A New Development Length Equation for Pretensioned Strands in Bridge Beams and Piles

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FOREWORD

This report, *A New Development Length Equation for Pretensioned Strands in Bridge Beams and Piles*, presents the results of research conducted by the Federal Highway Administration (FHWA) at the Turner-Fairbank Highway Research Center in McLean, Virginia, with assistance from personnel representing Construction Technology Laboratories, Inc.

This research was conducted to investigate the accuracy of the current American Association of State Highway and Transportation Officials (AASHTO) equations for the transfer and development of pretensioned strands. Research results indicated that the AASHTO equation was unconservative for members constructed with normal-strength concrete. FHWA researchers formulated new transfer and development length equations based on FHWA's research results, and then correlated these equations with results from 16 other research studies to make sure that the equations would be representative of the total applicable data to-date. Guidelines for the use of these equations for beams and piles are also provided.

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16. Abstract

In 1988, the Federal Highway Administration (FHWA) issued a memorandum that outlawed the use of 15.2-mm- (0.6-in-) diameter strands, restricted the spacing of strands, and applied a multiplier to the American Association of State Highway and Transportation Officials' (AASHTO) development length equation. This memorandum initiated considerable research on the subject of bond of pretensioned strands in concrete. Forty-one research studies have been undertaken since 1988 to clarify the issues in the memorandum.

One of the studies initiated was a large research study conducted at FHWA's Structures Laboratory at the Turner-Fairbank Highway Research Center. Phase I of the study involved 50 rectangular prestressed concrete specimens, while Phase II involved 64 members: 32 AASHTO Type II prestressed concrete I-beams and 32 prestressed concrete sub-deck panels. Half of these members for both phases contained uncoated strands, while the other half contained epoxy-coated strands. Only the results from the members containing uncoated strands in rectangular specimens and beams were discussed in this report.

Results from the FHWA research were used to evaluate the current AASHTO equation for development length as well as a development length equation proposed by Dr. C. Dale Buckner of the Virginia Military Institute. Research results indicated that the AASHTO equation was unconservative for members constructed with normal-strength concrete and the Buckner equation was inconsistent for members constructed with normal-strength concrete. Both equations were conservative for members constructed with high-strength concrete.

Because of these results, it was determined that a new development length equation was needed -- an equation that could provide conservative predictions of transfer and development lengths for all concrete strengths, yet not be overly conservative for high-strength concretes. FHWA researchers formulated new transfer and development length equations based on FHWA's full-size beam research results, and then correlated these equations with results from other research studies to make sure that the equations would be representative of the total applicable data to date. Data from 16 studies were used in the correlation process. The transfer and development length equations based only on the FHWA beam results were refined based on the correlation process, and new transfer and development length equations were proposed. Guidelines for the use of these equations for beams and piles were provided.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: Volumes greater than 1000 l shall be shown in m^3.				
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celcius temperature	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

(Revised September 1993)

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CHAPTER 1: INTRODUCTION AND BACKGROUND

Prestressed concrete bridges are a main staple of America's bridges. Almost half of the new bridges constructed throughout America have prestressed concrete superstructures, indicating that prestressed concrete has become the material of choice for new bridges.[1] This represents a major development over the last half century, since the use of prestressed concrete for bridges began in the United States in 1949 with the initiation of construction of Philadelphia's Walnut Lane Bridge.[2] There are two types of prestressed concrete: pretensioned and post-tensioned. Pretensioned members are first constructed by tensioning the prestressing strands in a stressing bed. The concrete is then cast and cured, and once the concrete reaches a certain specified strength, the strands are released (or detensioned). The prestressing force in a prestressing strand is then transferred from the strand to the concrete by bond. This occurs in the end regions of pretensioned members.

Post-tensioned members are constructed by first placing and aligning empty ducts in a form. The concrete is then cast and cured, and the strands are placed inside the ducts. Once the concrete reaches a certain specified strength, the strands are tensioned and permanently anchored in place. The strands transfer their initial prestress force to the concrete through use of the permanent end anchorages.

WHAT IS DEVELOPMENT LENGTH?

Because the prestressing strands in pretensioned members transfer their force all the way up to the ultimate load of the member to the concrete by bond, a certain distance is needed to effectively allow bond to develop between the concrete and the strands. This distance, which is measured from the end of the member, is called the development length. Because the prestressing force is transferred from the strands to the concrete through the permanent end anchorages in a post-tensioned member—rather than through bond—there is no development length for a post-tensioned member.[3-5]

The development length is made up of two components: transfer length and flexural bond length. (See figure 1.) The transfer length is the distance from the end of the member needed to fully transfer the effective prestressing force by bond from the prestressing strand to the concrete. The effective prestressing force corresponds to the effective prestressing stress in the strand, f_{se}. As seen in figure 1, the transfer length corresponds to the distance needed from the end of the member to develop the effective prestressing stress, f_{se}, in the strand. The flexural bond length is the length needed beyond the transfer length to achieve bonding between the prestressing strand and the concrete to attain the stress in the strand at the ultimate load of the member, f^*_{su}. (See references 3 through 6.)

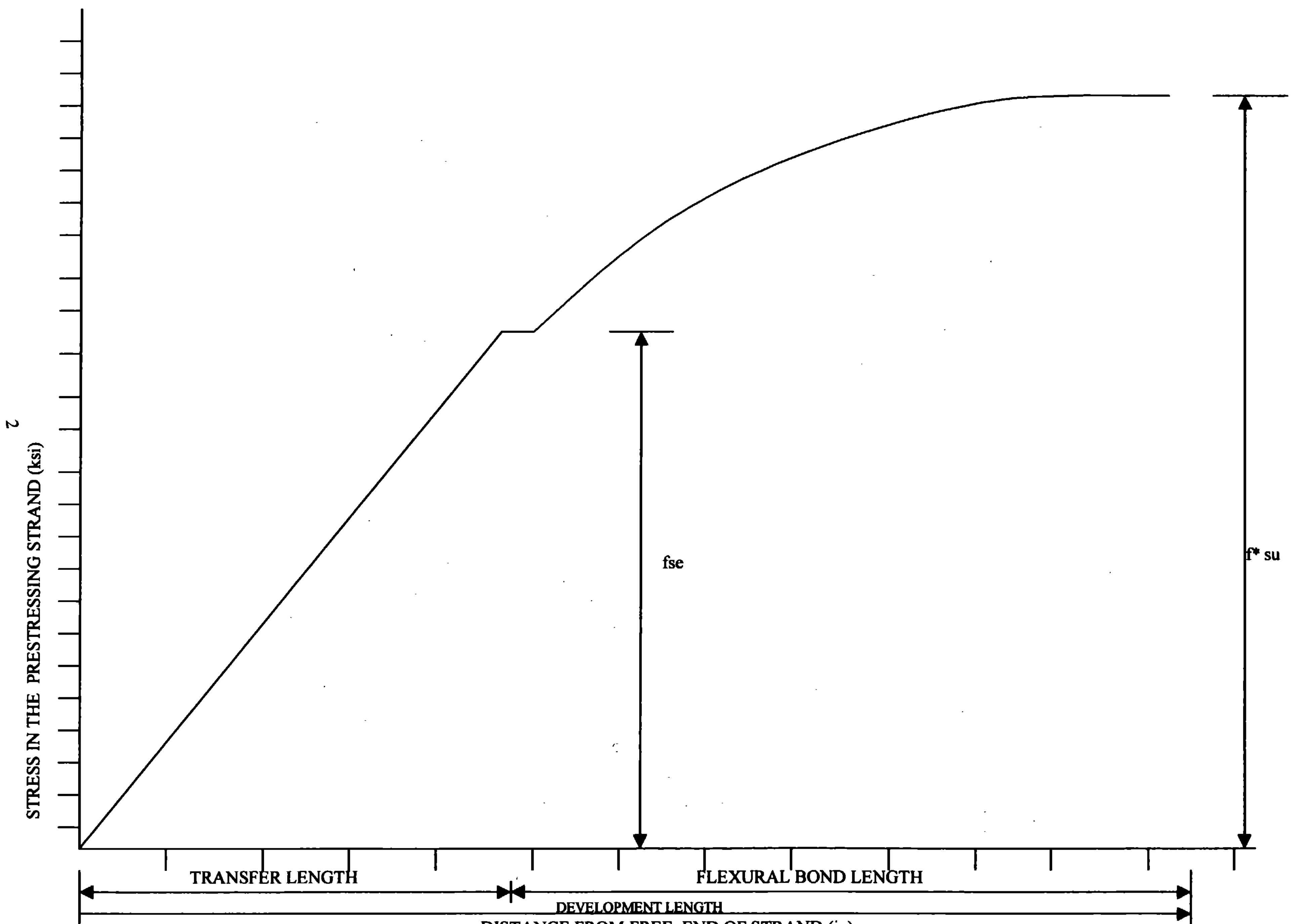

Figure 1. Development length of prestressing strand.

CURRENT AASHTO EQUATION

Currently, the equation for development length is the same in both the *American Association of State Highway and Transportation Officials (AASHTO) Bridge Specifications*[7] (AASHTO Specifications) and in the *American Concrete Institute's (ACI) Building Code Requirements for Structural Concrete*[6] (ACI building code). These organizations use different notations in their equations; however, for this paper, the current AASHTO notations will be used. See appendix A for a list of notations.

In Article 9.28.1 of the AASHTO Specifications,[7] the development length of a pretensioned concrete member is given by Equation 9-32 as:

$$(f^*_{su} - \frac{2}{3} f_{se})(D) \qquad (1)$$

where f^*_{su} and f_{se} are as previously defined, and D is the nominal diameter of the strand in inches. This article in the AASHTO Specifications is similar to Article 12.9.1 in the ACI building code.[6] Henceforth in this paper, Equation (1) will be referred to as the "AASHTO equation." This equation is sometimes confusing to designers because it does not separate the development length into its constituent parts, but rather merges the two parts together. The equation can be rewritten in terms of its constituent parts as:

$$L_d = \frac{f_{se} D}{3} + (f^*_{su} - f_{se})(D) \qquad (2)$$

where

$$L_d = development\ length$$

$$\frac{f_{se} D}{3} = transfer\ length,\ and$$

$$(f^{*}_{su} - f_{se})(D) = \text{flexural bond length.}$$

There is one other provision in the AASHTO Specifications related to development length, and that is found in Article 9.20.2.4.[7] This article states that the transfer length component of the development length for strand may be assumed to be equal to 50 times the nominal diameter of the strand. This AASHTO article is similar to the ACI building code Article 11.4.3.[6] Both expressions are used to help determine the component of a draped prestressing strand that can be used to resist shear.

FHWA 1988 MEMORANDUM

On October 26, 1988, the Federal Highway Administration (FHWA) issued a memorandum concerning the use of prestressing strands in pretensioned applications for prestressed concrete bridges. The memorandum:

- Disallowed the use of 15.2-mm- (0.6-in-) diameter strands in pretensioned applications.
- Restricted the minimum center-to-center strand spacing to four times the nominal diameter of the strand.
- Increased the required development length for fully-bonded and debonded strands by 1.6 and 2.0 times AASHTO Equation 9-32, respectively.[14]

The FHWA memorandum indicated that its restrictions were adopted only as an interim measure, until research results indicated otherwise and AASHTO adopted the results.

FHWA issued this memorandum to address two technical incompatibilities. The first incompatibility was between the strands used in the research studies leading to the AASHTO equation and the strands now in use. The second incompatibility was that the development length values resulting from the AASHTO equation did not agree with research results published at that time.[3]

The first incompatibility was between the strands used to create the AASHTO equation and the strands now in use. The research data on which the AASHTO equation was based was for stress-relieved strands with ultimate strengths of 1720 MPa (250 ksi). The stress in these strands immediately after prestress transfer could not exceed 70 percent of the ultimate stress of the strands. In contrast, the current practice is to use low-relaxation strands with ultimate strengths of 1860 MPa (270 ksi). Since 1986, the stress in these strands can go up to (but cannot exceed) 75 percent of the ultimate stress of the strands. Thus, current strands are stronger and can accommodate a higher overall stressing force than the strands used to create the AASHTO equation. Also, the prestressing force is now allowed to attain a higher percentage of the ultimate strength of the strands than was allowed for the data used to create the AASHTO equation.

The second incompatibility involved research results. The research was conducted in the 1980's by North Carolina State University.[4] It indicated that the development lengths of uncoated strands with ultimate strengths of 1860 MPa (270 ksi) were greater than that predicted by the AASHTO equation.

HISTORY OF THE AASHTO EQUATION

Due to the questions concerning the validity of the AASHTO equation, it was decided by FHWA to sponsor a study that would determine how the AASHTO equation came to be. Construction Technology Laboratories, Inc. (CTL) conducted this study for FHWA. By consulting past versions of AASHTO and ACI documents, as well as published and unpublished ACI Committee 423 documents and individuals from this committee, they found that the AASHTO equation reflected average values rather than conservative ones.[8]

Their study found that the AASHTO equation was first used in the 1963 ACI Building Code[9] and was adopted for use in the AASHTO Specifications in 1973.[10] The equation was based on two research studies. These studies, conducted during the 1950's and 1960's by Hanson and Kaar[11] and by Kaar, Lafraugh, and Mass,[12] used stress-relieved strands with an ultimate strength of 1720 MPa (250 ksi) and included little work on 15.2-mm- (0.6-in-) diameter strands. In both of these studies, the stress in the strands immediately after prestress transfer could not exceed 70 percent of the ultimate stress in the steel strand. Even though the AASHTO equation was based on these research studies, it does not reflect the values for development length that Kaar and Hanson propose. CTL researchers Tabatabai and Dickson further explain:

> "The ACI 318R-63 Commentary also states that the transfer and flexural bond equations are based on published papers by Kaar et al. and Hanson and Kaar ... However, those two papers do not specifically propose the equations represented in the code. In fact, Hanson and Kaar recommended embedment lengths far more conservative than AASHTO Equation 9-32 to prevent general bond slip."[8]

The portion of the AASHTO equation relating to transfer length, $f_{se}D/3$, is based on an assumed average value of bond stress. Because of this, the above expression for transfer length represents an average value for transfer length, and not a conservative value. The commentary for the 1963 ACI building code confirms this by stating that "the value of $f_{se}D/3$ for transfer length is an average value based on data reported by Kaar et al. ..."[13]

Similarly, the expression for flexural bond length contained within the AASHTO equation, $(f^*_{su} - f_{se})D$, also represents average values for flexural bond lengths and not conservative values. Tabatabai and Dickson state:

> "The flexural bond length relationship $((f^*_{su} - f_{se})D)$ is the equation of a line drawn through data points on a graph of $(f^*_{su} - f_{se})$ versus $((L - L_t)/D)$. In the view of the ACI Committee 423 [joint ASCE-ACI Committee 423 on Prestressed Concrete], the proposed line represented a reasonable mean for the data points without being unreasonable for long bonded lengths."[8]

Because both components of the AASHTO development length equation reflect reasonable means and not conservative values, then the AASHTO equation itself can only be expected to be an average equation and not a conservative equation.

RESEARCH STUDIES

As a result of the FHWA memorandum, 41 research studies have been undertaken since 1988 to clarify the issues in the memorandum on prestressing strand transfer and development lengths. Some of these studies have been completed and some are still underway. If these studies are added to the bulk of research undertaken prior to the FHWA memorandum, then a total of more than 60 studies have been undertaken on this topic. For some of these studies, the transfer and development length experimentation was the sole objective of the study; for others, it was an important constituent of a broader study objective.

FHWA has undertaken its own study on bond of prestressing strand. Entitled "Investigation of Development Length of Uncoated and Epoxy-Coated Prestressing Strand," the study began in the spring of 1990 and consisted of two phases. Phase I involved rectangular prestressed concrete specimens ranging in size from 102 x 102 x 3658 mm (4 in x 4 in x 12 ft) to 356 x 356 x 8534 mm (14 in x 14 in x 28 ft). Three strand sizes were used in the following diameters: 9.5 mm, 12.7 mm, and 15.2 mm (3/8 in, 0.5 in, and 0.6 in), and the specimens contained either one or four strands. Some specimens contained only uncoated strands, while others included only epoxy-coated strands. The rectangular specimens were fabricated and tested at FHWA's Structures Laboratory, part of the Turner-Fairbank Highway Research Center in McLean, Virginia.

The results for the specimens containing uncoated strands indicated that the AASHTO expressions for transfer and development length were not conservative for specimens containing multiple uncoated strands of all diameters. The details of this phase of the research are provided in other publications. (See references 3, 15, and 16.)

Prior to 1993, research results from many of the other studies were being issued and these contained numerous conflicting recommendations. In an effort to resolve these conflicting recommendations, FHWA initiated in January 1993 an impartial review of the research results available up to that time. Dr. C. Dale Buckner, on sabbatical from the Virginia Military Institute, performed this review during 1993, and issued a summary report.[17] Included within this summary report was the proposed new development length equation shown below:

$$L_d = \frac{f_{si} D}{3} + \lambda (f^*_{su} - f_{se})(D) \qquad (3)$$

where:

L_d = Development length, (in)

f_{si} = Stress in prestressed reinforcement at time of initial prestress (immediately after release in a pretensioned member), (psi)

D = Diameter of strand, (in)

λ = Multiplying factor applied to flexural bond length

f^*_{su} = Average stress in prestressing steel at ultimate load, (psi)

f_{se} = Effective steel prestress after losses, (psi)

Dr. Buckner theorized that the flexural bond length (the second term in Equation (3) above) was dependent upon the strain in the strand at maximum load—if more strain was present, then more flexural bond length would be required. Dr. Buckner's constant λ reflects that. For general applications, the expression for λ is:

$$\lambda = (0.6 + 40\,\epsilon_{ps}) \qquad (4)$$

where: ϵ_{ps} = Strain in prestressing steel at ultimate load, (microstrain)

Henceforth in this paper, Equation (3) will be referred to as the "Buckner equation." It should be noted that many of the studies that used high-strength concrete (high-strength concrete being concrete with an f'_c greater than 55 MPa [8 ksi]) had not been completed at the time of the Buckner study, so Buckner was not able to use their results.

Phase II of the FHWA study began after the Buckner report had been drafted, and it was planned to use the FHWA Phase II members to evaluate the proposed Buckner equation. Phase II involved full-size prestressed concrete girders and deck panels that were fabricated at a precast concrete plant. A total of 32 AASHTO Type II prestressed concrete I-girders and 32 prestressed concrete deck panels were fabricated in February through April of 1994. The girders used 12.7- and 15.2-mm- (0.5- and 0.6-in-) diameter strands, while the deck panels contained 9.5-mm- (3/8-in-) diameter strands. Some specimens contained only uncoated strands, while others included only epoxy-coated strands. Composite decks were added at the FHWA Structures Laboratory to some of the girders and deck panels. Testing was completed in summer 1995, and the results of these tests showed that the AASHTO equation was not conservative for girders with uncoated strands. It also showed that the Buckner equation was inconsistent for girders with uncoated strands—sometimes it was conservative and sometimes it was not conservative. See chapter 2 for detailed results from the FHWA Phase II study.

FHWA 1996 MEMORANDUM

By the early part of 1996, considerable research had been completed on the transfer and development length of uncoated prestressing strands, especially 15.2-mm- (0.6-in-) diameter strands spaced at 50.8 mm (2 in). Also at this time, FHWA was encouraging the use of high-performance concrete (HPC) in bridges and some States formed partnerships with FHWA to design and construct HPC bridges. In order to effectively use the higher strengths found in some HPC, the use of larger diameter prestressing strands was needed. This resulted in a need to use 15.2-mm- (0.6-in-) diameter strands, and a need to use these strands at the current spacing for 12.7-mm- (0.5-in-) diameter strands, namely 50.8 mm (2 in). A number of research studies had not only demonstrated successful use of the 15.2-mm- (0.6-in-) diameter strands at 50.8-mm (2-in) spacings, but had also demonstrated successful use of 12.7-mm- (0.5-in-) diameter strands at 44-mm (1.75-in) spacings.

Therefore, FHWA evaluated the data available at that time on 12.7- and 15.2-mm- (0.5- and 0.6-in-) diameter strands at reduced spacings of 44.4 mm and 50.8 mm (1.75 in and 2 in), respectively, and issued a revised memorandum in May 1996. This memorandum:

- Allowed the use of 15.2-mm- (0.6-in-) diameter strands in pretensioned applications.
- Allowed the following center-to-center strand spacings:
  –15.2-mm- (0.6-in-) diameter strands at 50.8-mm (2-in) spacing
  –12.7-mm- (0.5-in-) diameter strands at 44.4-mm (1.75-in) spacing
- Retained the multipliers for fully-bonded and debonded strands for use with the AASHTO development length equation.

The memorandum stated that the multipliers were to be retained "... until such time as a currently proposed development length equation by FHWA is reviewed and commented upon by the AASHTO Bridge Subcommittee on Bridges and Structures ..."(18)

The AASHTO Subcommittee on Bridges and Structures' Technical Committee #T-10 on Concrete, commonly known as AASHTO T-10, followed up the FHWA 1996 memorandum with a proposed revision to Article 9.26.2.1 of the AASHTO bridge specifications.(7) This revised article specified the minimum spacing for certain diameters of prestressing strands, and effectively allowed the spacings cited in the FHWA memorandum. This revised article was passed by the AASHTO Subcommittee on Bridges and Structures by letter ballot in fall 1997.

FORMULATION OF NEW DEVELOPMENT LENGTH EQUATION AND CONTENTS OF THIS REPORT

Because the FHWA transfer and development length experimentation on the Phase II members showed that the AASHTO and Buckner equations were "unconservative" in many instances, a new development length equation was needed. A new development length equation has been developed by FHWA based on the FHWA Phase II research results for beams. This new development length equation is representative of the FHWA research results and of research results from other studies as well. This equation has been correlated with research results from other studies.

Chapters 2 and 3 will describe the FHWA Phase II research results in detail and show how those results were used to develop the new equation. Chapters 4 and 5 will discuss the correlation of the new equation with research results from other studies and will draw final conclusions based on those correlations.

CHAPTER 2: FHWA PHASE II RESEARCH STUDY RESULTS

As discussed in the previous chapter, the FHWA study entitled "Investigation of Development Length of Uncoated and Epoxy-Coated Prestressing Strand" began in the spring of 1990 and consisted of two phases. Phase I involved 50 rectangular prestressed concrete members, while Phase II involved 64 members: 32 AASHTO Type II prestressed concrete I-beams and 32 prestressed concrete sub-deck panels. For Phase II, half of the beams and half of the deck panels contained uncoated strands, while the other halves contained epoxy-coated strands. This report discusses only the 16 beams that contained uncoated strands in Phase II.

