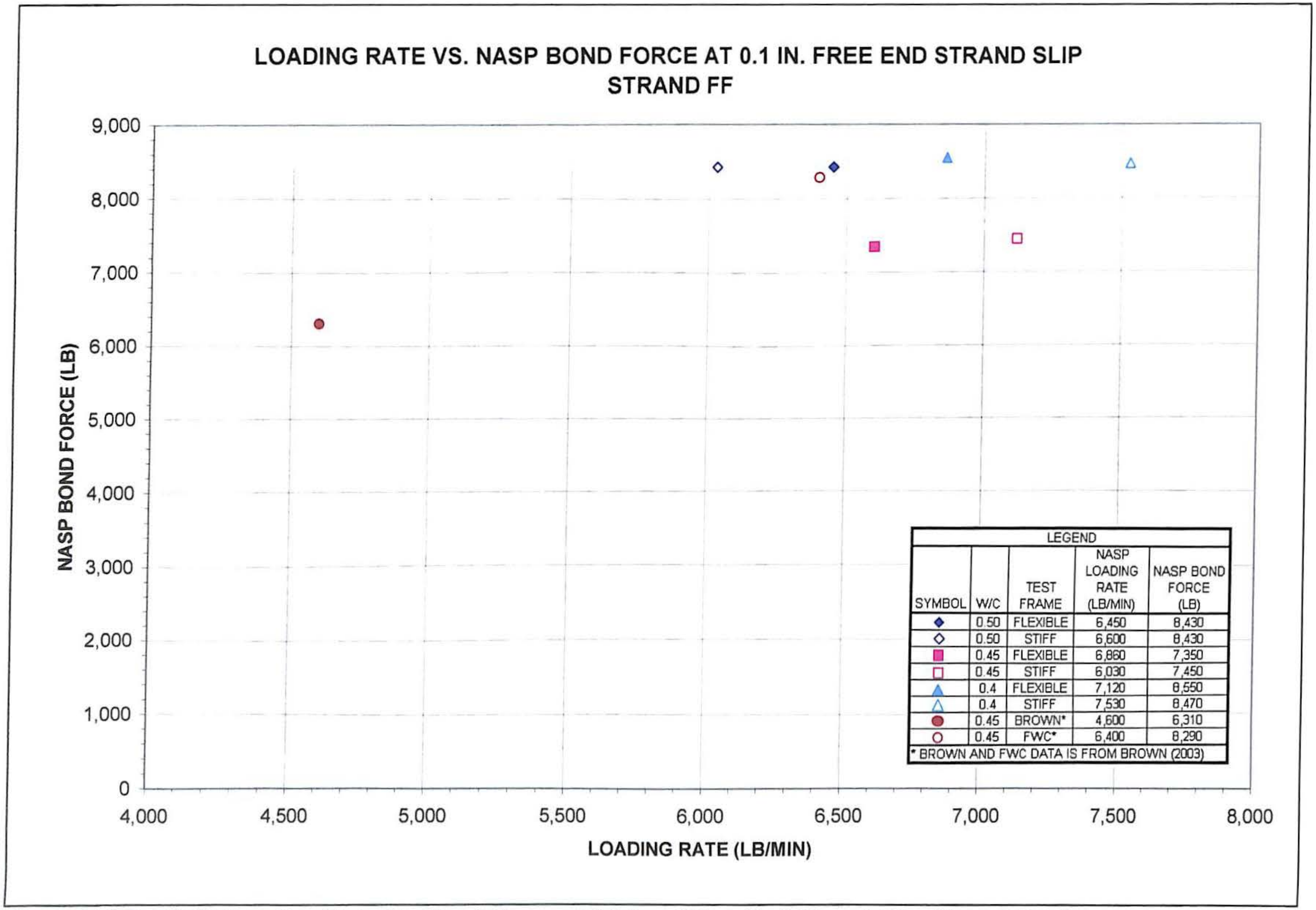


Figure F.3.2. NASP Loading Rate Vs. Bond Force at 0.1 in. of Free End Strand Slip, Strand "FF".

VITA (1)

Carisa Beth Grieve

Candidate for the Degree of

Master of Science

Thesis: INVESTIGATING THE EFFECT OF MORTAR STRENGTH AND LOADING RATE ON THE NASP BOND TEST

Major Field: Civil Engineering

Biographical:

Personal Data: Born in Fort Worth, Texas, On February 21, 1979, the daughter of Ronnie and Patricia Lemons.

Education: Graduated from James Martin High School, Arlington, Texas in May 1997; received Bachelor of Science degree in Civil Engineering from Colorado School of Mines, Golden, Colorado in December 2000. Completed the requirements for the Master of Science degree with a major in Civil Engineering at Oklahoma State University in May, 2004.

Experience: Employed by Freese and Nichols, Inc., Fort Worth, Texas as an intern during summers, 1997-1999; employed by Colorado School of Mines, Golden, Colorado as an undergraduate research assistant, 2000; employed by Frank Neal and Associates, Fort Worth, Texas as an engineer in training, 2001; employed by Beicker Engineering, Inc., San Antonio, Texas as an engineer in training, 2001-2002; employed by Envirotech Engineering and Consulting, Inc., Enid, Oklahoma as an engineer in training, 2002; employed by Oklahoma State University, Department of Civil Engineering as a graduate research assistant, 2002 to present.

Professional Memberships: American Society of Civil Engineers.