DESCRIPTION OF MEMBERS

Beams

Three different strand patterns were used in the beams—strand patterns A, B, and C (see figure 2). These strand patterns were chosen to investigate how various strand diameters and spacings affected development length, and to see if any of these patterns resulted in cracking of the member at detensioning. Strand pattern A contained eight strands, 12.7 mm (0.5 in) in diameter, spaced at 50.8 mm (2 in) in one row in the bottom flange and two strands of the same diameter in the top flange. Strand pattern B contained nine strands, 12.7 mm (0.5 in) in diameter, spaced at 44.4 mm (1.75 in) in one row in the bottom flange and two strands of the same diameter in the top flange. Strand pattern C had eight strands, 15.2 mm (0.6 in) in diameter, spaced at 50.8 mm (2 in) in one row in the bottom flange and two strands of the same diameter in the top flange. All of the strands were fully stressed.

All of the beams were 9.46 m (31 ft) long, and all contained single-leg stirrups that were placed on alternate sides of the cross-section every 76.2 mm (3 in). For the first 0.92 m (3 ft) of each end of the beam, confinement reinforcement was placed in the top and bottom flanges. Accelerated curing was used for all members. Detensioning was accomplished through flame-cutting.

Six of the beams were made composite with a cast-in-place concrete deck that was cast at the FHWA Structures Laboratory. The other 10 beams did not have a deck cast on them. This was done to see how a higher maximum strain in the strand at failure would affect development length. When a deck is made composite with the beam and this composite member is then tested to failure, a high strain in the strand at failure can be achieved for flexural failures. These strains are typically near 0.03 or 3 percent. If the non-composite beam is tested to failure, typically the strain in the strand at a flexural failure is near the yield point of the strand, which is 0.01 or 1 percent. Table 1 lists the beams used in the study that had uncoated strands.

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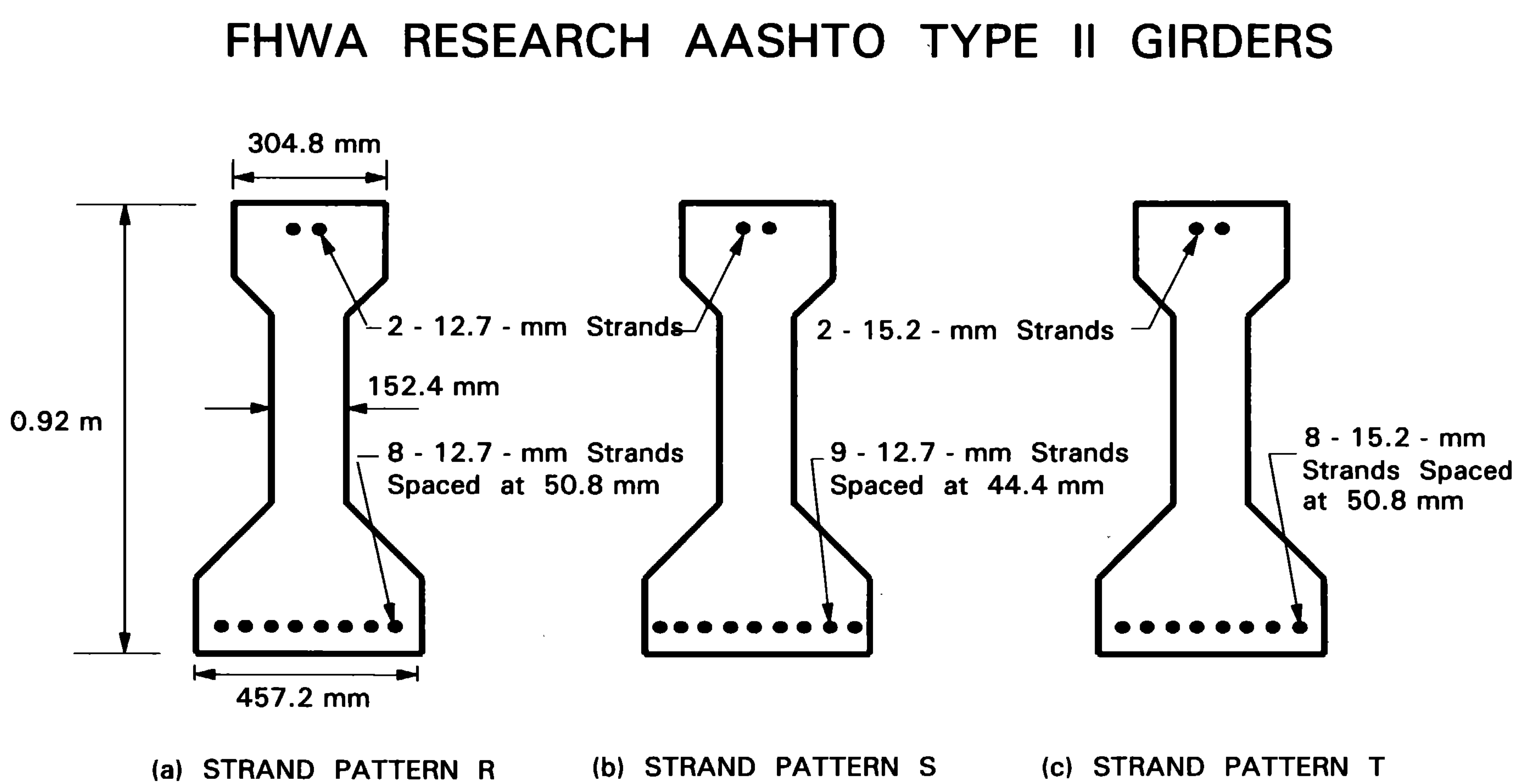


Figure 2. Cross-sectional views of FHWA research beams.

Table 1. AASHTO Type II beams with uncoated strands used in FHWA research study.

Girder No.	Strand Pattern	Made Composite With Slab?	Design f'_c (psi)	Measured f'_c (psi)	Design f_{pt} (psi)	Measured f_{pt} (psi)	Design f_{se} (ksi)	Actual f_{se} (ksi)
5U5-1	R	No	5,000	6,470	202,500	204,100	169.7	170
5U5-2	R	No	5,000	6,470	202,500	204,100	169.7	170
5U10-1	R	No	10,000	9,640	202,500	203,600	171.4	170
5U10-2	R	No	10,000	9,640	202,500	203,600	171.4	170
5U5-3	R	Yes	5,000	6,220	202,500	204,100	170.2	165
5U5-4	R	Yes	5,000	6,220	202,500	204,100	170.2	160
5U5-5	S	No	5,000	6,320	202,500	204,300	166.3	190
5U5-6	S	No	5,000	6,320	202,500	204,300	166.3	190
5U5-7	S	Yes	5,000	6,560	202,500	204,300	167.2	160
5U5-8	S	Yes	5,000	6,560	202,500	204,300	167.2	160
6U5-1	T	No	5,000	6,130	202,500	209,100	159.5	195
6U5-2	T	No	5,000	6,130	202,500	209,100	159.5	195
6U10-1	T	No	10,000	10,860	202,500	212,200	162.4	190
6U10-2	T	No	10,000	10,860	202,500	212,200	162.4	190
6U5-3	T	Yes	5,000	6,440	202,500	209,100	160.4	170
6U5-4	T	Yes	5,000	6,440	202,500	209,100	160.4	165

1 psi = 6.89 kPa, 1 ksi = 6.89 MPa

Prestressing Strand

All of the prestressing strands used in the beams were uncoated seven-wire, Grade 270 (1860 MPa [270 ksi] guaranteed ultimate tensile strength), low-relaxation strand, conforming to ASTM Standard A 416-90a.(19) The strand was used in the "as-received" condition, having occasional surface rust visible, but no pitting.

Concrete

Two different concrete mixes were used for the beams: one normal-strength mix and one high-strength mix. This was done to investigate the effect of concrete strength on development length. Twelve of the beams were fabricated with the normal-strength concrete mix. This normal-strength mix was designed to yield a 28-day compressive strength between 34.4 MPa and 44.8 MPa (5 ksi and 6.5 ksi). The limits of concrete strength, or "windows" of strength, were used so that differentiation could be made between the normal-strength and high-strength concretes. This mix was also designed to have a compressive strength at release of the prestress force equal to 27.6 MPa (4 ksi). Actual average compressive strengths for the normal-strength concretes were 31.5 MPa (4.6 ksi) at release, 44.2 MPa (6.4 ksi) for air-cured cylinders at 28 days, and 49.2 MPa (7.2 ksi) for moist-cured cylinders at 28 days.

Four of the beams were fabricated with the high-strength concrete mix. This mix was designed to yield a 28-day compressive strength between 68.9 MPa and 89.6 MPa (10 ksi and 13 ksi). Again, a window of strength was specified so that differentiation could be made between the normal-strength and high-strength concrete. The mix was also designed to have a compressive strength at release equal to 48.2 MPa (7 ksi). Actual average compressive strengths for the high-strength concretes were 54.3 MPa (7.9 ksi) at release, 71.3 MPa (10.4 ksi) for air-cured cylinders at 28 days, and 74.2 MPa (10.8 ksi) for moist-cured cylinders at 28 days.

INSTRUMENTATION

Mechanical Gauge Points

Mechanical gauge points (called Whittemore points) were attached to each beam on the exterior concrete surface to measure concrete surface strains. For most of the beams, these Whittemore points were placed at regularly spaced intervals along the top and bottom flanges at the level of the strands; some of the beams only had Whittemore points along the bottom flange. Spacings for the Whittemore points were 100 mm (3.94 in) for the top and bottom flanges at the ends of the beams, and 200 mm (7.87 in) for the top flanges along the mid-span regions of the beams. The distances between the Whittemore points were read both prior to and after detensioning. The differences in values between the two sets of readings were used to determine the strains in the concrete after detensioning. A full set of readings were also taken when the concrete was 7, 14, and 28 days old; prior to and after a composite deck was cast; and immediately before the development length experimentation.

End Slip

The end slip of each strand at both ends of each beam was measured. Before detensioning, a small channel-shaped fixture was attached to a strand adjacent to the end of a beam. Holes were bored in the legs of the fixture so it could accommodate a digital depth gauge that was used to measure the distance from the outer leg of the fixture to the concrete surface. This distance was measured before detensioning.

After detensioning, each strand was cut so that a minimum of 50.8 mm (2 in) still projected from the end of the member and the channel-shaped fixture was still attached to the projecting ends of strands. The distances were measured again after detensioning. The difference between these two distance values was the end slip of the strand at detensioning. A full set of end slip readings were taken at the same time that the mechanical gauge points were read for all of the time intervals described previously.

The end slip of the strand was also measured during the development length experimentation. Before a beam's development length testing, the last set of end slip readings were taken and then the channel-shaped fixtures were removed. Linear voltage displacement transducers (LVDTs) were attached to the projecting ends of all strands at the end of the member to be tested. As loading on that end of the member progressed, the LVDTs measured any changes in distances between where the LVDTs were attached to the strands and the concrete surface. Any changes in these distances, known as end slips, were caused by the strands losing their bond during the development length test loading and slipping inward.

Jacking Force

Each strand for each beam was tensioned individually using a single-strand prestressing jack. The jack was connected to a pressure gauge to measure force from the pressure applied to the jack. The actual elongation of each strand was also measured. This elongation was used, along with the physical properties of the strands, to calculate the stress in the strand prior to release.

Loads

Single-point loads during the development length experimentation were applied using a hydraulic jack and were measured using load cells placed at the hydraulic jack. The load cells were attached to a data acquisition system, which allowed the load values to be read and displayed on a computer screen every 2 seconds.

Electrical Resistance Strain Gauges

One or two electrical resistance strain gauges (hereafter simply called "strain gauges") were attached to the top of the member prior to the development length test for a given end of the beam. If the beam was non-composite, then one strain gauge was used and it was affixed to the top flange of the beam approximately 152 mm (6 in) away from the load. If the beam was composite with a deck, then two strain gauges were affixed to the top of the deck, one on either side of the load (approximately 152 mm [6 in] away from the load). These strain gauges were hooked up to a data acquisition system and were recorded continuously as the load changed during the development length test.

Deflection

Deflections were measured at mid-span and at the single-load point using linear potentiometers. These devices were also connected to the data acquisition system and were read and recorded continuously as the load changed during the development length test.

Other Measurements

Many other measurements were taken during this large research study. Other components of the study that were measured included:

- Internal concrete temperatures during curing.
- Cambers.
- Concrete shrinkage.
- Unit weights of the concrete.
- Moduli of elasticity of the concrete at release and at 28 days.
- Split-cylinder concrete strengths.
- Compressive strengths of the concrete (taken at the same time as the mechanical gauge readings and end slip readings).
- Coefficient of thermal expansion of the concrete.
- Pull-out strengths of untensioned strands in concrete blocks.
- Amounts of any phosphate residue on the surface of the strands.

However, the results and analyses from these measurements are beyond the scope of this paper.

TRANSFER LENGTH RESULTS

The transfer length is the distance required from the end of the member to fully transfer the effective prestressing force from the prestressing strand to the concrete by bond (see figure 3). There are three different methods for determining the transfer length using a graph: the 100-percent plateau method, the 95-percent plateau method, and the slope-intercept method. In all three methods, the strains calculated from the mechanical gauge point (Whittemore point) readings are plotted. These plots typically show a region of linearly varying strain leading to a region of constant strain, called the strain plateau. This is shown in figure 4.

For the 100-percent plateau method, the points defining the plateau are identified. The strain values of these points are then averaged together to produce a single constant strain value for the plateau. A horizontal line is drawn on the graph to represent the constant strain plateau (see figure 4). At the point where the linearly varying portion of the plot intersects the constant strain plateau, a vertical line is drawn to the x-axis. The value on the x-axis that corresponds to its intersection with the vertical line is the 100-percent plateau transfer length.

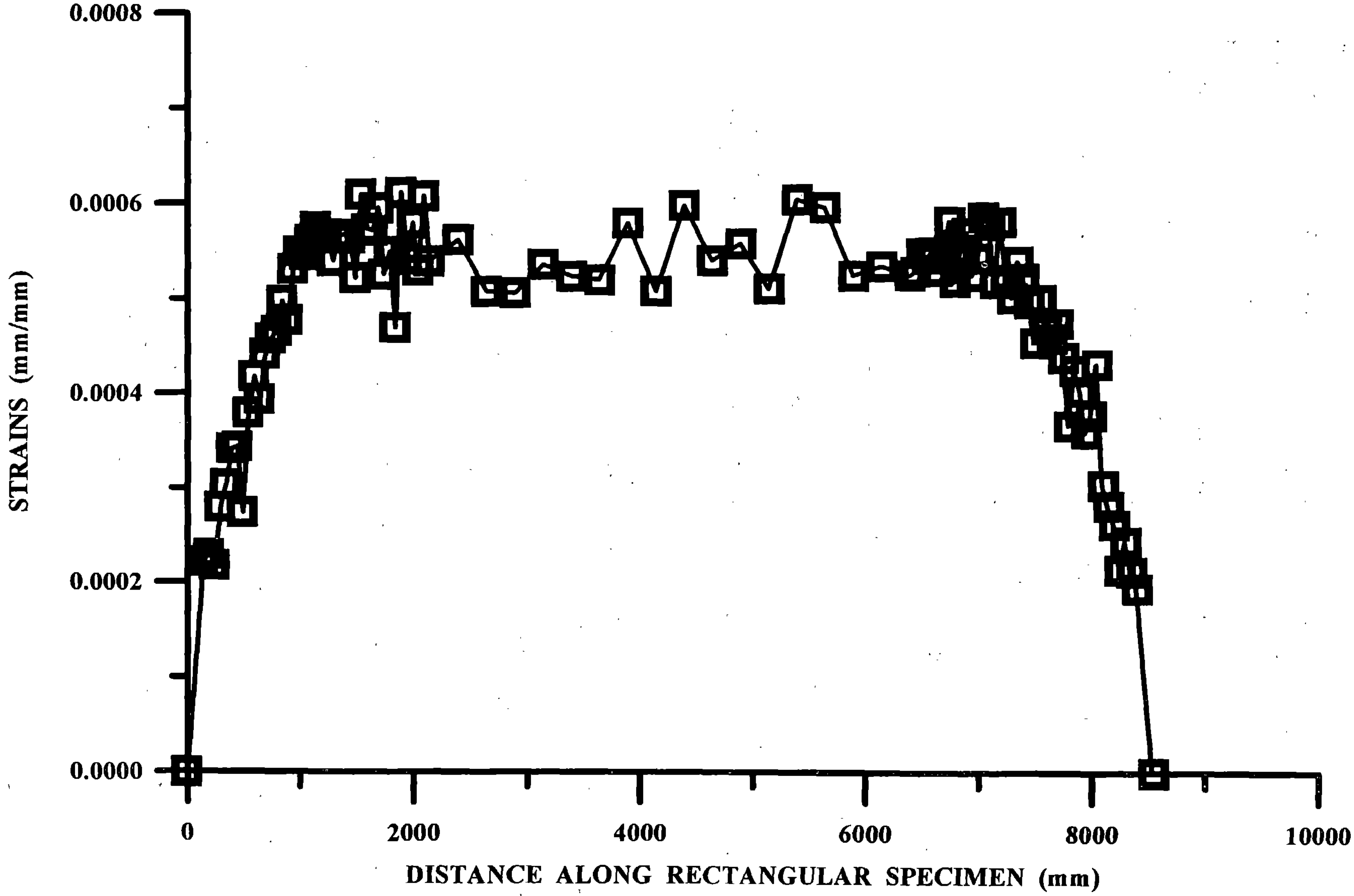


Figure 3. Strain profile for typical pretensioned concrete rectangular specimen at 28 days.

The 95-percent plateau method is similar to the 100-percent plateau method, and it is also shown in figure 4. The steps are initially the same, including determining and drawing the horizontal line representing the constant strain plateau; however, once the constant strain plateau has been determined, it is multiplied by 0.95 to obtain a 95-percent constant strain value. Then a second horizontal line, representing the 95-percent constant strain plateau, is drawn on the graph. At the point where the linearly varying portion of the plot intersects this 95-percent constant strain plateau, a vertical line is drawn to the x-axis. The value on the x-axis that corresponds to its intersection with the vertical line is the 95-percent plateau transfer length.

The slope-intercept method consists of plotting two lines on the graph: a horizontal line representing the 100-percent strain plateau and a second straight line that represents the points in the linearly varying strain region (see figure 4). The intersection of these two straight lines is the key point, and the location of this point is highly dependent upon how the second line is drawn. At the intersection point, a vertical line is drawn to the x-axis. The value on the x-axis that corresponds to its intersection with the vertical line is the slope-intercept transfer length.

The approximately 60 research studies on development length to date have used all three methods to determine transfer length. To be able to compare transfer lengths of one study to transfer lengths of another study, some consistent method of determining transfer lengths needs to be employed. Buckner examined this issue in his work for FHWA, and in the process, he identified several possible sources of error that influence the various methods of determining transfer lengths. He identified errors caused by overlapping gauge lengths, shear lag, and beam weight in eccentrically prestressed members.[(17)] His study concluded that:

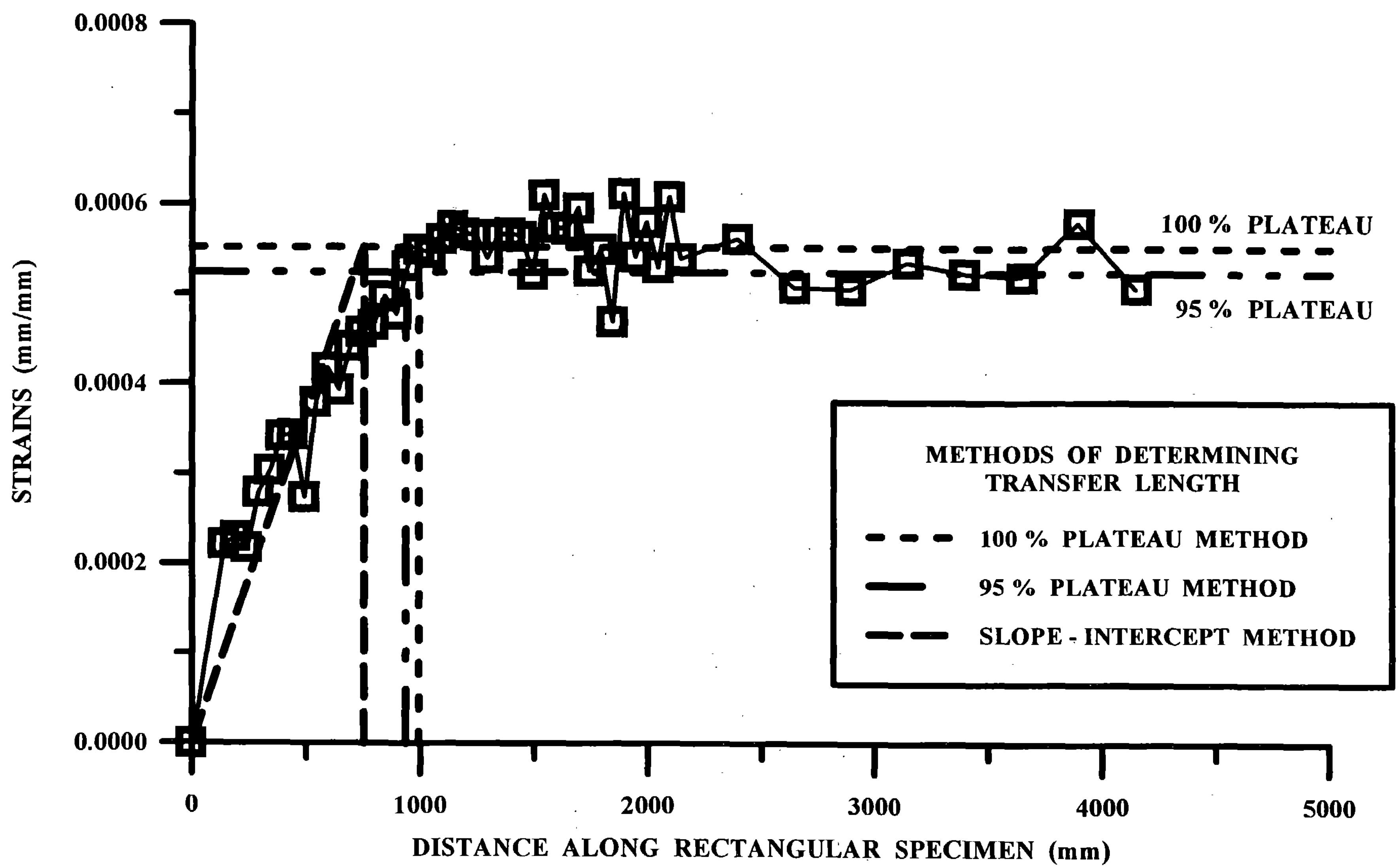
> "The 95 percent constant strain method ... provides a simple, objective, upper-bound estimate of transfer length and is recommended as the basis for reporting transfer lengths in future studies."[(17)]

Because of Buckner's recommendation, the FHWA Phase II transfer lengths were determined by the 95-percent plateau method, even though the FHWA Phase I transfer lengths had been determined by the 100-percent plateau method. The transfer lengths for the Phase II beams are provided in table 2.

The Phase II transfer lengths are those determined at 28 days of concrete age. The 28-day values of transfer length were used for two reasons:

- Magnitude of the Whittemore readings at 28 days.
- Growth of transfer length with time.

The Whittemore readings taken at transfer were small measurements, and their magnitudes were close to the accuracy of the instrument. However, after 28 days had elapsed, the magnitudes of the Whittemore readings increased because of creep and shrinkage. The plots of the strains determined from the Whittemore readings were more defined at 28 days than at the time of transfer.



100 % PLATEAU TRANSFER LENGTH = 992 mm (39.0 in)
95 % PLATEAU TRANSFER LENGTH = 936 mm (36.8 in)
SLOPE - INTERCEPT TRANSFER LENGTH = 750 mm (29.5 in)

Figure 4. Strain profile illustrating methods for determining transfer length for one end of a rectangular specimen.

Hence, the transfer lengths determined from these strains were more easily and reliably determined.

The FHWA Phase II transfer length values for beams with uncoated strands grew an average of 30 percent from the time of transfer to 28 days. After 28 days, the transfer length only grew an additional 7 percent (average) until the time of the development length test, which occurred at an age of 185 days, or approximately 6 months (on average).

It should be noted that Russell and Paulsgrove, in their report based on the strand bond research done at the University of Oklahoma, cited that transfer length increased 30 percent between transfer and the development length test.[20] For the FHWA data, most of the growth took place between transfer and 28 days. Because of this and because of the larger magnitude of the Whittemore readings, which resulted in better plots of the strain data, the 28-day values of transfer length were chosen as the cited data.

The values shown in table 2 are compared with predicted transfer length values computed using the AASHTO equation ($f_{se}D/3$) and the Buckner equation ($f_{si}D/3$). It is evident (as shown in table 2) that measured values of transfer length were greater than the values predicted by AASHTO and Buckner for all of the strands in normal-strength concrete. It is also evident that measured values of transfer length were less than the values predicted by AASHTO and Buckner for all of the strands in high-strength concrete. A third observation from this table is that measured transfer length values for a given size of strand were consistently lower in high-strength concrete than in normal-strength concrete. However, neither the values predicted by the AASHTO equation nor the values predicted by the Buckner equation indicate this trend.

Based on this FHWA data, it was concluded that a more conservative equation was needed for transfer lengths in normal-strength concrete and that a revised equation was needed that would give lower transfer length values when high-strength concrete was used. This would provide a substantiated benefit for designers to use high-strength concrete.

Table 2. Measured and predicted transfer lengths at 28 days for FHWA research beams.

Girder No.	Strand Pattern	Measured f'_c (psi)	FHWA Measured Transfer Length End A (in)	FHWA Measured Transfer Length End B (in)	Predicted AASHTO Transfer Length (in)	Predicted Buckner Transfer Length (in)
5U5-1	R	6,470	46.1	*	28.3	33.8
5U5-2	R	6,470	46.0	*	28.3	33.8
5U10-1	R	9,640	12.4	15.7	28.6	33.8
5U10-2	R	9,640	25.2	23.7	28.6	33.8
5U5-3	R	6,220	45.7	41.8	28.4	33.8
5U5-4	R	6,220	49.6	31.1	28.4	33.8
5U5-5	S	6,320	55.9	43.5	27.7	33.8
5U5-6	S	6,320	41.5	46.7	27.7	33.8
5U5-7	S	6,560	40.0	47.2	27.9	33.8
5U5-8	S	6,560	40.0	42.4	27.9	33.8
6U5-1	T	6,130	53.5	57.4	31.9	40.5
6U5-2	T	6,130	61.6	59.8	31.9	40.5
6U10-1	T	10,860	19.1	26.9	32.5	40.5
6U10-2	T	10,860	22.8	26.0	32.5	40.5
6U5-3	T	6,440	56.1	55.6	32.1	40.5
6U5-4	T	6,440	63.0	40.6	32.1	40.5

* No plateau was reached by the end of instrumentation (at 56 in), so no transfer length could be determined.

1 in = 25.4 mm, 1 psi = 6.89 kPa

DEVELOPMENT LENGTH RESULTS

Development length experimentation was performed on each end of each girder, provided that there was sufficient length to perform a second test on a girder. The girders were fabricated in identical pairs in order to represent a specific girder type, such as "15.2-mm- (0.6-in-) diameter strands at 50-mm (2-in) spacing in high-strength concrete." In that way, a total of four development length tests could be performed for each girder type (pair of girders).

Each end of each girder was instrumented with linear variable displacement transducers (LVDT) on each strand. These LVDTs measured the distance that a strand would slip into the girder end during loading of the girder. Electrical resistance strain gauges were placed along the top fiber of the concrete. For a non-composite girder, the top fiber of the concrete was the surface of the top flange; for the composite girder, the top fiber of the concrete was the surface of the deck.

Deflection was measured using linear potentiometers at the load point and at mid-span for each of the tests. Loads applied to the girders were measured using a load cell. All of these measuring devices were connected to a data acquisition system and were read continuously during a development length test.

Testing for development length was a trial-and-error (or iterative) approach. For the first test, the development length was calculated from a known equation, such as the AASHTO or Buckner equation. This length was called the embedment length. A single-point load was applied to the girder at a distance from the end of the girder equal to the embedment length (see figure 5). This load was incrementally increased until either a bond failure or a flexural failure occurred.

A bond failure was marked by the strands slipping inward. The FHWA researchers set a measurement of 0.25 mm (0.01 in) of slip as the threshold value for determining whether a strand had failed in bond slip. It was determined that if the strand slipped by that value, then the stress state in the beam had been significantly altered. Each of the girders had two top strands, and either eight or nine bottom strands. If a majority of the bottom strands slipped, or if the girder could not support the increased load as the strands were slipping, then the test was deemed a bond failure. In general, the slip failures were accompanied by shear cracking in the end regions. Of the 13 girder tests that resulted in bond failures, 12 of these were accompanied by shear cracking. A bond failure signified inadequate embedment length.

A flexural failure was marked by flexural cracking at or near the load point, flexure-shear cracking as the cracks got farther away from the load point, and crushing of the compression zone concrete. If the compression zone concrete was crushed, or if the girder would not support increased load, then the test was deemed a flexural failure. In general, when a flexural failure occurred, no strands slipped—even though some shear cracks may have been present in the end regions at the time of flexural failure. Typically, one would think that a flexural failure would be marked by the breaking of strands rather than the crushing of concrete. However, because of the full row of strands in the bottom row of the girder and because of the relatively small compression zone at the top of the girder (alone) or at the top of the composite girder, the

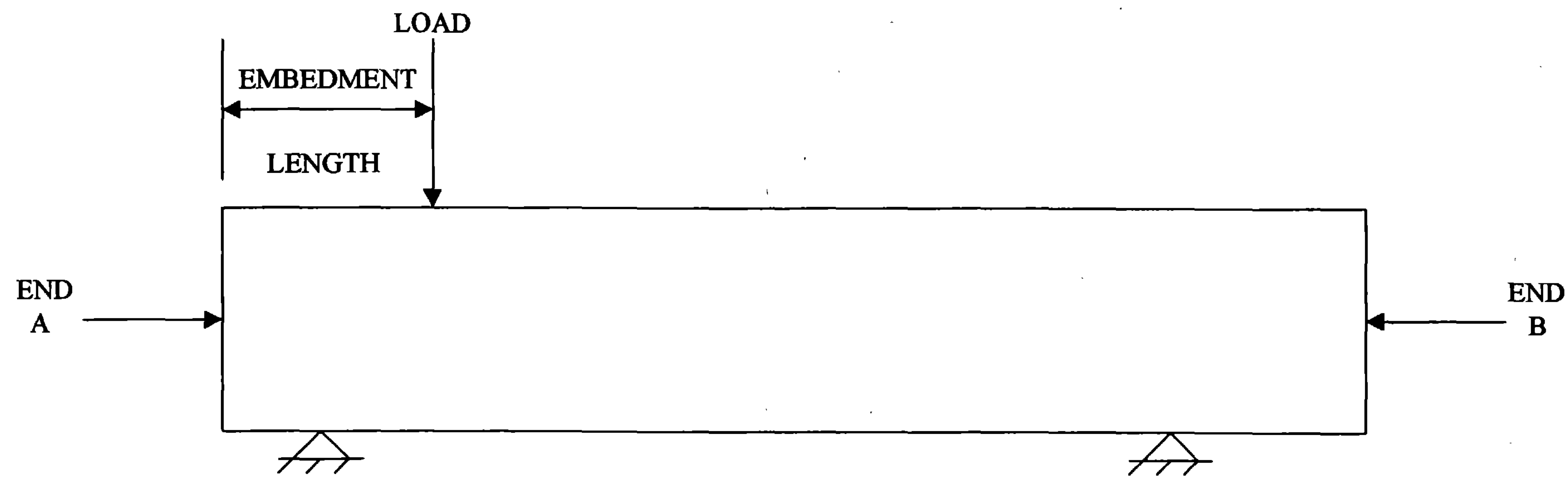


Figure 5. Development length test set-up.

compression zone was actually weaker than the tension zone. Thus, the compression zone failed first. A flexural failure signified adequate embedment length.

In a few instances, a flexural failure occurred at the same time as a bond failure of some strands. In those instances, the failure was labeled a slip/flexural failure. Because slip and flexural failures were occurring at the same time, that type of failure signified that the embedment length being tested equaled the actual development length.

The reason for girder failure at one end of the girder dictated what the embedment length would be for the opposite end of that girder. If a flexural failure occurred, then the embedment length was decreased for the test on the opposite end. If a bond failure occurred, then the embedment length was increased for the test on the opposite end. This iterative approach was employed until both ends of both girders were tested and the development length was determined. The results from all of the FHWA Phase II girder development length tests are shown in tables 3, 4, and 5. Table 3 includes data for the normal-strength, non-composite girders (girders alone); table 4 includes data for the high-strength, non-composite girders; and table 5 provides data for the normal-strength concrete girders made composite with a deck.

The values in tables 3, 4, and 5 are compared with predicted values of development length from the AASHTO and Buckner equations. For every girder fabricated with normal-strength concrete, the measured development length was longer than the AASHTO-predicted development length. The Buckner equation was inconsistent when compared to the measured values of development length in normal-strength concrete; sometimes the Buckner value was shorter than the measured value and sometimes it was longer than the measured value. When compared to measured values for high-strength concrete, the development length values predicted by AASHTO and Buckner were consistently longer than the measured values. It is also evident from tables 3 and 4 that the measured values of development length for a given strand size were shorter for high-strength concrete than for normal-strength concrete.

Because of the unconservative AASHTO equation and the inconsistent Buckner equation for normal-strength concrete, it was determined that a new development length equation was needed—a new equation that could provide conservative predictions of development length for all concrete strengths, yet not be overly conservative for high-strength concretes.

STRUCTURAL ANALYSIS

Because the FHWA girder results indicated the need for a new development length equation, the FHWA researchers decided to examine a host of variables for possible inclusion in a new equation. These variables were:

- Concrete compressive strength at 28 days, (f'_c).
- Square root of concrete compressive strength at 28 days, ($\sqrt{f'_c}$).

Table 3. Development length test results for normal-strength, non-composite FHWA research beams.

Girder No.	End	Embedment Length, (in)	Span Length, (in)	Maximum Sustained Load, (kips)	Failure Type	AASHTO Predicted Development Length, (in)	Buckner Predicted Development Length, (in)
5U5-1	A	76	360	110.0	Slip/Shear	75.8	90.2
5U5-1	B	112	270	170.4	Flexural	75.8	90.2
5U5-2	A	88	360	124.4	Slip/Shear	75.8	90.2
5U5-2	B	106	222	202.0	Slip/Shear/Flexural	75.8	90.2
5U5-5	A	112	270	159.9	Slip	76.5	89.5
5U5-5	B	116	222	212.0	Flexural	76.5	89.5
5U5-6	A	96	222	224.5	Slip/Shear	76.5	89.5
5U5-6	B	96	222	231.1	Slip/Shear	76.5	89.5
6U5-1*	A	143	282	205.0	Flexural	92.6	104.2
6U5-2	A	101	222	200.0	Slip/Shear	92.6	104.2
6U5-2	B	131	258	221.9	Slip/Shear	92.6	104.2

* No test for End B due to insufficient beam length remaining.

1 in = 25.4 mm, 1 kip = 1000 lb = 4448 N

Table 4. Development length test results for high-strength, non-composite FHWA research beams.

Girder No.	End	Embedment Length, (in)	Span Length, (in)	Maximum Sustained Load, (kips)	Failure Type	AASHTO Predicted Development Length, (in)	Buckner Predicted Development Length, (in)
5U10-1	A	101	222	222.5	Flexural	76.5	104.4
5U10-1	B	86	222	247.0	Flexural	76.5	104.4
5U10-2	A	76	222	219.5	Slip/Shear	76.5	104.4
5U10-2	B	86	222	236.7	Flexural	76.5	104.4
6U10-1	A	114	234	275.0	Flexural	94.4	117.5
6U10-1	B	102	228	285.0	Slip/Flexural	94.4	117.5
6U10-2	A	94	222	274.0	Slip/Shear	94.4	117.5
6U10-2	B	98	222	260.0	Slip/Shear	94.4	117.5

1 in = 25.4 mm, 1 kip = 1000 lb = 4448 N

Table 5. Development length test results for normal-strength, composite FHWA research beams.

Girder No.	End	Embedment Length, (in)	Span Length, (in)	Maximum Sustained Load, (kips)	Failure Type	AASHTO Predicted Development Length, (in)	Buckner Predicted Development Length, (in)
5U5-3	A	106.0	222	264.0	Slip/Shear	77.4	120.4
5U5-3	B	126.0	246	260.0	Slip/Shear/Flexural	77.4	120.4
5U5-4*	A	138.0	360	183.0	Flexural	77.4	120.4
5U5-7	A	131.0	257	298.0	Flexural	78.5	132.4
5U5-7	B	100.0	198	297.0	Slip/Shear	78.5	132.4
5U5-8	A	115.0	225	280.0	Slip/Shear	78.5	132.4
5U5-8	B	119.0	232	319.6	Slip/Shear/Flexural	78.5	132.4
6U5-3†	A	178.5	360	245.7	Flexural	96.8	158.4
6U5-3†	B	193.5	360	245.7	Flexural	96.8	158.4
6U5-4‡	B	143.0	360	263.5	Flexural	96.8	158.4

* No test for End B due to insufficient beam length remaining.
† Test was a combined test for both Ends A and B, with a load placed 178.5 inches from End A (which also meant that the load was placed at 193.5 inches from End B).
‡ No test for End A due to insufficient beam length remaining.

1 in = 25.4 mm, 1 kip = 1000 lb = 4448 N

- Concrete compressive strength at transfer of prestress, (f'_{ci}).
- Square root of concrete compressive strength at transfer of prestress, ($\sqrt{f'_{ci}}$).
- Concrete modulus of elasticity at 28 days, (E_c).
- Concrete modulus of elasticity at transfer of prestress, (E_{ci}).
- Concrete unit weight, (w_c).
- Depth of concrete rectangular stress block, (a).
- Prestressing strand diameter, (D).
- Area of prestressing steel strand, (A_s*).
- Stress in prestressing strand prior to transfer of prestress, (f_{pt}).
- Effective prestress, (f_{se}).
- Stress in prestressing strand at ultimate strength of the member, (f^*_{su}).
- Strain in prestressing strand at ultimate strength of the member, (ϵ_{su}).

Many of these parameters were measured during experimentation or were easily determined from measured values. Parameters that fit into this category were the concrete strengths at various ages (and the square roots of these concrete strengths), the strand diameter and area, the modulus of elasticity of the concrete at various ages, the concrete unit weight, and the stress in the strand prior to transfer. However, certain parameters were not measured nor were easily determined from measured values. Parameters that fit into this category were the depth of the rectangular stress block, the effective prestress, and the stress and strain of the strands at ultimate strength of the member. The AASHTO Specification(7) does provide equations for calculating the amount of prestress losses and thereby determining the effective prestress, f_{se}. These equations have been routinely used for normal-strength concrete, but insufficient data existed to determine whether or not they were accurate for high-strength concrete. The AASHTO Specification also provides an equation (Equation 9-17) for calculating the stress in the strand at ultimate strength of the member, f^*_{su}. This AASHTO equation was based on the ACI Building Code(6) equation to determine the same parameter. Section R18.7.2 of the Commentary for the ACI Building Code on the ACI equation indicates that use of that equation is appropriate when all of the prestressing strand is in the tension zone, and that a strain compatibility and equilibrium method of analysis should be used in lieu of the equation when any of the prestressing strand is in the compression zone. For the FHWA girders, two of the prestressing strands were located in the compression zone (top flange), thereby eliminating use of the ACI and AASHTO equations and necessitating the use of a strain compatibility method of analysis.

In order to determine these hard-to-obtain parameters, two computer programs were developed by Dr. Fassil Beshah and Dr. Nicolas Gagarin that were based on the strain compatibility method of analysis. For the first program, inputs were required for concrete strength, modulus of elasticity, and modulus of rupture; properties of the strand; and loading and cross-section information for a given development length test. The load and deflection data from the test was then used in combination with the aforementioned input items in an iterative approach that yielded the effective prestress force, f_{se}. For the second program, all of the inputs and values determined by the first program were used as well as the measured external concrete strain of the top fiber (compression fiber) at the load point. This concrete top fiber strain was measured continuously during each FHWA girder development length test. By using the value of this strain at failure, then the stress and strain of the bottom strands at failure of the member, f^*_{su} and ϵ_{su}, respectively, were determined by an iterative approach contained within the second computer program. The distance to the neutral axis, "a", and the depth of the rectangular stress block, "c", were also results obtained from this program. The results from these two computer programs are provided in tables 1, 6, and 7. Table 1 provides f_{se} values determined from the structural analyses (listed as "actual f_{se}"), while tables 6 and 7 provide f^*_{su} and ϵ_{su} values determined from the structural analyses and are termed "actual f^*_{su}" and "actual ϵ_{su}." Table 6 provides the values for members constructed with normal-strength concrete (compressive strength between 34.4 MPa and 44.8 MPa [5 ksi and 6.5 ksi]), while table 7 provides the values for members constructed with high-strength concrete (compressive strength between 68.9 MPa and 89.6 MPa [10 ksi and 13 ksi]).

In general, the strains in the strands at flexural failure for the non-composite girders (girders alone) were very close to the yield point of the strand, 0.010 mm/mm (0.010 in/in). This contrasted with the strains in the strands at flexural failure for the composite girders (girders with decks), where the strains were greater than or equal to 0.030 mm/mm (0.030 in/in). This difference in strain values was first noted by Buckner(17) and is significant for two reasons. First, the strand that meets ASTM A 416(19) is guaranteed to be able to sustain an elongation of 3.5 percent, which is equivalent to a strain of 0.035 mm/mm (0.035 in/in). Therefore, as the strain in the strand approaches 0.035, the development length test is approaching the capacity of the strand. Second, engineers are familiar with the property of all materials whereby as a material elongates in one direction, that elongation is accompanied by contraction in the transverse direction of the material. This is the basis for the familiar Poisson's ratio. This property applies to strand. As the strand elongates to a strain of 0.030 or greater, then it contracts in diameter. This contraction is in a direction that pulls the strand away from the concrete/strand interface, so the more the strand elongates, the more the cross-sectional dimension contracts, and the potential for bond worsens. Therefore, the strains in the strands indicated that the worst-case scenario for bond of strands had been tested.

Table 6. Development length test results from structural analysis of FHWA research beams constructed with normal-strength concrete.

Girder No.	End	Design f^*_{su}, (ksi)	Actual f^*_{su}, (ksi)	Design ϵ_{su}, (in/in)	Actual ϵ_{su}, (in/in)
5U5-1	A	264.8	180	0.0147	0.00631
5U5-1	B	264.8	266	0.0147	0.01621
5U5-2	A	264.8	208	0.0147	0.00729
5U5-2	B	264.8	267	0.0147	0.01608
5U5-3	A	268.2	261	0.0292	0.01127
5U5-3	B	268.2	264	0.0292	0.01337
5U5-4	A	268.2	268	0.0292	0.03136
5U5-5	A	263.9	264	0.0135	0.01427
5U5-5	B	263.9	*	0.0135	*
5U5-6	A	263.9	265	0.0135	0.01437
5U5-6	B	263.9	264	0.0135	0.01384
5U5-7	A	268.5	268	0.0337	0.02854
5U5-7	B	268.5	246	0.0337	0.00868
5U5-8	A	268.5	†	0.0337	†
5U5-8	B	268.5	269	0.0337	0.04008
6U5-1	A	260.6	262	0.0112	0.01222
6U5-2	A	260.6	210	0.0112	0.00735
6U5-2	B	260.6	263	0.0112	0.01315
6U5-3	A	268.3	268	0.0305	0.02431
6U5-3	B	268.3	268	0.0305	0.02431
6U5-4	B	268.3	268	0.0305	0.03144

* Not able to be determined because all strain and deflection data were lost due to power failure in laboratory.

† Not able to be determined because there was an error in recorded strains during testing.

1 in/in = 1 mm/mm, 1 ksi = 6.89 MPa

Table 7. Development length test results from structural analysis of FHWA research beams constructed with high-strength concrete.

Girder No.	End	Design f^*_{su}, (ksi)	Actual f^*_{su}, (ksi)	Design ϵ_{su}, (in/in)	Actual ϵ_{su}, (in/in)
5U10-1	A	267.3	263	0.0218	0.01271
5U10-1	B	267.3	267	0.0218	0.02067
5U10-2	A	267.3	261	0.0218	0.01139
5U10-2	B	267.3	267	0.0218	0.02076
6U10-1	A	265.6	264	0.0161	0.01416
6U10-1	B	265.6	265	0.0161	0.01538
6U10-2	A	265.6	264	0.0161	0.01336
6U10-2	B	265.6	262	0.0161	0.01190

1 in/in = 1 mm/mm, 1 ksi = 6.89 MPa

CHAPTER 3: FORMULATION OF A NEW DEVELOPMENT LENGTH EQUATION FROM FHWA RESULTS

As was described in the previous chapter, results from the FHWA development length experimentation on full-size girders indicated that the existing AASHTO equation was unconservative and that the Buckner equation was inconsistent. Therefore, a new development length equation was needed. FHWA decided to formulate the equation based on the FHWA full-size girder research results, and then correlate this equation with other research results to make sure that the equation would be representative of the total applicable data to date. This chapter will describe the formulation of the new equation based on the FHWA full-size girder research results.

NEW TRANSFER LENGTH EXPRESSION

As was previously described in chapter 1, the transfer length is a component of the development length. However, it is also a stand-alone parameter because of its use in the shear provisions of the AASHTO Specifications and the ACI Building Code. In these provisions, the transfer length of the strand is used to determine the component of prestressing force in a draped (inclined) strand that can be used to resist shear. Therefore, it was important to have an expression for transfer length that could be used independent of the total development length expression.

A number of parameters were investigated for possible use in the new transfer length equation. These included:

- Concrete compressive strength at 28 days, (f'_c).
- Square root of concrete compressive strength at 28 days, ($\sqrt{f'_c}$).
- Concrete compressive strength at transfer of prestress, (f'_{ci}).
- Square root of concrete compressive strength at transfer of prestress, ($\sqrt{f'_{ci}}$).
- Concrete modulus of elasticity at 28 days, (E_c).
- Concrete modulus of elasticity at transfer of prestress, (E_{ci}).
- Concrete unit weight, (w_c).
- Prestressing strand diameter, (D).
- Area of prestressing steel strand, (A_s*).
- Stress in prestressing strand prior to transfer of prestress, (f_{pt}).
- Effective prestress, (f_{se}).

These parameters were investigated initially by inspecting the data and then evaluating the most promising parameters, which were evaluated along with the measured 28-day values of transfer length using regression analyses. It should be noted that the unit weight of the concrete was examined, but it was determined that there was not enough variation of the unit weight within the FHWA girder study to include it as a variable.

Regression analyses are statistical studies of relationships between variables.[21] Before doing regression analyses, researchers plotted different combinations of the variables' measured values. As the FHWA researchers plotted data, it was apparent that the relationship between some of the variables was approximately linear. They performed straight-line regression analyses to test the assumption that the relationship was of the following form:

$$y = mx + b \tag{5}$$

where:

	$y =$	dependent variable
	$m =$	slope of the line
	$x =$	independent variable
	$b =$	y-intercept

The method of least squares was used to determine the best-fit line. This method minimizes the y distances between the measured y-values and the y-values calculated using the equation y=mx+b. This method was used to determine best-fit lines for different combinations of variables. The FHWA researchers could then compare the best-fit lines between different combinations of variables using a parameter called the "coefficient of determination." The coefficient of determination is a measure of the strength of a linear relation.[21] The coefficient of determination can vary between zero and one. A coefficient of determination of zero indicates that there is no relationship and a coefficient of determination of one indicates a perfect relationship. The combination of variables and best-fit line that had the highest coefficient of determination was then chosen as the best obtainable transfer length equation.

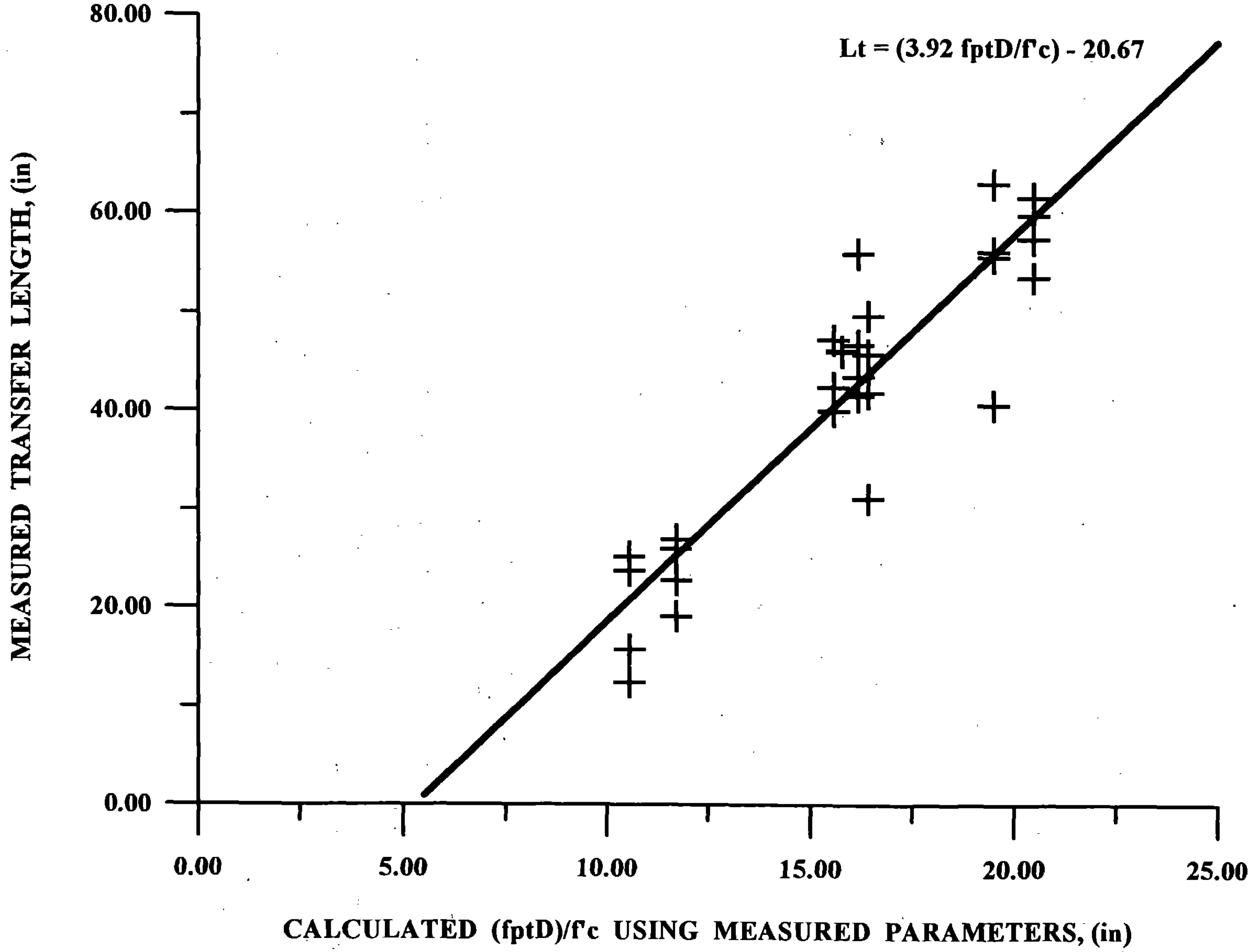
The measured (or actual) values of the variables listed above were used in the regression analysis process just described. The best-fit line that obtained the highest coefficient of determination was chosen as the transfer length equation and is shown below:

$$L_t = \frac{3.92 f_{pt} D}{f'_c} - 20.67 \tag{6}$$

where L_t is the transfer length.

Figure 6 shows a graph of the measured values of transfer length versus the expression $f_{pt}D/f'_c$ for data from the FHWA Phase II beams. The expression $f_{pt}D/f'_c$ also contains the measured (actual) values of the parameters in it. Equation (6) is also plotted in this figure. It should be noted that Equation (6) is a best-fit line; it is not a conservative equation nor does it contain any factor of safety. It simply predicts mean values for transfer length based on the measured data from the FHWA Phase II girders.

Because designers would not be privy to actual values of parameters while they were doing their design, the FHWA researchers plotted Equation (6) on a graph of measured values of transfer length versus the expression $f_{pt}D/f'_c$, which contained design values of those parameters. This is shown in figure 7. It can be seen from this figure that Equation (6) is not necessarily the best-fit line for this set of data. However, it is a more conservative line for the design values.



1 in = 25.4 mm

Figure 6. Relationship between measured values of fptD/f'c and measured transfer length.

FHWA researchers decided to modify Equation (6) slightly by rounding-off the values of the constants to obtain:

$$L_t = \frac{4 f_{pt} D}{f'_c} - 21 \qquad (7)$$

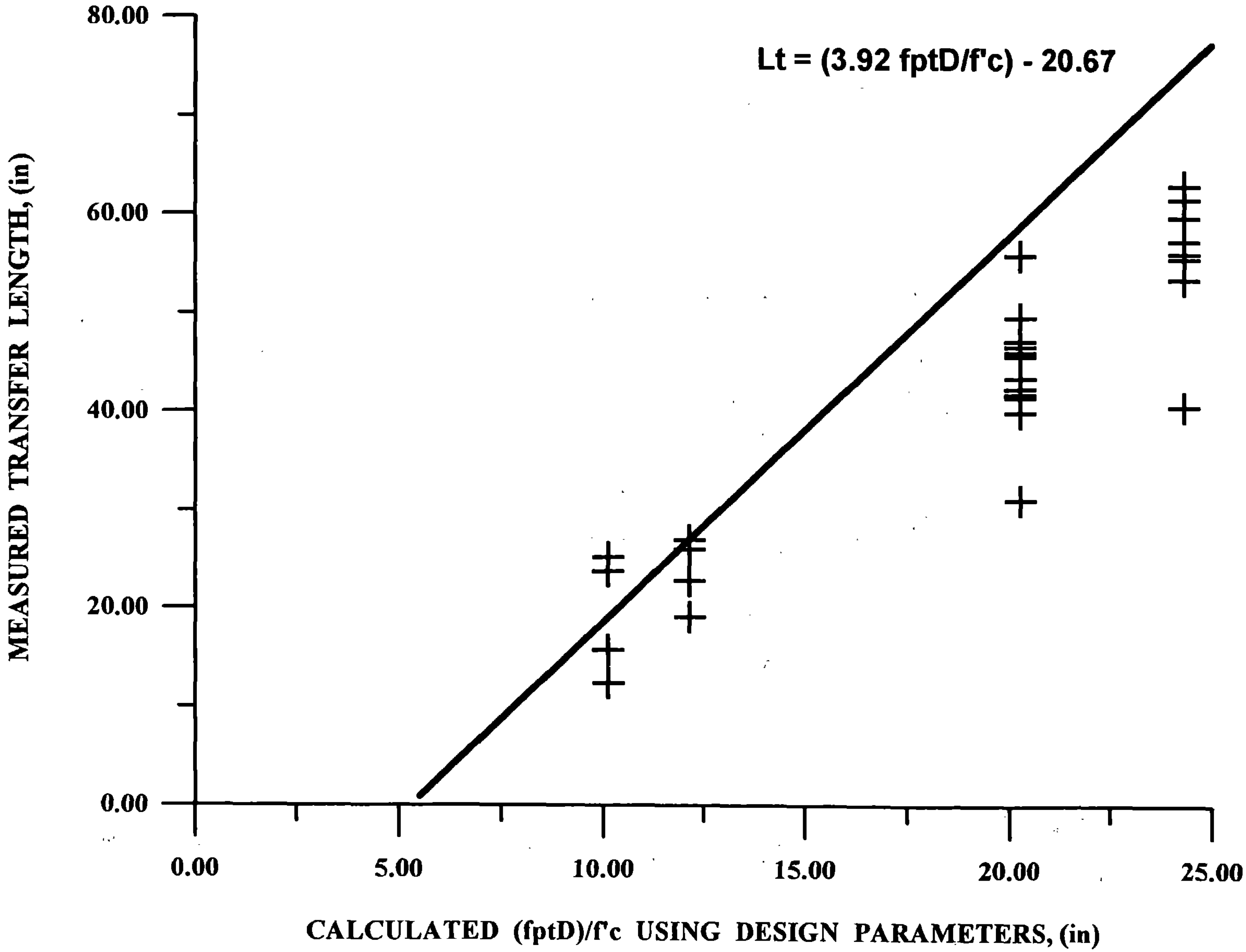
and then to adopt Equation (7) as the proposed transfer length expression pending correlation with data from other studies. It should be noted that Equation (7) was presented as a potential transfer length equation in a progress report to the AASHTO Technical Committee on Concrete (AASHTO Committee No. T-10) on May 13, 1996 in Philadelphia, PA.

NEW FLEXURAL BOND LENGTH EXPRESSION

The flexural bond length represents the additional bond length needed beyond the transfer length to support the ultimate strength of the member. It is not a stand-alone expression, but rather a major portion of the development length. The following parameters were investigated for possible use in the flexural bond length expression:

- Concrete compressive strength at 28 days, (f'_c).
- Square root of concrete compressive strength at 28 days, ($\sqrt{f'_c}$).
- Concrete modulus of elasticity at 28 days, (E_c).
- Concrete unit weight, (w_c).
- Depth of concrete rectangular stress block, (a).
- Prestressing strand diameter, (D).
- Area of prestressing steel strand, (A_s*).
- Effective prestress, (f_{se}).
- Stress in prestressing strand at ultimate strength of the member, (f^*_{su}).
- Strain in prestressing strand at ultimate strength of the member, (ϵ_{su}).

It should be noted that the unit weight of the concrete was examined, but it was determined that there was not enough variation of the unit weight within the FHWA girder study to include it as a variable. The rest of the parameters were investigated initially by inspection of data and then the most promising parameters were numerically evaluated along with the "measured" values of flexural bond length. There is no way to exactly measure the flexural bond length. The term "measured flexural bond length" means that for a given end of a beam, the measured transfer length was subtracted from the embedment length used in that end's development length test to obtain the "measured" flexural bond length.



1 in = 25.4 mm

Figure 7. Relationship between design values of fptD/f'c and measured transfer length.

The parameters listed previously were numerically evaluated using a computer program called TableCurve 2D. This program numerically scans data for a given set of parameters and evaluates the relationship between the parameters against a set of equation types to provide the best-fit equations for the parameters. Using this program, the parameters were evaluated with TableCurve 2D and more than 3400 equation types were initially examined. These included the following:

- Linear equations.
- Non-linear equations.
- Log (Base 10) equations.
- Natural log (Base e) equations.
- Semi-log (Base 10) equations.
- Semi-natural log (Base e) equations.

TableCurve 2D uses the method of least squares to determine the best-fit equations.(22) The FHWA researchers also used an option on the TableCurve 2D program that applies a simple equation filter, which weeds out the more complex of the equation types. This was done to ensure that the equation selected was not only an accurate equation, but an understandable and usable equation as well.

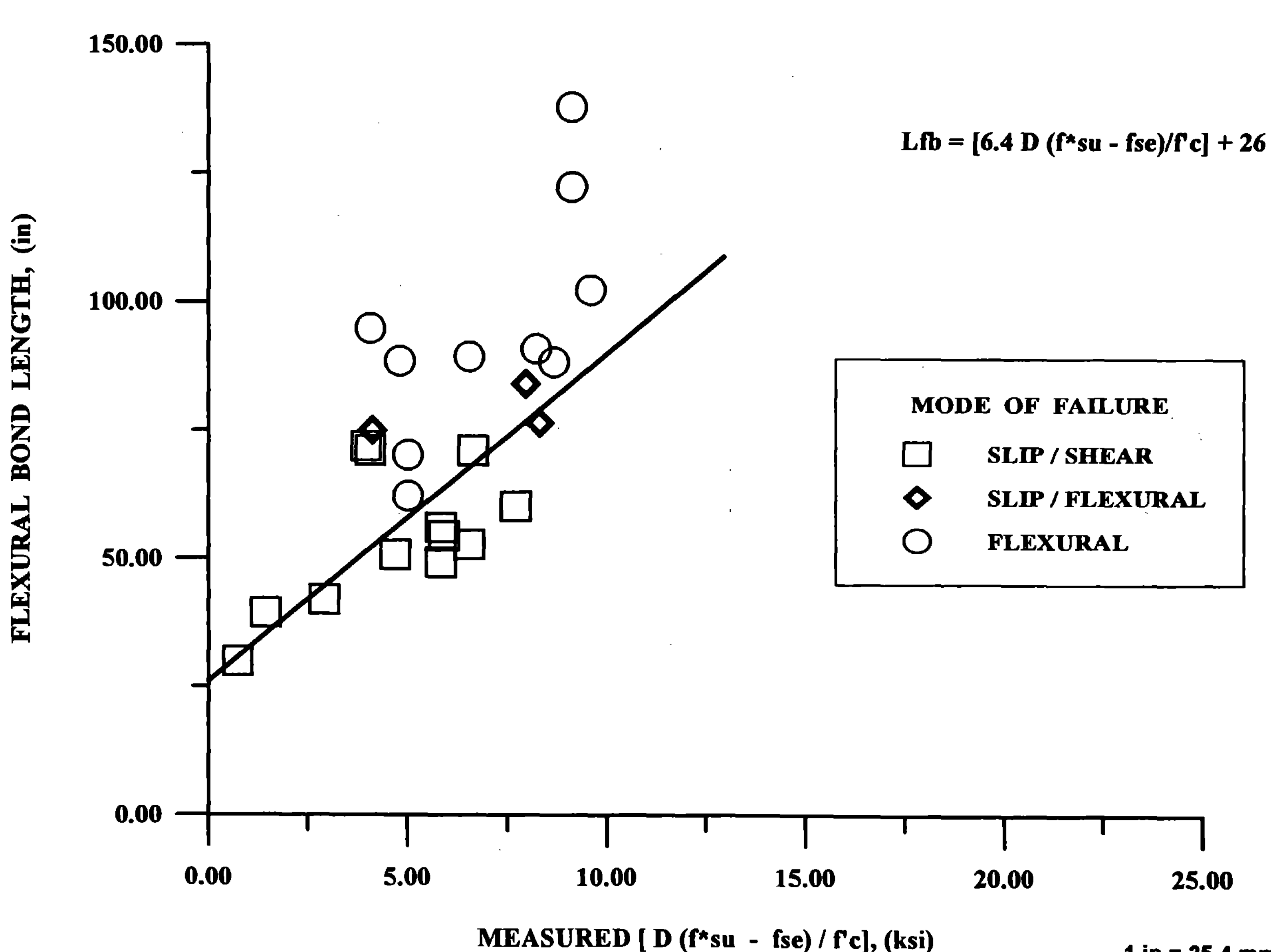
The TableCurve 2D program produced a list of possible equations that were ranked by their degree-of-freedom (DOF)-adjusted coefficient of determination. This DOF-adjusted coefficient of determination is a measurement of the goodness of fit of the data with a given equation.(22) The FHWA researchers could then compare the best-fit line equations using the DOF-adjusted coefficient of determination and select the best equation. The combination of variables and best-fit equation that had the highest DOF-adjusted coefficient of determination and was also understandable and usable was then chosen as the best obtainable flexural bond length equation.

The measured (or actual) values of the variables listed above were used in the analysis process just described. The best-fit equation that was chosen as the flexural bond length equation is shown below:

$$L_{fb} = \frac{6.4\,(f^{*}_{su} - f_{se})\,(D)}{f'_{c}} + 26 \qquad (8)$$

where L_{fb} is the flexural bond length.

Figure 8 shows a graph of the measured values of flexural bond length versus the expression $(f^{*}_{su} - f_{se})D/f'_{c}$. The expression $(f^{*}_{su} - f_{se})D/f'_{c}$ also contains the measured (actual) values of the parameters in it. Equation (8) is also plotted in this figure. It should be noted that Equation (8) is a best-fit curve; it is not a conservative equation nor does it contain any factor of safety. It is simply a reasonable mean for the measured data from the FHWA Phase II beams.



1 in = 25.4 mm
1 ksi = 6.89 MPa

Figure 8. Relationship between measured values of D(f*su - fse)/f'c and flexural bond length for FHWA Phase II beams only.

Because designers would only have design values of parameters and not have the actual values of the parameters while they were doing their design, the FHWA researchers plotted Equation (8) on a graph of measured values of transfer length versus the expression $(f^*_{su} - f_{se})D/f'_c$, which contained design values of those parameters. This is shown in figure 9. Figure 9 illustrates that Equation (8) is not necessarily the best-fit curve for this set of data. However, it is a conservative curve for the design values. FHWA researchers decided to adopt Equation (8) as the proposed flexural bond length expression pending correlation with data from other studies.

NEW DEVELOPMENT LENGTH EXPRESSION

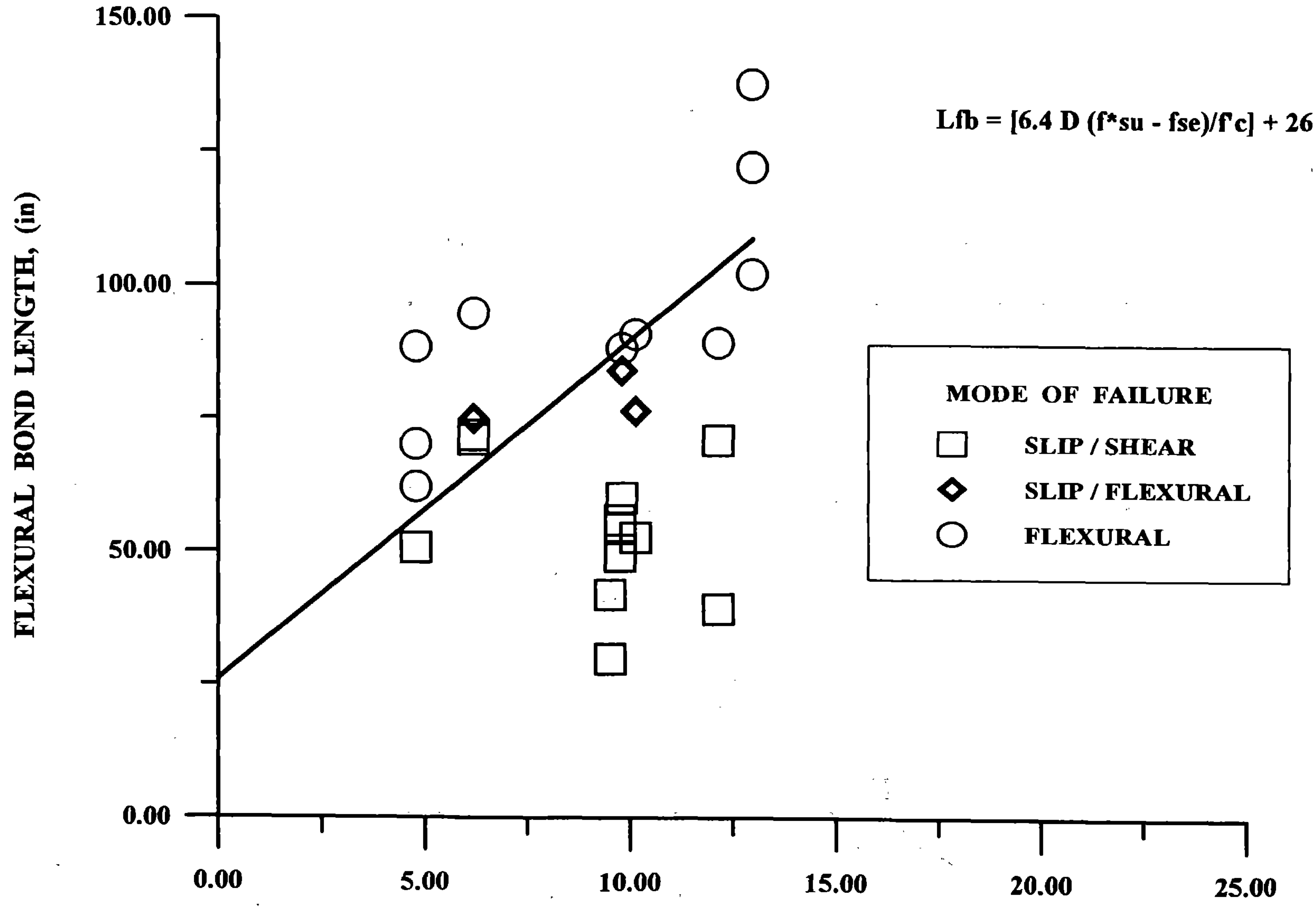
The development length is the sum of the transfer length and the flexural bond length. Therefore, the new development length expression is the sum of Equation (7) and Equation (8), and is shown below:

$$L_d = \left[\frac{4\, f_{pt}\, D}{f'_c} - 21\right] + \left[\frac{6.4\,(f^*_{su} - f_{se})(D)}{f'_c} + 26\right] \qquad (9)$$

The first term of this equation represents the transfer length, while the second term represents the flexural bond length. Equation (9) is shown graphically with the FHWA beam data in figure 10. Note that the measured values for the parameters are used in figure 10.

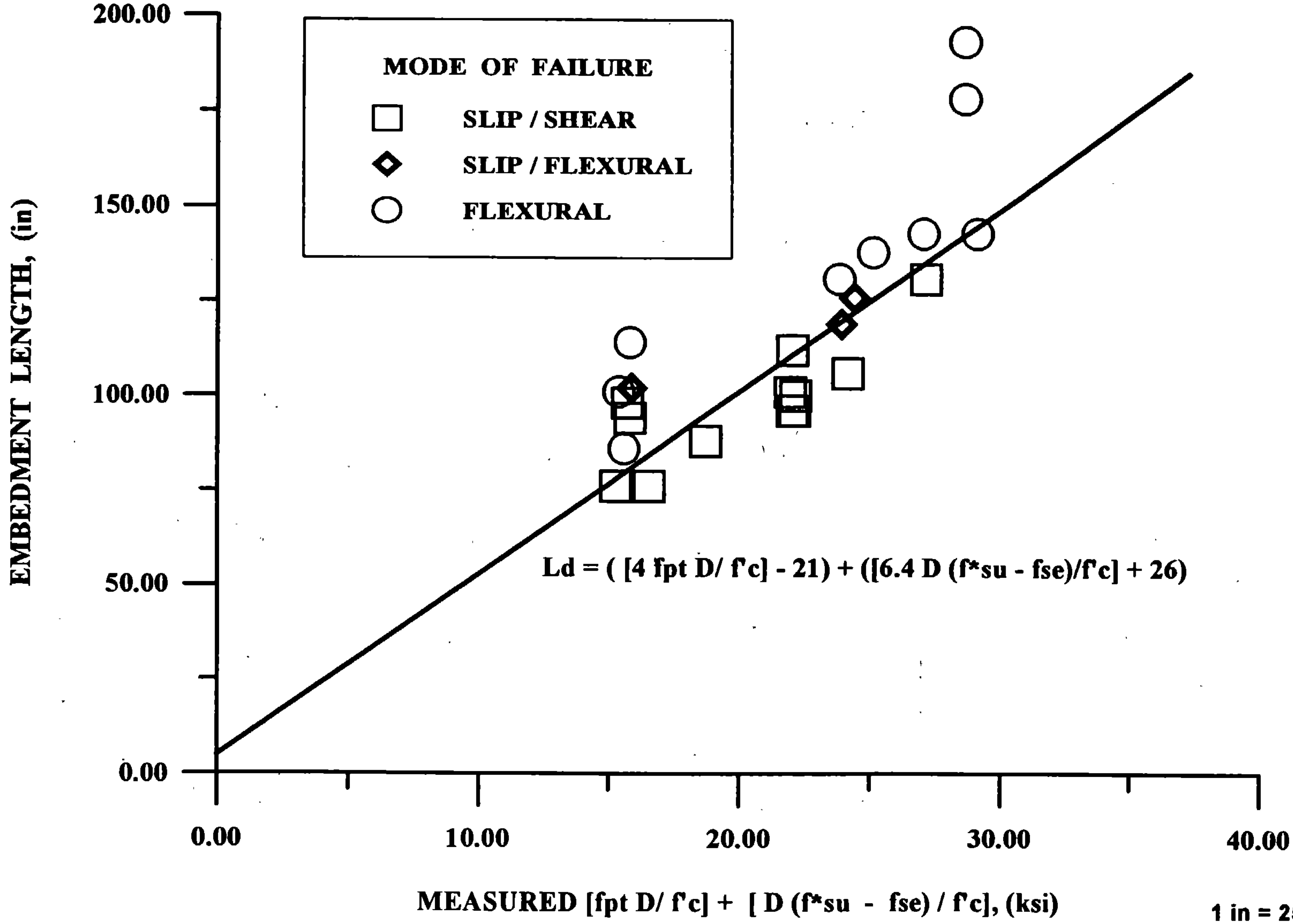
In this figure, flexural, slip/shear, and combination flexural/slip failures are shown. Although no failure of an actual bridge beam is desirable, if a beam failure is unavoidable, the engineer prefers a warning so that loss of life can be prevented. The flexural failures are the failures that provide warning signs (such as vertical cracks and excessive deflection); these are the "good" failures. The slip/shear failures occur without warning and are the failures that should be prevented. The combination flexural/slip failures are failures in which flexure and slip occur at exactly the same time and indicate that the embedment length being tested is the actual development length for that beam.

In figure 10, all slip failures should fall below the curve. This would indicate that the embedment length (or development length) calculated (or provided) is greater than that which would cause a slip failure. Flexural failures can fall above or below the curve because they are the "good" failure types. Figure 10 shows that the curve is adequate for ensuring against a slip failure, but it does not provide a safety margin.



1 in = 25.4 mm
1 ksi = 6.89 MPa

Figure 9. Relationship between design values of D(f*su - fse)/f'c and flexural bond length for FHWA Phase II beams only.



1 in = 25.4 mm
1 ksi = 6.89 MPa

Figure 10. Relationship between measured values of fpt D/ f'c + (D(f*su - fse)/f'c) and embedment length for FHWA Phase II beams only.

Figure 11 shows similar data to figure 10, except that in figure 11, the design values for the parameters are used. Similar conclusions can be drawn for figure 11 as were drawn for figure 10.

The FHWA researchers decided to adopt Equation (9) as the proposed development length expression pending correlation with data from other studies. The correlation of this expression with data from other studies will be discussed in chapter 4.

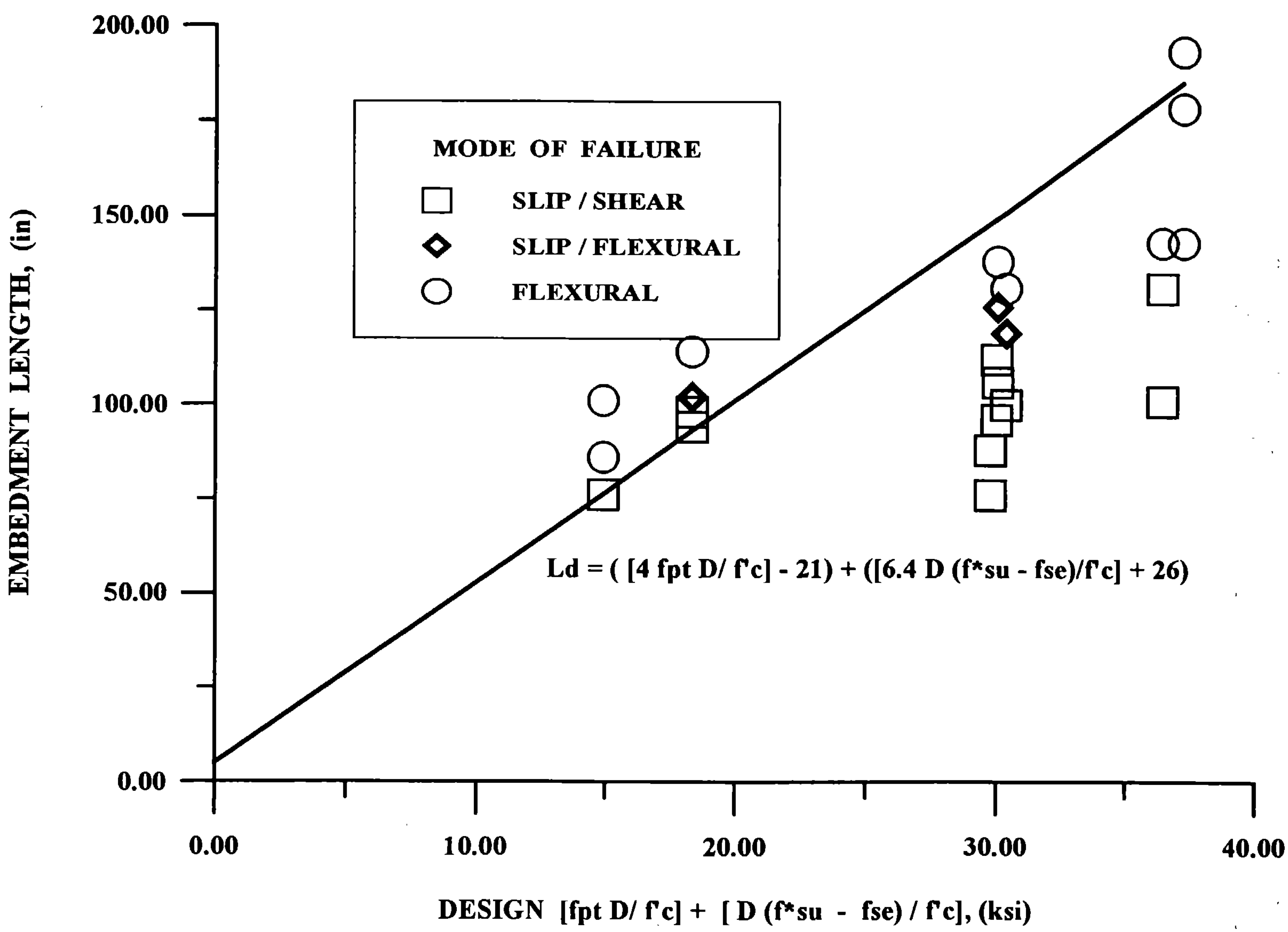


Figure 11. Relationship between design values of fpt D/ f'c + D(f*su - fse)/f'c and embedment length for FHWA Phase II beams only.

1 in = 25.4 mm
1 ksi = 6.89 MPa

CHAPTER 4: CORRELATION OF NEW DEVELOPMENT LENGTH EQUATION WITH OTHER RESEARCH RESULTS

The new expressions for transfer length, flexural bond length, and development length (Equations (7), (8), and (9) from chapter 3) correlated well with the FHWA Phase II beam data. The correlation of the new equations with the FHWA beam data was a necessary, but not sufficient, criterion for acceptance of the new equations. The results of other research studies also had to correlate well with the new equations. Therefore, the FHWA researchers needed to find all of the research results on the topic of bond of pretensioned strands in concrete and ascertain which of those research studies could be correlated with the equations.

For the new equations to correlate with the other studies, the other studies had to use similar materials and had to have key measurements taken. If the appropriate components were missing, the data were not correlated with the new equations for transfer length, flexural bond length, and development length. Once researchers determined which studies had the appropriate components and were eligible to correlate with the new equations, they began the actual correlation process. This chapter will describe that process.

BREADTH OF ALL STUDIES

FHWA undertook a worldwide literature search to find all of the research that had been conducted on the topic of bond of pretensioned members. A total of 62 studies were discovered on this topic. For some of the studies, the transfer and development length experimentation was the sole objective of the study; for others, transfer and development length experimentation was an important constituent of a broader study objective.

A list of all of the studies to date on the topic of bond of pretensioned members is provided in Appendix B. The dates listed in that table are the dates when the main publication (report or article) that emanated from the study was published.

Prior to the FHWA memorandum in 1988, which included the years 1949 to 1988, there had only been 20 research studies undertaken on this topic. Once the FHWA memorandum was issued, a flood of studies ensued. From the period of 1988-1994, there were 21 studies undertaken. The Buckner study was completed and his new equation was proposed in 1994. From 1994 to the present, there have been 21 additional studies undertaken or ongoing. These numbers indicated that an incredible amount of research had occurred on this topic in the last decade—42 studies to date. The review and correlation of these studies presented a formidable challenge to the FHWA researchers.

DATA REQUIREMENTS FOR CORRELATION OF OTHER RESEARCH RESULTS WITH NEW EQUATION

To ensure that correct comparisons were being made (i.e., comparing apples to apples and not apples to oranges), the FHWA researchers set up data requirements for the studies. Each study had to meet (conform to) these requirements to be eligible to participate in the correlation process. These requirements focused on the materials used in the study, as well as the type of measurements made in the study. If a study met these requirements, then its data were used in the correlation process; if a study did not meet these requirements, then its data were not used in the correlation process. Note that some of the studies listed in table 8 were theoretical studies, meaning that no new data were introduced, but a new equation or theory of bond was presented. These theoretical studies were examined and their theory considered where appropriate.

Data requirements or criteria were established in the following categories:

- Concrete.
- Prestressing Strand.
- Structural Member Types.
- Measurements:
  - Transfer Length Measurements.
  - Development Length Measurements.

The specifics of the criteria established for these items are described in the following sections:

Concrete

There was no restriction placed on the concrete compressive strength; concrete of all compressive strengths were considered. The only restriction on the concrete was that it had to be normal-weight concrete—lightweight concrete was not included in this comparison/correlation.

Prestressing Strand

The criteria for the prestressing strand follow:

- The prestressing strand used in the members needed to be uncoated strand; no epoxy-coated strand was considered.
- The strands could be either Grade 250 or Grade 270 strands—their guaranteed ultimate tensile strengths were either 1722 MPa or 1860 MPa (250 ksi or 270 ksi).
- Only low-relaxation strand was considered because that is what is commonly available now; stress-relieved strands were not considered for the correlation.
- The strands needed to be fully-bonded for the full length of the member; debonded strands were beyond the scope of this study.
- The sizes of strands that were included were diameters of 9.5 mm, 12.7 mm, 12.7 mm Special, and 15.2 mm (3/8 in, 0.5 in, 0.5 in Special, and 0.6 in).
- The strands could be arranged in any strand pattern, including draped strands.

Structural Member Types

Both full-size beams and rectangular specimens were considered. The full-size beam types considered were:

- AASHTO Standard I-Beams.
- State-Specific I-Beams.
- Bulb-Tees.
- Box Beams.

- T-Beams.
- Solid Rectangular Beams.

These full-size beams were considered both with and without composite decks. The cross-sections of the full-size beam types considered are shown in figures 12 and 13. Rectangular specimens of all dimensions were also considered. Pretensioned concrete decks and sub-deck panels were not considered as they were beyond the scope of the current study.

Because there was limited data available on transfer and development lengths of pretensioned piles, Dr. Beshah and Dr. Gagarin (of Starodoub, working for Construction Technology Laboratories, Inc. under a contract with FHWA) conducted a theoretical analysis of two different pile sections. The pile sections considered in the theoretical study are shown in figure 13. One of the sections was a 610-mm- (24-in-) square solid-section pile, and the other was a 762-mm- (30-in-) square pile with a centralized 483-mm (19-in) void.

Measurements

Because some of the studies only measured transfer length, while others measured transfer and development length (and a few only measured development length), separate criteria were set up for transfer length and for development length.

The criteria for transfer length data required that actual concrete strains be measured and that the transfer length be determined from those strain measurements. Transfer lengths that were calculated from other measurements (such as end slips), and were not determined from strain plots, were not considered.

The preferred method of determining the transfer lengths from the strain plots was the 95-percent plateau method. Other methods of determining transfer length, such as the 100-percent plateau method or the slope-intercept method, were allowed. However, if a method other than the 95-percent method was used, then the actual strain data had to be recorded and available for use so that the FHWA researchers could re-plot the actual strain data and determine the 95-percent plateau transfer lengths from it. Although this was a time-consuming process for the FHWA researchers, it ensured that correct transfer length values were being compared.

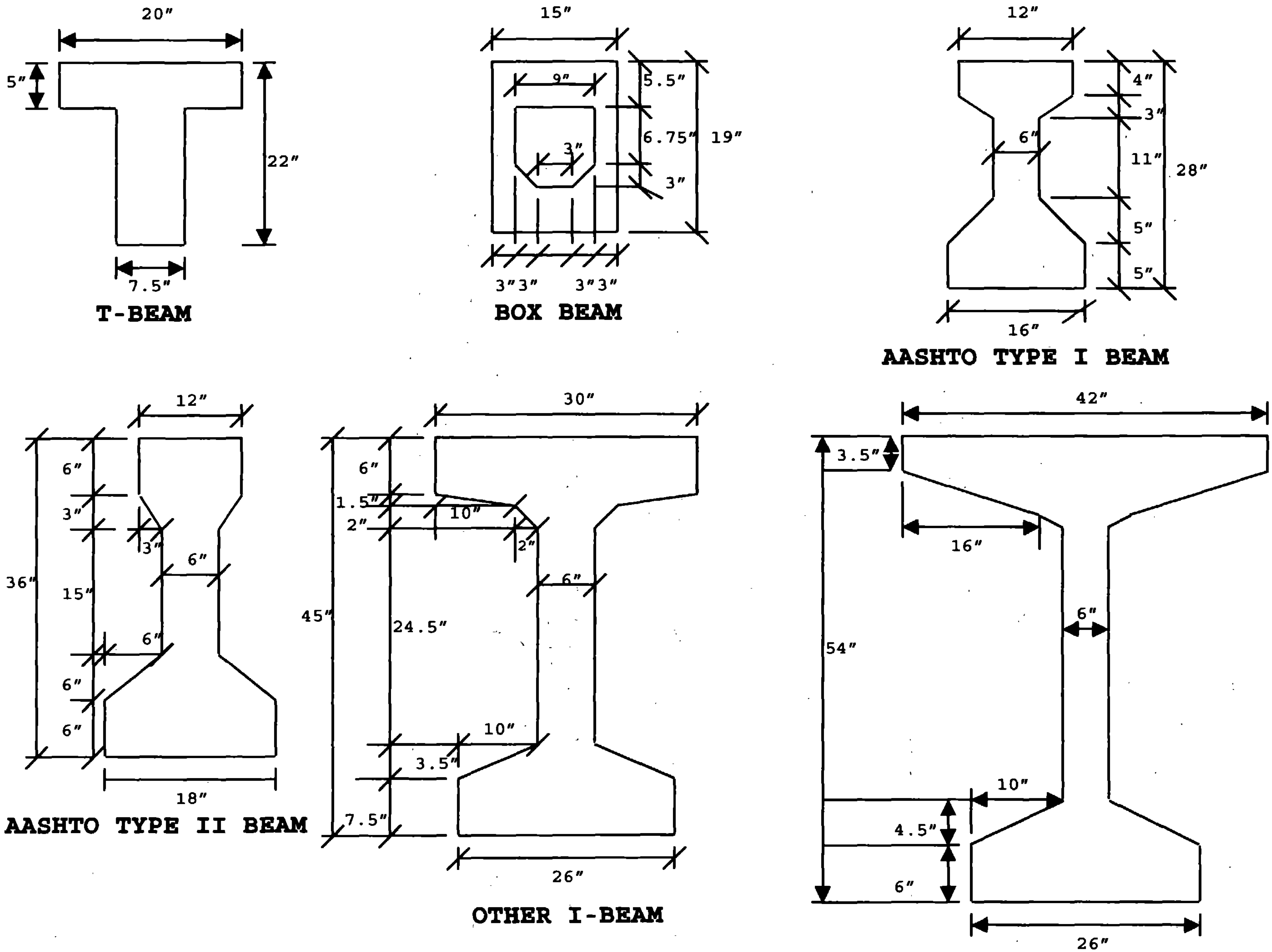


Figure 12. Cross-sections of full-size beams and piles. 1 in = 25.4 mm

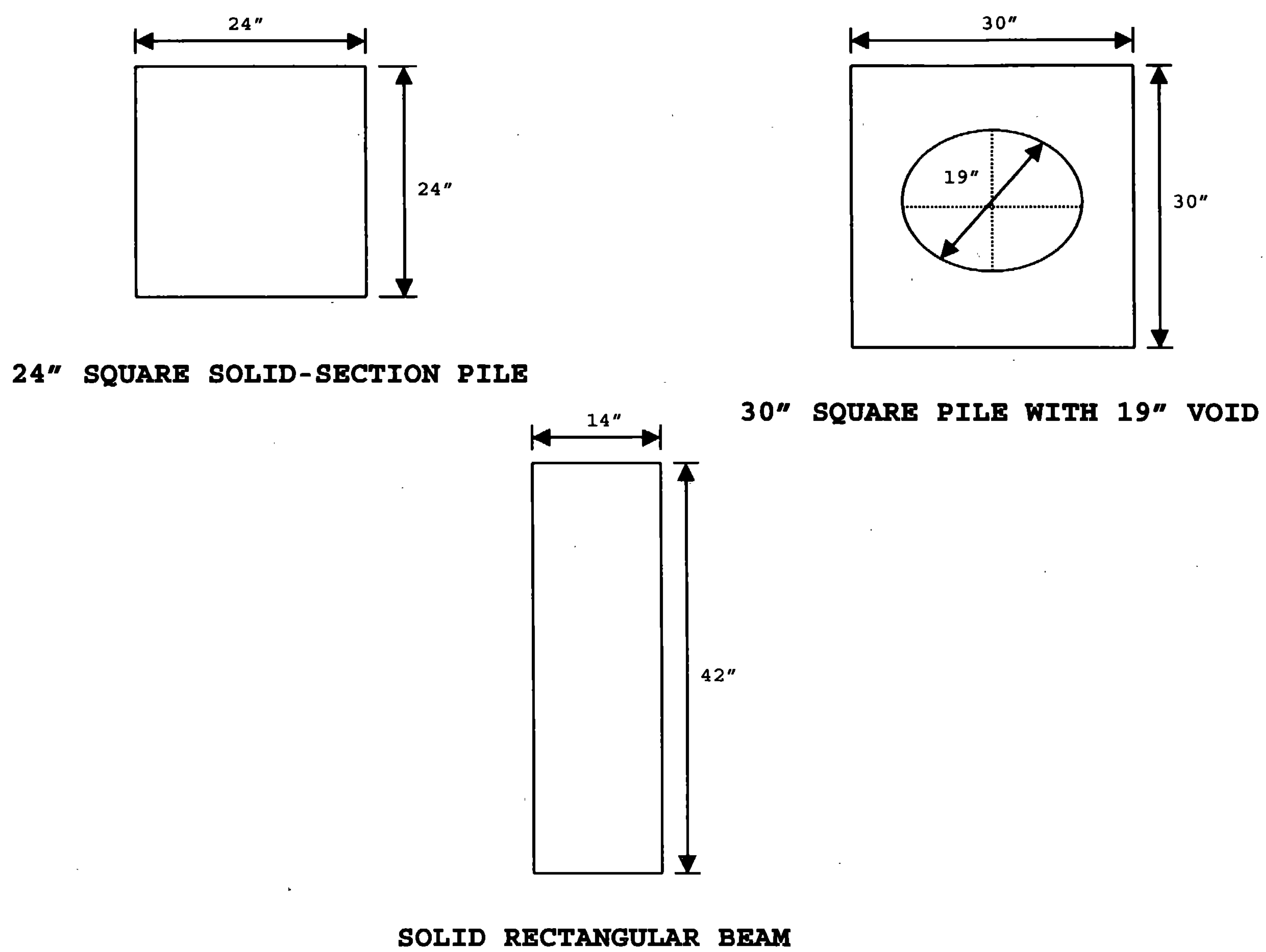


Figure 13. Additional cross-sections of full-size beams and piles. 1 in= 25.4 mm

In one instance, other researchers had determined transfer lengths by the 100-percent plateau method, and FHWA researchers established a correlation between the transfer length determined by the 100-percent method and the transfer lengths determined by the 95-percent method, so that all of the data did not have to be re-plotted for that study.

The criteria for the development length tests focused on certain measurements that had to be made during the test. The end slip of the strands had to be measured during the test so that the researchers could numerically determine if any of the strands had slipped. Tests that included only visual estimations of strand end slips and not physical end slip measurements were not considered. End slips during development length testing were typically very small numbers (on the order of 0.025 to 0.25 mm [0.001 to 0.01 in]) and were hard to determine visually. Therefore, they needed to be measured.

Additional requirements for development length testing included the measurement of deflection and/or concrete surface strain during the test. If either deflection or concrete surface strain (or both) were measured, then a strain-compatibility analysis could be conducted that would provide the needed parameters for use in the correlation of the new equations.

The development length testing could be conducted with either one-point or two-point loading; either one was acceptable for use in the correlation.

OTHER STUDIES USED IN THE CORRELATION WITH THE NEW EQUATIONS

All 62 of the bond-related studies were examined in detail to see whether their study focus, instrumentation, and data met the criteria described in the previous section. As was previously noted, these studies were on a variety of topics related to the bond of pretensioned strands. For some of the studies, the transfer and development length experimentation was the sole objective of the study; for others, transfer and development length experimentation was an important constituent of a broader study objective.

If their study focus, instrumentation, and data met the criteria described previously, then that study was included in the correlation. If their study focus, instrumentation, and data did not meet the criteria, then the study results were not included in the correlation. Also, if the study focus, instrumentation, and data met the criteria, but some data were not readily available, that data had to be given to FHWA researchers by a certain date to be included in the correlation process. If the data were not received by the cut-off date, then the study results were not included in the correlation.

FHWA researchers determined that 16 of the other studies, in addition to the FHWA Phase I (rectangular specimen) data, met the criteria and their data were provided to FHWA researchers by the cut-off date. Those studies were included in the correlation. Many fine studies were not included in the correlation. This does not imply that those studies were invalid. It simply means that those studies had objectives that were beyond the scope of this report, that measurements were made or materials used which were other than those described in the criteria above, or that the study data could not be supplied by the cut-off date.

Each of the studies included in the correlation are briefly described below. Tables 8 and 9 list the studies, the types of members used in the studies, and the number of data points from each study that were used in the correlation.

Auburn University

Auburn University researchers conducted a study for the Alaska Department of Transportation and Public Facilities that focused on the effect of strand spacing on the development length of pretensioned strands. Experimentation was conducted on 559-mm- (22-in-) high T-beams that contained nine 12.7-mm- (0.5-in-) diameter strands. The spacing of the strands was either 44.4 mm or 50.8 mm (1.75 in or 2.0 in). The concrete compressive strength varied between normal-strength concrete (41.3 to 55.1 MPa [6 to 8 ksi]) and high-strength concrete (68.9 to 82.7 MPa [10 to 12 ksi]). Both transfer and development length were determined for the 14 T-beams.(23-24)

University of Colorado at Boulder

Researchers at the University of Colorado at Boulder tested 15.2-mm- (0.6-in-) diameter strands contained in box beams to determine their transfer and development lengths for the Colorado Department of Transportation and FHWA. The objective of the study was to verify the adequacy of the members containing the 15.2-mm- (0.6-in-) diameter strands. If adequate, that size of strand would then be used in box girders in a high-performance concrete bridge project in Colorado.

Although three box beams were fabricated and made composite with a concrete deck, only two of the members (beam numbers 1 and 2) were used in the correlation with the new FHWA equation because of time constraints. The box beams were 381 mm (15 in) wide and 483 mm (19 in) high, with a 229-mm- (9-in-) wide void and a 70-mm- (2.75-in-) thick composite deck.

The concrete compressive strength of the beams at 28 days was 71.0 MPa (10,300 psi), and the concrete compressive strength of the deck at 7 days was 48.2 MPa (7000 psi). Both transfer length and development length were determined for these members.(25-26)

Table 8. Research studies with full-size beams included in the correlation with FHWA equations.

Organization	Beam Type	Strand Dia. (in)	Transfer Length Data Points Used	Development Length Data Points Used
Auburn University	T-Beam	0.5	26	22
Univ. of Colorado at Boulder	Box Beam	0.6	8	4
FHWA Phase II*	AASHTO II	0.5	36	15
		0.6	24	10
Florida DOT	AASHTO II	0.5	10	0
		0.5 Special	0	16
		0.6	5	5
Univ. of Minnesota	MN-45M I-Beam	0.6	4	4
Purdue University	AASHTO I	0.5	0	3
	Box Beam	0.5	0	1
Univ. of Texas at Austin for TxDOT	Texas 22" I-Beam	0.5	6	6
		0.6	12	9
Univ. of Texas at Austin—Louetta Road (Hoblitzell/ Buckner Beams)	Solid Rectangular	0.6	3	3
Univ. of Texas at Austin—San Angelo	Texas "C" I-Beam	0.6	16	8
Tulane Univ./CTL	Bulb-Tee	0.5	30	0
Virginia DOT	AASHTO II	0.6	8	4
Totals			188	110

*Used in original formulation of equations. 1 in = 25.4 mm

Table 9. Research studies with rectangular specimens included in the correlation with FHWA equations.

Organization	Strand Dia. (in)	Transfer Length Data Points Used	Development Length Data Points Used
FHWA Phase I	3/8	32	8
	0.5	32	8
	0.6	32	8
Preston and Janney	0.5	2	0
Louisiana State Univ./ Univ. of New Orleans	3/8	4	0
McGill University	0.5	28	0
	0.62	24	0
North Carolina State University	3/8	24	7
	0.5	48	10
	0.6	28	5
Univ. of Oklahoma	0.5	36	18
Univ. of South Florida/ Univ. of Illinois at Chicago	0.5 Special	5	0
Univ. of Texas at Austin for TxDOT	0.5	18	0
	0.6	16	0
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Totals		329	64

1 in = 25.4 mm

FHWA Phase I

Because the new equations were developed based on the data on full-size beams from the FHWA Phase II study, the data from the FHWA Phase I rectangular specimens were not included. Therefore, the data from the rectangular specimens containing uncoated strands were incorporated into the correlation. These rectangular specimens ranged in size from 102 mm x 102 mm x 3658 mm (4 in x 4 in x 12 ft) to 356 mm x 356 mm x 8534 mm (14 in x 14 in x 28 ft).

Three strand sizes were used in the following diameters: 9.5 mm, 12.7 mm, and 15.2 mm (3/8 in, 0.5 in, and 0.6 in), and the specimens contained one or four strands. The concrete compressive strength of the rectangular specimens at 28 days was between 34.4 MPa and 44.8 MPa (5000 psi and 6500 psi). The FHWA researchers determined transfer length at many ages and development lengths for these members.[3,15-16]

Florida Department of Transportation

Researchers at the Florida Department of Transportation's Structural Laboratory in Tallahassee, FL, conducted experimentation on pretensioned strands in full-size AASHTO Type II girders.

The girders were 914 mm (36 in) high and were 12.5 m (41 ft) long. Each one had a composite deck that was 203 mm (8 in) thick and 1067 mm (42 in) wide. Thirty-three girders were fabricated for this study, and they contained either 12.7-mm-, 12.7-mm Special, or 15.2-mm- (0.5-in-, 0.5-in Special, or 0.6-in-) diameter strands. The study examined the effects of the amount (percentage) of debonded strands and the amount of shear reinforcement on the transfer and development lengths of the strands. The design concrete compressive strength at 28 days was 41.3 MPa (6000 psi) for the girders and the deck.

The transfer and development lengths were determined for all of the members. Sixteen of the girders, all fully-bonded, were used in the correlation with the FHWA equations, but not all of the data were available for all of the girders by the cut-off date. Therefore, the transfer lengths were used for the "A1-00" girders, the development lengths were used for the "B0-00" and "B1-00" girders, and the transfer and development lengths were used for the "C1-00" girders.[27-28]

Preston and Janney

This was the first study to report on the bond of Grade 270 strands. The members consisted of six rectangular specimens, each containing one 12.7-mm- (0.5-in-) diameter strand. Two of the specimens contained Grade 270 stress-relieved strands that had a clean and bright surface condition. These two specimens were included in the correlation before it was discovered that the strands were stress-relieved. Despite the fact that they were stress-relieved, they were kept in the study to allow researchers to see their effect on the data.

The rectangular specimens measured 88.9 mm x 108 mm x 2.44 m (3.5 in x 4.25 in x 8 ft). The design compressive strength of the concrete at 28 days was 34.4 MPa (5000 psi). Only transfer lengths were determined for these specimens (specimens #2 and #6), and these transfer lengths were used in the correlation.[29]

Louisiana State University and the University of New Orleans

Researchers from Louisiana State University and the University of New Orleans conducted this research study for the Precast/Prestressed Concrete Institute to find a standard test for determining bond characteristics of prestressing strand. As part of this research, four rectangular pretensioned concrete specimens were constructed using a single 9.5-mm- (3/8-in-) diameter, Grade 270, low-relaxation strand in each specimen; for two of the specimens, the strand was uncoated, and for two of the specimens, the strand was epoxy-coated. The rectangular specimens measured 89 mm x 89 mm (3.5 in x 3.5 in) in cross-section, and 2.44 m (8 ft) in length.

Only transfer lengths were determined for the specimens; there was no development length testing. The transfer lengths for the rectangular specimens containing uncoated strands were used in the correlation.[30]

McGill University

This research study was conducted by McGill University with funding by the Natural Sciences and Engineering Research Council of Canada and was completed under the Networks of Centres of Excellence program.

The main objective of the study was to examine the impact that the high compressive strength of the concrete has on transfer and development lengths. The main variables were the size (diameter) of the strands (9.5 mm, 12.7 mm, and 15.7 mm [3/8 in, 0.5 in, and 0.62 in]), and the 28-day compressive strength of the concrete (31 to 89 MPa [4500 to 12,900 psi]).

Canadian-manufactured strand was used in the study and not U.S.-manufactured strand. Note that the largest size of strand had a diameter of 15.7 mm (0.62 in), and was not the more common American size of 15.2 mm (0.60 in).

The smallest-size strands were stress-relieved strands; members containing this strand were not used in the correlation. The other two larger-size strands were low-relaxation strands, and members containing these strands were used in the correlation.

Twenty-two rectangular specimens were fabricated as part of the study; their sizes ranged from 100 mm x 200 mm (3.9 in x 7.9 in) to 200 mm x 250 mm (7.9 in x 9.8 in). Transfer lengths at transfer and at 21 days, as well as development lengths, were determined for the specimens. FHWA researchers did not receive the development length data by the cut-off date, but did have access to the transfer length data. Therefore, only the transfer lengths for the 14 specimens containing the low-relaxation strands were used in the correlation with the new FHWA equations.[31]

University of Minnesota

A study on the applicability of using high-strength concrete to build full-size bridge girders took place at the University of Minnesota. This research consisted of fabricating and testing two full-size Minnesota 45M I-girders with composite decks. These girders were 1.14 m (45 in) high and 40.5 m (132 ft-9 in) long. Each girder contained 46 uncoated 15.2-mm- (0.6-in-) diameter strands, spaced at 50.8 mm (2 in). The concrete compressive strengths of these girders at 28 days varied between 77 and 83 MPa (11,100 and 12,100 psi). Transfer lengths, cambers, and prestress losses were determined for these girders, and then fatigue testing ensued. After fatigue testing, the girders were tested to failure to determine their ultimate and shear capacities. The transfer and development lengths for both girders were used in the correlation.[(32-34)]

North Carolina State University

A total of 60 rectangular specimens were constructed as part of the North Carolina State University study for the North Carolina Department of Transportation and the FHWA. The principal objective of the study was to evaluate the bond of various diameters and surface conditions of epoxy-coated prestressing strands. To provide a comparison, many members were also constructed with uncoated strands. Thirty-two of the 60 specimens contained epoxy-coated strands, while 28 of them contained uncoated strands. The strand diameters varied between 9.5 mm, 12.7 mm, and 15.2 mm (3/8 in, 0.5 in, and 0.6 in), while the compressive strength of the concrete at 28 days ranged from 31.9 to 59.2 MPa (4630 to 8590 psi). Transfer lengths were determined at six different time intervals, and development length tests were conducted both with and without fatigue testing. Elevated temperature tests were also performed on some of the specimens. Transfer and development lengths for 27 of the specimens with uncoated strands were used in the correlation.[(4,35)]

University of Oklahoma

Researchers at the University of Oklahoma conducted research for the Precast/Prestressed Concrete Institute to investigate a standardized method for measuring bond performance of prestressing strands. Their research consisted of using strands from three different anonymous strand manufacturers in tensioned and untensioned pull-out tests and comparing these results to transfer lengths measured in rectangular specimens to see if a correlation existed.

The strand-surface condition from one of the manufacturers' strands was varied to demonstrate its effect on transfer and development lengths. The four different strand-surface conditions were:

- As-Received—Just as it was received from the strand manufacturer.
- Cleaned—Cleaned with a solvent.
- Weathered—Exposed to the environment for a given time period before being used.
- Lubricated—Lubricated with silane.

Seventeen rectangular specimens were constructed as part of this study. University of Oklahoma researchers performed transfer and development length experimentation with these specimens. The specimens were 152 mm (6 in) wide by 305 mm (12 in) high, and either 5.2 m (17 ft) or 7.3 m (24 ft) long. Each specimen contained two 12.7-mm- (0.5-in-) diameter strands.

The concrete compressive strength at 28 days ranged from 33.0 to 43.1 MPa (4790 to 6260 psi). The transfer and development lengths for 17 of the rectangular specimens (specified as AA, BA, CA, CC, CS, and CW specimens in that study) were used in the correlation with the new FHWA equation.[(20,36-37)]

Purdue University

A total of 10 precast, prestressed beams (8 I-beams and 2 box beams) were constructed and tested as part of the Purdue study for the Indiana Department of Transportation and the FHWA. Eight of these beams (six I-beams and two box beams) were made into four sets of two continuous beams, complete with a composite deck cast on top of them. The I-beams were standard AASHTO Type I pretensioned concrete I-beams, while the box beams were standard Indiana State Type CB-27 box beams (686 mm [27 in] high with a 470-mm- [18.5-in-] high void). The other two I-beams were stand-alone beams (i.e., not made continuous). For each of the five pairs of beams, one of the beams contained fully-bonded 12.7-mm- (0.5-in-) diameter strands, while the other contained some debonded 12.7-mm- (0.5-in-) diameter strands. The design concrete compressive strength at 28 days was 41.3 MPa (6000 psi) for the beams.

The main objective of the study was to evaluate the effects of strand debonding on the flexural and shear behavior of precast, pretensioned bridge beams made continuous with a cast-in-place slab and diaphragm. The four sets of beams were first tested compositely, and then broken apart. The outer ends of the beams were then tested individually for shear and bond. The two stand-alone beams were tested only for shear and bond. The shear and bond tests functioned as development length tests. No transfer lengths were determined. One of the sets of I-beams contained stress-relieved strands, while the other three sets of I-beams and the box beams contained low-relaxation strands. Only the test results from the three I-beams and one box beam containing fully-bonded, low-relaxation strands were used in the correlation with the new FHWA equations.[38-39]

University of South Florida and the University of Illinois at Chicago

The main objective of the study by the University of South Florida and the University of Illinois at Chicago for the Florida Department of Transportation was to compare the transfer lengths of fiberglass strands and steel strands in prestressed-concrete members.

A total of 12 rectangular specimens were constructed, and 5 of these contained a single 12.7-mm (0.5-in) Special diameter concentric strand that was uncoated. The 12.7-mm (0.5-in) Special diameter strand has a slightly larger cross-sectional area than the normal 12.7-mm- (0.5-in-) diameter strand—namely, 107.7 mm^2 (0.167 in^2) for the 12.7-mm (0.5-in) Special diameter strand compared to 98.7 mm^2 (0.153 in^2) for the 12.7-mm- (0.5-in-) diameter strand. The other seven rectangular specimens contained fiberglass strands. The rectangular specimens measured 152 mm x 102 mm x 2590 mm (6 in x 4 in x 102 in). The compressive strength of the concrete at 28 days ranged from 6500 psi to 7300 psi. Only transfer lengths were determined for the rectangular specimens; no development length testing was performed. The transfer length results for the five rectangular specimens containing the steel strands were used in the correlation.[40]

University of Texas at Austin for Texas DOT

This huge study, which was conducted by the University of Texas at Austin for Texas DOT and FHWA, consisted of constructing and testing 74 prestressed concrete members. Fifty of the members were rectangular specimens (containing 1, 3, or 5 strands), while 24 of the members were full-size AASHTO-type I-beams (containing anywhere from 4 to 24 strands). The main objective of the study was to develop design guidelines for transfer length, development length, and debonding for uncoated steel strands in pretensioned concrete beams.

The sizes of the rectangular specimens varied from 127 mm (5 in) high x 102 mm (4 in) wide to 330 mm (13 in) high x 127 mm (5 in) wide, and contained either 12.7-mm- (0.5-in-) or 15.2-mm- (0.6-in-) diameter strands spaced primarily at 50.8 mm (2 in). Transfer length was determined for these rectangular specimens.

The full-size, AASHTO-type I-girders ranged in size from 559 mm (22 in) high to 1016 mm (40 in) high, both with and without composite decks. The debonding length, pattern, and cut-off (abrupt versus gradual) were varied, and beams with fully-bonded strands were included for comparison. The beams contained multiple 12.7-mm- (0.5-in-) or 15.2-mm- (0.6-in-) diameter strands, spaced at 50.8 mm (2 in). All of the beams underwent static loading to failure, but some of the beams experienced fatigue loading prior to the static loading to failure. Transfer and development lengths were determined for all of the girders.

Only data from members with fully-bonded strands were used for the correlation with the new FHWA equations. Transfer length results from 17 rectangular specimens, and transfer and development length results from 10 full-size, AASHTO-type I-girders were used in the correlation. (See references 41 through 47.)

University of Texas at Austin—Louetta Road Overpass Project

In July 1993, FHWA and the Texas Department of Transportation, in conjunction with the University of Texas at Austin, signed a cooperative agreement for the Louetta Road Overpass High Performance Concrete (HPC) Bridge Project. This agreement included, among other items, two designs of the two parallel bridges that made up the Louetta Road Overpass: one design with normal concrete and the other design with HPC. The design for the HPC option involved the use of 15.2-mm- (0.6-in-) diameter pretensioned strands at a spacing of 50 mm (1.97 in) in a new beam shape—the U-beam.

At the time of the design, the FHWA 1988 memorandum was in effect, and this placed a moratorium on the use of 15.2-mm- (0.6-in-) diameter strands and restricted the center-to-center strand spacing to four times the diameter of the strand. For 15.2-mm- (0.6-in-) diameter strands, this spacing restriction would have resulted in a spacing of 61.0 mm (2.4 in). Therefore, research was needed to demonstrate that the 15.2-mm- (0.6-in-) diameter strands could be used successfully at 50-mm (1.97-in) spacing in a beam constructed with the same concrete that would be in the actual bridge beams.

Researchers first contemplated testing a U-beam containing the full complement of 15.2-mm- (0.6-in-) diameter strands. However, because of the capacity limitations of conventional structural laboratories (such as the Ferguson Laboratory at the University of Texas at Austin), a U-beam with the full complement of strands (more than 50 strands) could not be tested to failure. Therefore, a deep beam was designed that would simulate the flexural and bond behaviors of the U-beams. The deep beam chosen was a solid rectangular section and was called the Hoblitzell-Buckner beam, named after its designers, Jim Hoblitzell of FHWA and Dale Buckner of Virginia

Military Institute. Two of the Hoblitzell-Buckner beams were constructed at Texas Concrete in Houston, TX, and shipped to and tested at the University of Texas at Austin.

The beams measured 1067 mm (42 in) in height and were 356 mm (14 in) wide, and contained six 15.2-mm- (0.6-in-) diameter strands spaced at 50.8 mm (2 in) in one row at the bottoms of the sections. Three No. 9 steel reinforcing bars were also placed in the tops of the beams. The concrete's design compressive strengths were 41.1 MPa (6000 psi) at detensioning and 55.2 MPa (8000 psi) at 28 days. Actual concrete compressive strengths were 48.5 MPa (7040 psi) at detensioning, and ranged between 81.4 and 81.9 MPa (11,810 and 11,890 psi) at 28 days.

The concrete compressive strength of the beams had increased to just more than 90 MPa (13,000 psi) by the time that the beams underwent development length testing. Both transfer and development length tests were performed for each end of each beam. The results from these tests were included in the correlation with the new FHWA equations.[(48-49)]

University of Texas at Austin—San Angelo Bridge Project

Similar to the Louetta Road Overpass project, a second HPC bridge project was initiated in Texas for the San Angelo Bridge. The partners in this project were again FHWA and Texas DOT, in conjunction with the University of Texas at Austin. For this bridge, it was desired to use 15.2-mm- (0.6-in-) diameter strands at 50-mm (2-in) spacings in I-beams. The bridge consisted of two parallel structures, one that would primarily be designed and constructed with normal concrete and one that would primarily be designed and constructed with HPC.

Since this project took place while the 1988 FHWA memorandum was still in effect, the researchers were required to demonstrate that the 15.2-mm- (0.6-in-) diameter strands would work successfully at the reduced spacing in an I-beam constructed with concrete similar to that to be used in the actual bridge beams.

Two sets of two I-beams were fabricated for the research. One set of beams represented the normal concrete and one set represented the higher-strength HPC. All beams were Texas Type "C" beams, which were 1016 mm (40 in) high, with a 559-mm- (22-in-) wide bottom flange and a 356-mm- (14-in-) wide top flange. Each beam contained sixteen 15.2-mm- (0.6-in-) diameter strands in the bottom flange and four 15.2-mm- (0.6-in-) diameter strands in the top flange; all strands were spaced at 50.8 mm (2 in).

The normal concrete set of beams had an actual concrete compressive strength of 46.4 MPa (6740 psi) at 28 days, while the higher strength HPC set of beams had an actual concrete compressive strength of 88.0 MPa (12,770 psi) at 28 days. Each beam had a composite deck cast at the Ferguson Laboratory at the University of Texas at Austin. Transfer and development length tests were performed for each end of each beam. These results were used in the correlation with the new FHWA equations.[(49-50)]

Tulane University and Construction Technology Laboratories, Inc. (CTL)

Tulane University and Construction Technology Laboratories, Inc. (CTL) performed this study for the Louisiana Department of Transportation and Development. The prestressed concrete members they used were full-size Bulb-Tee girders and piles. The objective of the study was to evaluate and demonstrate the feasibility of using high-strength concrete in bridge members. Some of the girders were used in static tests to failure, while some underwent fatigue testing. Transfer lengths were determined for the girders used for the static tests, based on strains measured at release of prestress force, at 28 days, before and after the deck cast, and prior to the static test to failure. These transfer lengths were used in the correlation with the new FHWA equations.(51)

Virginia Department of Transportation

Virginia DOT and FHWA collaborated on an HPC bridge project called the Richlands Bridge. After designing the bridge using HPC, it was determined that it was most economical to use 15.2-mm- (0.6-in-) diameter strands at 50.8-mm (2-in) spacing in the high-strength HPC beams. However, similar to the two Texas projects, the FHWA 1988 memorandum was still in place; therefore, research was required to demonstrate that the 15.2-mm- (0.6-in-) diameter strands could be used successfully at the 50.8-mm (2-in) spacing in an I-beam made of concrete, which had the same mix design as the actual Richlands Bridge beams.

Two 9.4-m- (31-ft-) long AASHTO Type II pretensioned concrete I-beams with composite decks were fabricated and structural testing occurred at FHWA's Structures Laboratory at the Turner-Fairbank Highway Research Center in McLean, VA. The girders contained six straight 15.2-mm- (0.6-in-) diameter bottom strands, and two 15.2-mm- (0.6-in-) diameter draped strands, all spaced at 50.8 mm (2 in). The compressive strength of the concrete at release ranged from 55.0 MPa to 58.7 MPa (7980 psi to 8520 psi), and the concrete compressive strength at 28 days varied between 70.1 and 73.2 MPa (10,170 and 10,620 psi). The concrete compressive strength of the cast-in-place deck at 28 days was 81.2 MPa (11,790 psi). Transfer lengths were determined from strain measurements at release of the prestress force and immediately prior to the development length tests. Development length tests were conducted for all four ends of the beams. Both the transfer length and development length results were used in the correlation with the new FHWA equations.(52-54)

TRANSFER LENGTH AND DEVELOPMENT LENGTH ANALYSES

For all of the aforementioned studies, the data and results for a given study were recorded and determined according to the goals and procedures specific to that study. Those goals and procedures may have been different from the goals and procedures of FHWA's study. Therefore, to compare and correlate results among the studies, all of the data, including transfer length and development length results, had to be in similar forms.

Transfer Length Analysis

As was described in chapter 2, Buckner's study concluded that the recommended method for determining transfer lengths from measured strains was the 95-percent plateau method. This method was used to determine all of the transfer lengths for the FHWA Phase II (full-size members) study. However, not all the researchers who performed the other studies used this method to determine transfer lengths. Some studies determined transfer lengths using methods such as the slope-intercept method, the 100-percent plateau method, or the 95-percent plateau method. FHWA researchers had to recalculate the transfer lengths (from the other studies) that did not use the 95-percent plateau method so that they could accurately compare data from all the studies. This included the FHWA Phase I (rectangular specimens) study, which had been determined using the 100-percent plateau method.

FHWA researchers, to determine the transfer lengths by the 95-percent plateau method, used the following process. First, FHWA researchers gleaned concrete strain data from all of the reports and articles for the studies. (Frequently, the strain plots were available in an appendix of a report.) Next, the principal investigators for the research studies in question were contacted and were asked to provide electronic files of the concrete strain data. If the concrete strain plots or electronic files of the concrete strain data were available, then FHWA researchers calculated the 100-percent and 95-percent plateau values.

The 95-percent plateau values were physically plotted on the concrete strain graphs and the transfer lengths were then determined from the graphs. (In one instance, a within-study correlation was established between the 100-percent plateau transfer lengths and the 95-percent plateau transfer lengths. This correlation was used to calculate the remaining 95-percent transfer lengths for that study.) Although this process was time-consuming, it resulted in uniformly determined values of transfer length that could be correlated with one another.

Development Length Analysis

To correlate the development length results from other studies with the new FHWA development length equation, the parameters used directly in the new equation had to be obtained from the other studies' results. Specifically, the parameters fell into two categories: parameters that were provided in the study's associated literature, and parameters that had to be determined.

The parameters that were directly in the new equation were f_{pt}, f'_c, f^*_{su}, f_{se}, and D. If these five parameters were provided in the study's associated literature, then the FHWA researchers performed the correlation. However, if these five parameters were not directly provided in the study's associated literature, then those parameters had to be either obtained from the principal investigator, assumed, or calculated using structural analysis.

In general, if any of the five parameters were missing, FHWA researchers then contacted the principal investigator, if at all possible, and requested the information. For a variety of reasons, sometimes that information was available, sometimes it was not. If the information was not available, then FHWA researchers were required to assume reasonable values for the missing parameters or perform structural analysis, or both.

The strand diameter, D, was always provided in the literature, and the parameter f_{pt} was usually provided in the literature. If f_{pt} was not provided, then that parameter was assumed to equal 0.75 f'_s, (where f'_s is the ultimate stress of the prestressing steel), which is the maximum stress allowed prior to transfer for low-relaxation strands in pretensioned members, as stated in Article 9.15.1 of the AASHTO Specifications.[7]

FHWA researchers wanted both the design and actual values of f'_c to be able to correlate the data. Usually both of these f'c values were provided, but if only one was provided, then the other

f'_c value was assumed to equal the one that was provided. The parameters f_{se} and f^*_{su} were seldom provided, and these parameters were calculated using structural analyses.

FHWA researchers performed the same structural analyses that they performed for the FHWA Phase II (full-size beams) study, using the two computer programs developed by Dr. Beshah and Dr. Gagarin.

For the first program, inputs were required for concrete strength, modulus of elasticity, and modulus of rupture; properties of the strand; and loading and cross-section information for a given development length test. The load and deflection data from the test was then used in combination with the aforementioned input items in an iterative approach that yielded the effective prestress force, f_{se}.

The second program was used to determine f^*_{su}. For the second program, all of the inputs and values determined by the first program were used, as well as the measured value of the external concrete strain of the top fiber (compression fiber) at the load point, if it was available. As long as the measured value of the external concrete strain of the top fiber was available, then it was used with the other inputs in an iterative approach to determine the stress and strain of the bottom strands at failure of the member (f^*_{su} and ϵ_{su}, respectively).

If FHWA researchers did not receive data on the external concrete strain of the top fiber, then the second computer program would not work. Therefore, Dr. Beshah and Dr. Gagarin created a third computer program, which used the load and deflection data in a complex iterative approach that yielded f^*_{su} and ϵ_{su}. (Note that running the computer programs to determine the f_{se} and f^*_{su} values was a lengthy process that took approximately 3 hours per computer run.)

STATISTICAL COMPARISONS

In chapter 3, Equation (7) was presented as a best-fit equation for transfer length, and Equation (8) was presented as a best-fit equation for flexural bond length. When these two equations were combined, resulting in Equation (9), it was a best-fit line for overall development length. These equations were considered best fit with respect to the FHWA full-size girder data. Once the data were assembled from all of the other studies, including the FHWA rectangular specimen data, then FHWA researchers began determining whether these equations were representative of all of the data.

During this process of correlating Equations (7) through (9) with all of the data from other studies, a few other re-examinations and re-evaluations were conducted. The issue of unit weight as a possible parameter in Equations (7) through (9) was re-examined; the idea of using the area of strand rather than the diameter of strand as a parameter in the equations was re-evaluated; and the possibility of using log, natural log, semi-log, and natural semi-log as equation types was also evaluated.

FHWA researchers re-examined the unit weight as a potential parameter. They reviewed all of the other studies used in the correlation to find any measured values of unit weight. These values were scarce. The concrete mixes for each of the studies were then examined to see whether the researchers could determine the unit weight from the quantity and quality information on constituent mix materials.

Jon Mullarky, formerly of the National Ready Mix Concrete Association, examined the data and determined that accurate unit weight calculations could not be determined from the concrete-mix constituent information unless the specific gravity of the aggregates and the air content of the final mixture were provided. This information was not available for most of the studies, and values for these parameters varied too much for confident assumptions to be made. Researchers concluded that the information on unit weight was insufficient and, therefore, it could not be evaluated as a parameter for the equations.

FHWA researchers re-evaluated the idea of using the area of strand rather than the diameter of strand as a parameter in the equations using data from the other studies. They chose to re-evaluate the parameter because some states used 12.7-mm (0.5-in) Special diameter strand in their bridges. This size of strand has the same nominal diameter, but each of the seven wires comprising the Special strand is slightly larger than the wires in normal 12.7-mm- (0.5-in-) diameter strand. Therefore, the area of steel for the 12.7-mm (0.5-in) Special diameter strand is larger than it is for the normal 12.7-mm- (0.5-in-) diameter strand. The area of steel for the 12.7-mm (0.5-in) Special diameter strand is 107.7 mm^2 (0.167 in^2), while the area of steel for the normal 12.7-mm- (0.5-in-) diameter strand is 98.7 mm^2 (0.153 in^2).

While re-evaluating the idea of using the area of steel rather than the diameter of strand as a parameter in the equation, FHWA researchers also re-evaluated equation types that had logarithmic functions associated with them. The following equation types were re-examined with all of the data:

$$y = mx + b \tag{5}$$

$$\log_{10}(y) = m \log_{10}(x) + b \tag{10}$$

$$\ln(y) = m \ln(x) + b \tag{11}$$

$$y = m \log_{10}(x) + b \tag{12}$$

$$y = m \ln(x) + b \tag{13}$$

where $\log_{10}$ is the Base 10 logarithm and ln is the natural logarithm (logarithm to the Base e). For each of the above equations, researchers performed regression analyses on all the data to determine the constants m and b for two instances: (1) when diameter D was used as a parameter and (2) when the area of steel strand A^*_s was used as a parameter. Researchers then statistically analyzed the data to determine coefficients of correlation for each of these cases.

Coefficients of correlation are "... a measure of the strength of the linear relationship between the x and y variables."[21] The coefficients of correlation were compared, and FHWA researchers concluded that Equations (7), (8), and (9), using the diameter of the strand as a parameter, were the best overall choices for the transfer length, flexural bond length, and development length equations, respectively.

Even though it was determined that Equations (7) through (9) were the overall best-fit equations, these equations still only represented the mean of the data. They were not conservative equations. Therefore, the constants for these equations were adjusted so that the equations were conservative. They were no longer best-fit equations—they exceeded the requirement of a 95-percent confidence interval for the data. The revised transfer length equation is as follows:

$$L_t = \frac{4 f_{pt} D}{f'_c} - 5 \qquad (14)$$

The revised flexural bond length equation is as follows:

$$L_{fb} = \frac{6.4\,(f^*_{su} - f_{se})(D)}{f'_c} + 15 \qquad (15)$$

Equations (14) and (15) were then added together to provide the revised development length equation:

$$L_d = [\frac{4\, f_{pt}\, D}{f'_c} - 5\,] + [\frac{6.4\,(f^*_{su} - f_{se})(D)}{f'_c} + 15\,] \qquad (16)$$

During the review of these equations, there were relatively few data points encountered for members with a concrete compressive strength, f'_c, over 10,000 psi. Therefore, until more data becomes available, it was determined that **values of f'_c greater than 10,000 psi shall be taken as 10,000 psi for use in Equations (14), (15), and (16). Equations (14), (15), and (16) are English-unit equations. The values of the constants in Equations (14), (15), and (16) will change when the equations are converted for metric use.**

Figures 14 through 16 show all of the transfer length data, for all full-size beams and rectangular specimens. Along the y-axis, the actual measured transfer lengths from the various studies are plotted. Along the x-axis, the transfer lengths calculated by the AASHTO transfer length equation (see Equation (2)) for those members are plotted. The line drawn in the figures represents the cases in which the measured transfer lengths exactly equal the calculated transfer lengths using the AASHTO equation. Any points falling above this line are unconservative because the value that was measured was greater than the value that was predicted.

As discussed in chapter 1, the AASHTO transfer length equation was only meant to be an average expression, not a conservative one (see figure 14). In figure 14, the points are approximately divided, with approximately half of the points above the line and half below the line.

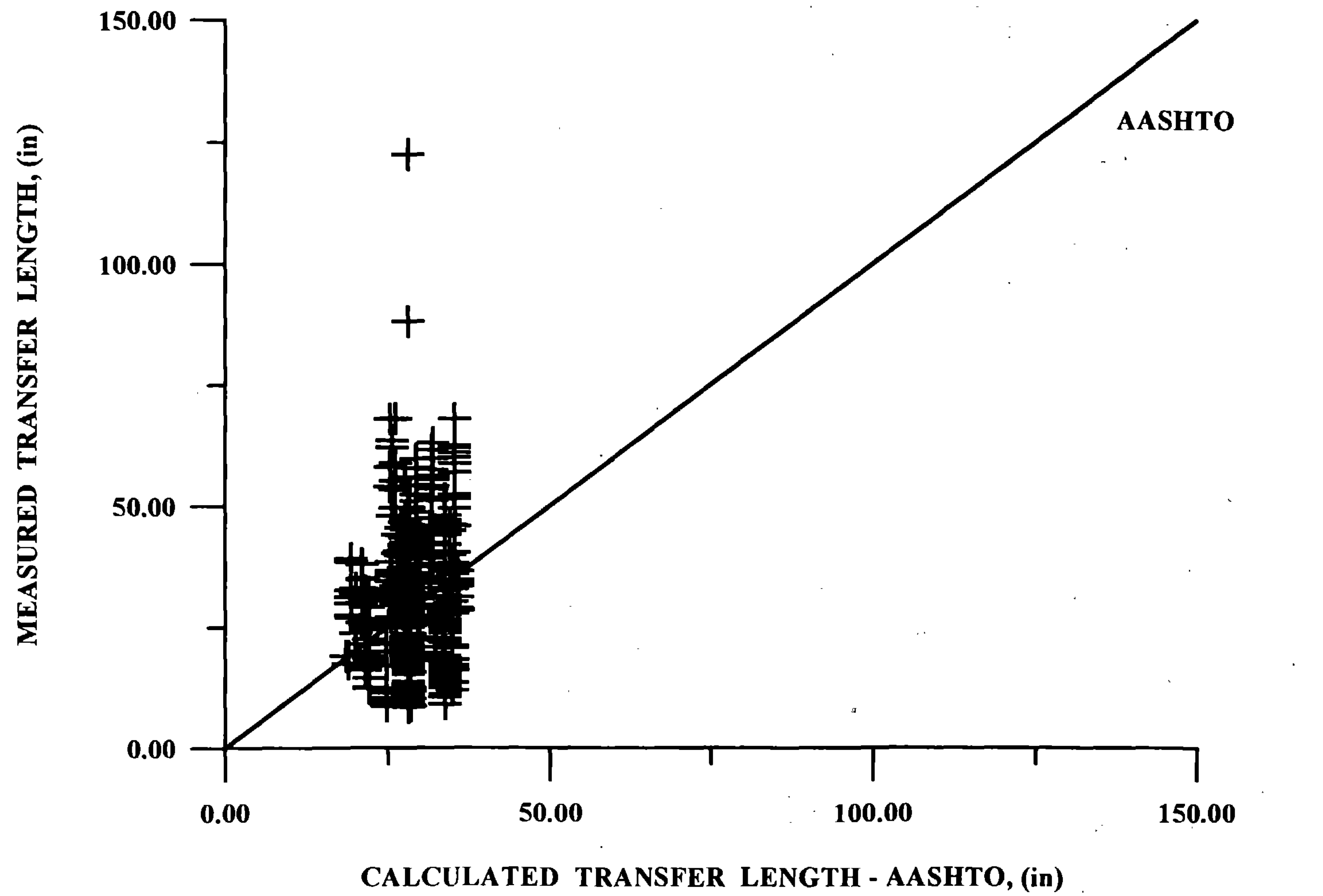
In figure 15, only the data for members with normal-strength concrete (f'_c between 34.4 and 48.2 MPa [5000 and 7000 psi]) were shown. This figure indicates that the current AASHTO equation is unconservative for predicting transfer lengths in this range of concrete strengths.

Data for members with high-strength concrete (f'c between 55.1 and 68.9 MPa [8000 and 10,000 psi]) were plotted in figure 16. The transfer lengths determined in the research studies involving the high-strength concrete were consistently shorter than transfer lengths in normal-strength concrete. Because those transfer lengths are consistently shorter, the AASHTO transfer length equation is conservative for members constructed with high-strength concrete.

Figures 17 through 19 compare all of the transfer length data to transfer length predictions by Buckner's transfer length equation, $f_{si}D/3$ (see Equation (3)). It is apparent from figure 17 that Buckner's transfer length equation is more an average expression than a conservative expression because a considerable number of points fall above the line.

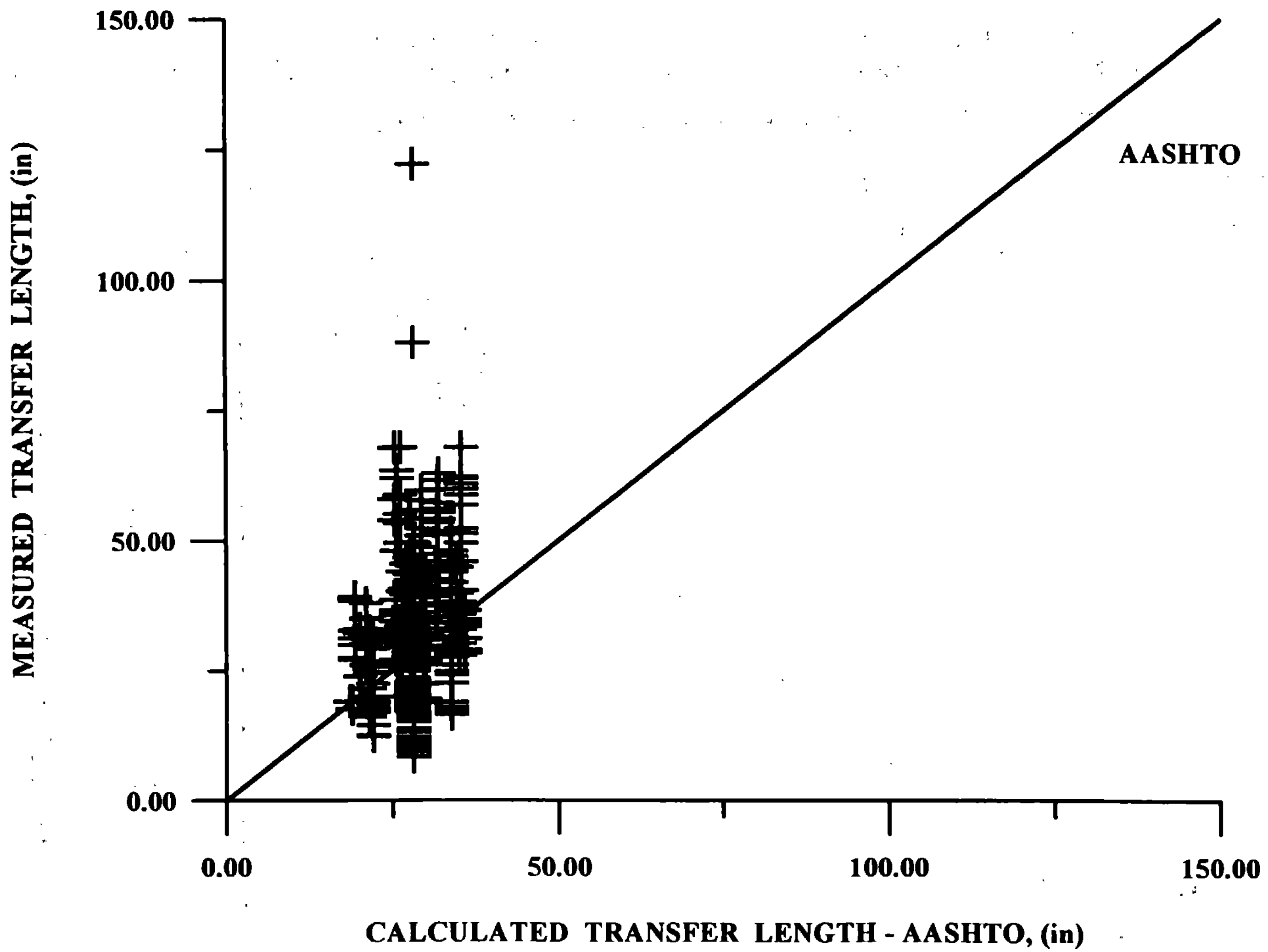
The same trend is seen in figure 18, which illustrates the data and the Buckner-predicted transfer lengths for all members constructed with normal-strength concrete. However, when only data and predicted values for transfer lengths in high-strength concrete members were plotted, the Buckner transfer length equation was conservative (see figure 19).

Figures 20 through 22 show the transfer length data and transfer length predictions using the FHWA transfer length equation (Equation (14)). Figure 20 shows only a few data points falling above the line, which indicates that the FHWA equation is conservative. The same conclusion was drawn from the data shown in figure 21, which is the graph of data and FHWA-predicted transfer lengths for members constructed with normal-strength concrete.



1 in = 25.4 mm

Figure 14. Comparison of measured transfer length values for all members with the AASHTO equation.



1 in = 25.4 mm

Figure 15. Comparison of measured transfer length values for members having concrete strengths between 34.4 and 48.2 MPa (5000 and 7000 psi) with the AASHTO equation.

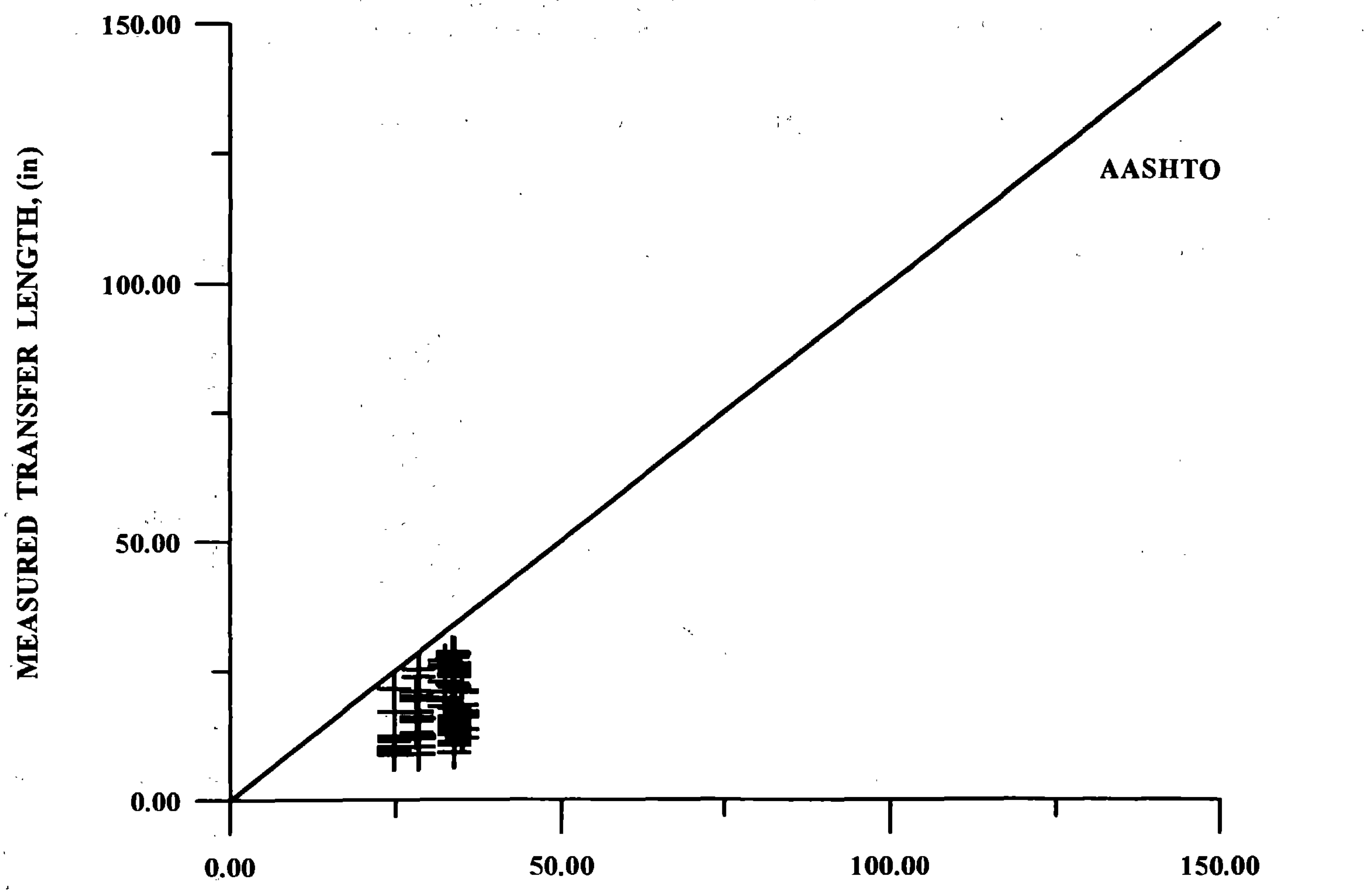


Figure 16. Comparison of measured transfer length values for members having concrete strengths between 55.1 and 68.9 MPa (8000 and 10,000 psi) with the AASHTO equation.

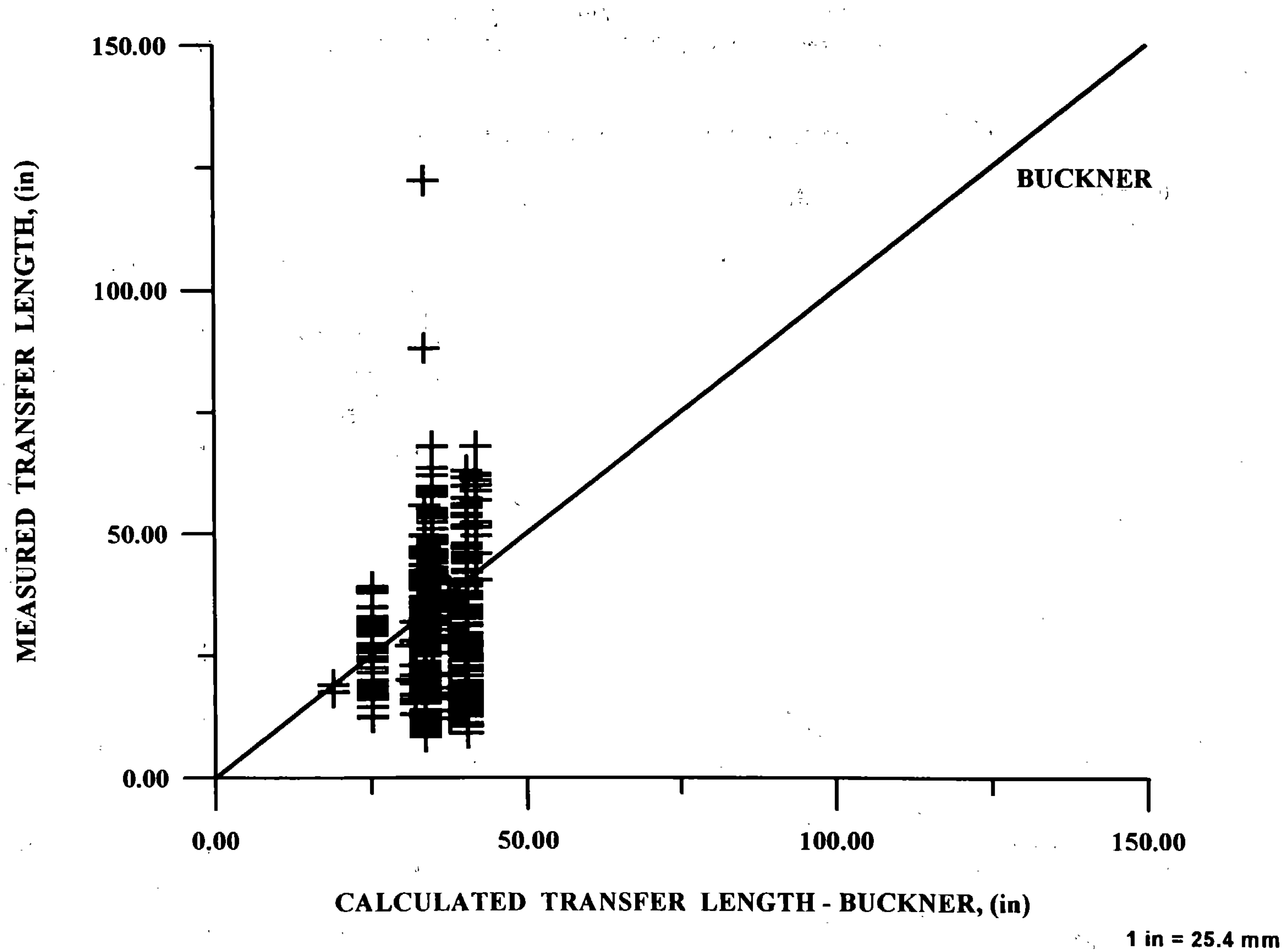


Figure 17. Comparison of measured transfer length values for all members with the Buckner equation.

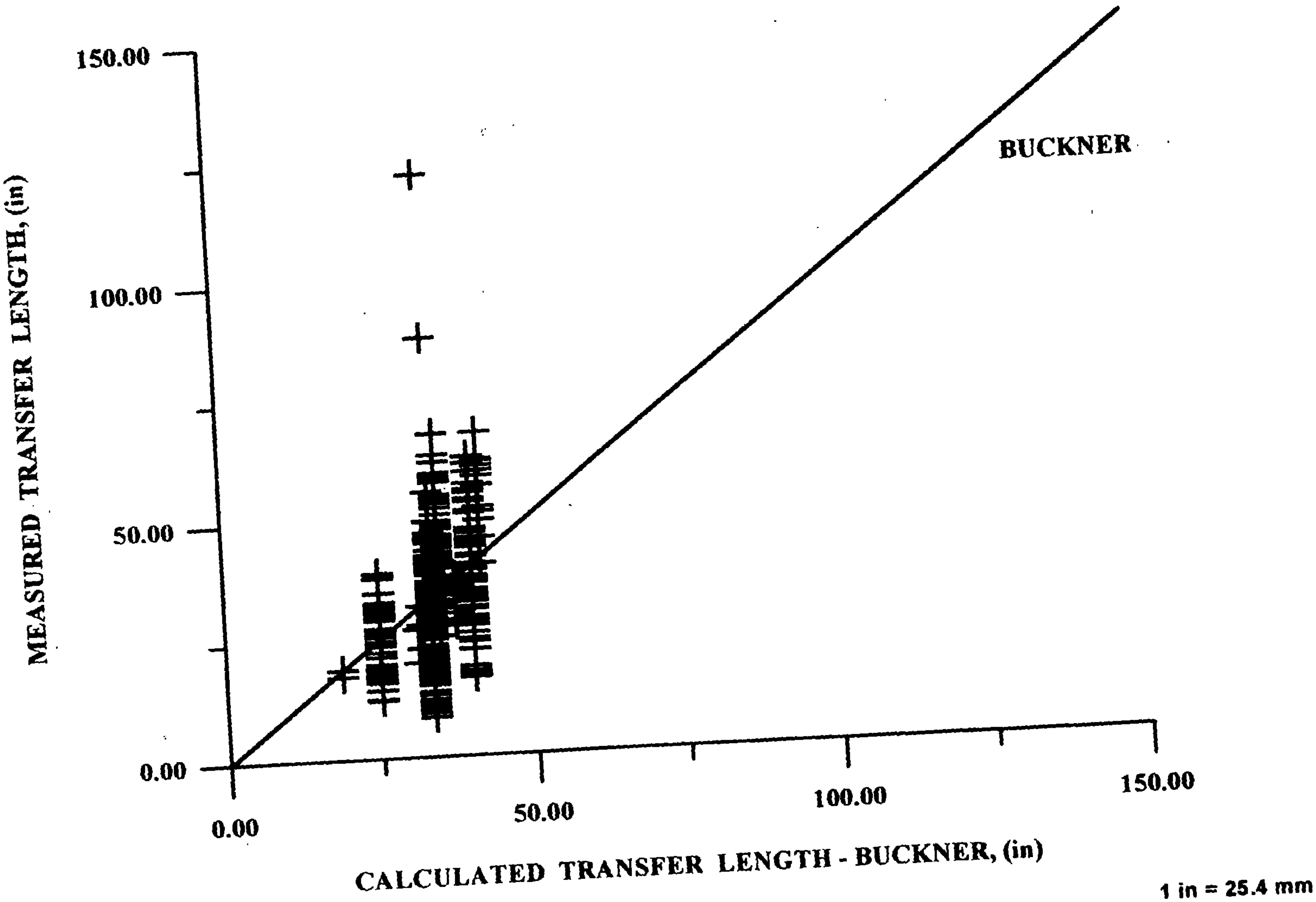


Figure 18. Comparison of measured transfer length values for members having concrete strengths between 34.4 ... (5000 and 7000 psi) with the Buckner equation.

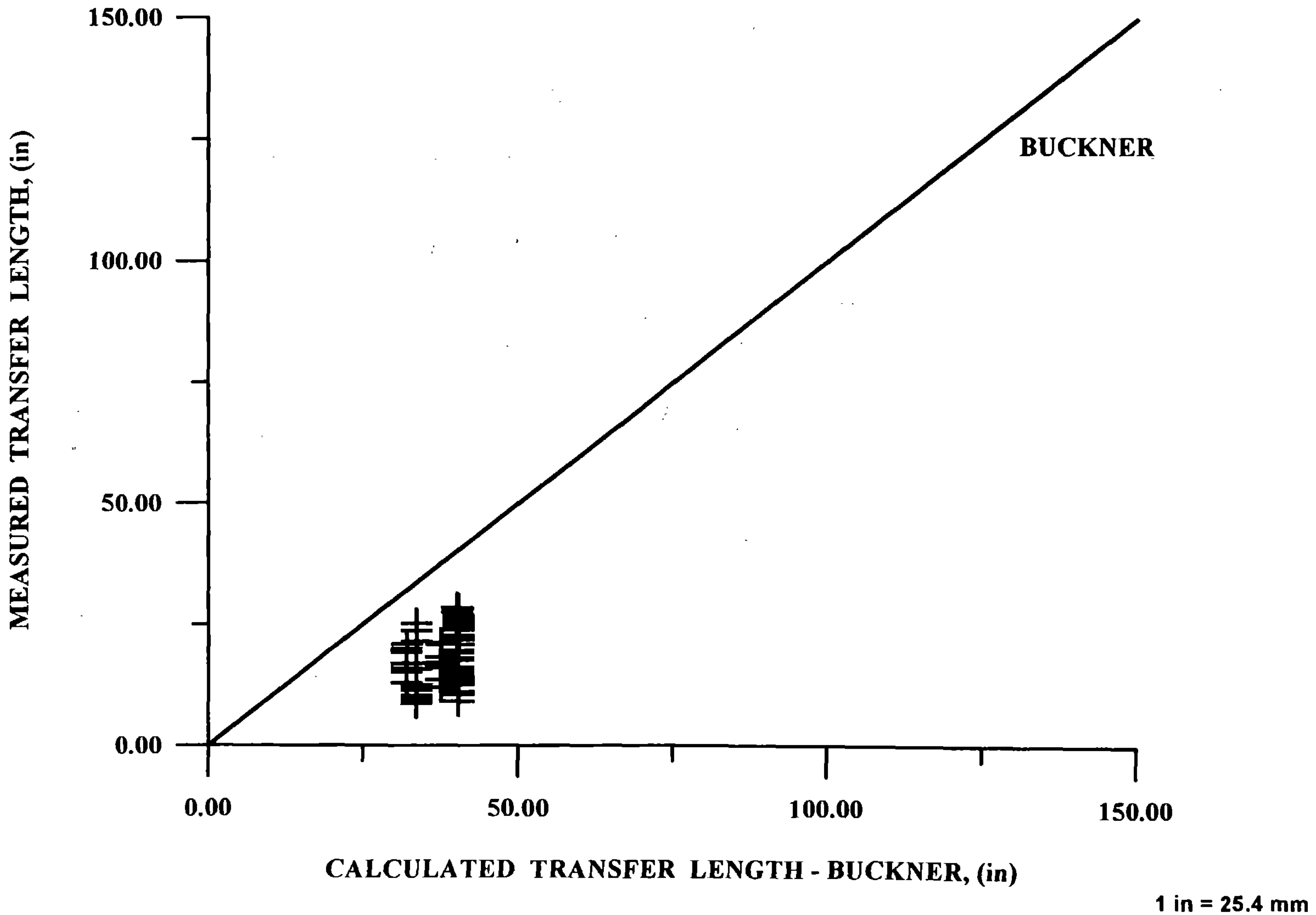
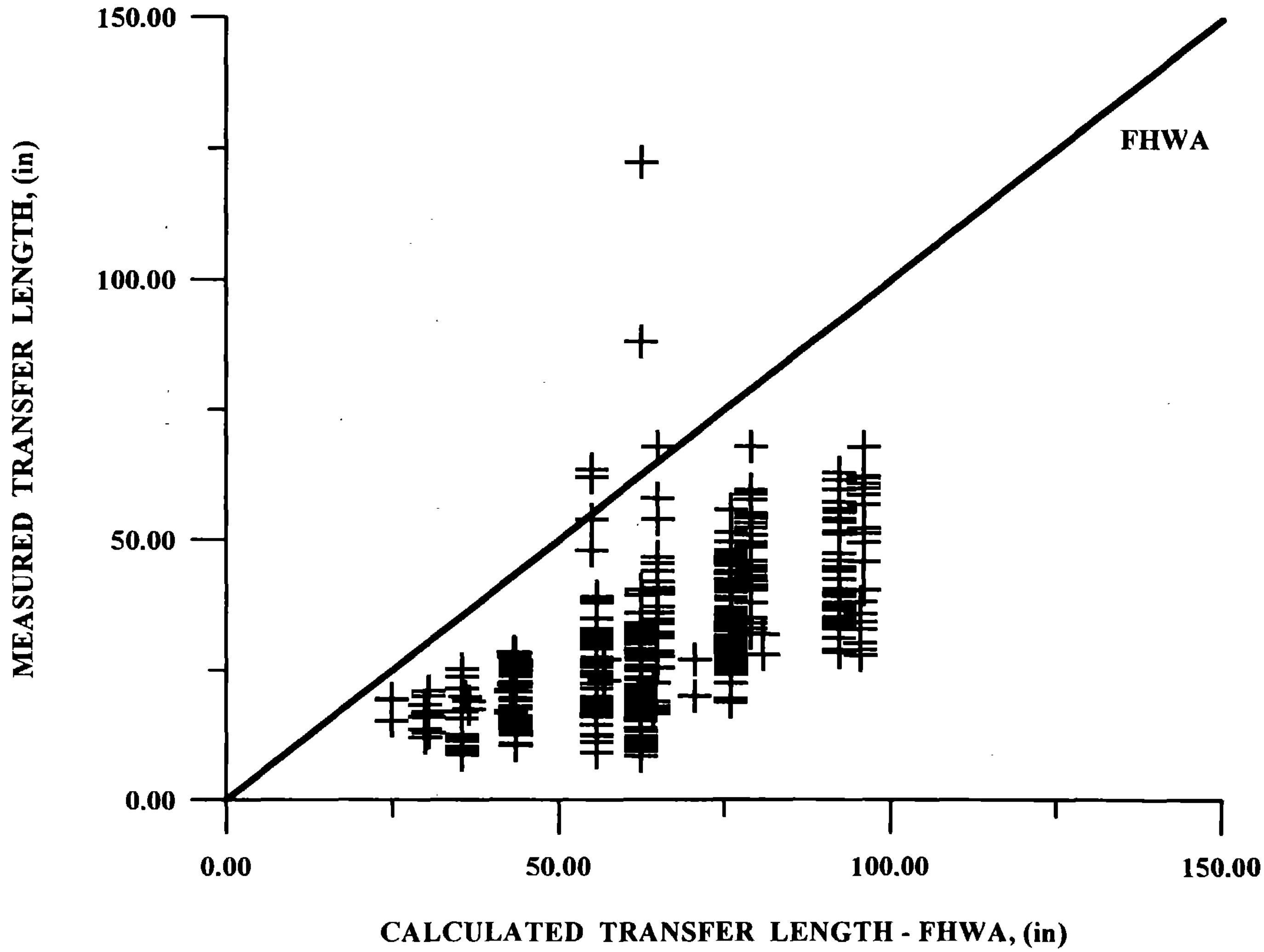
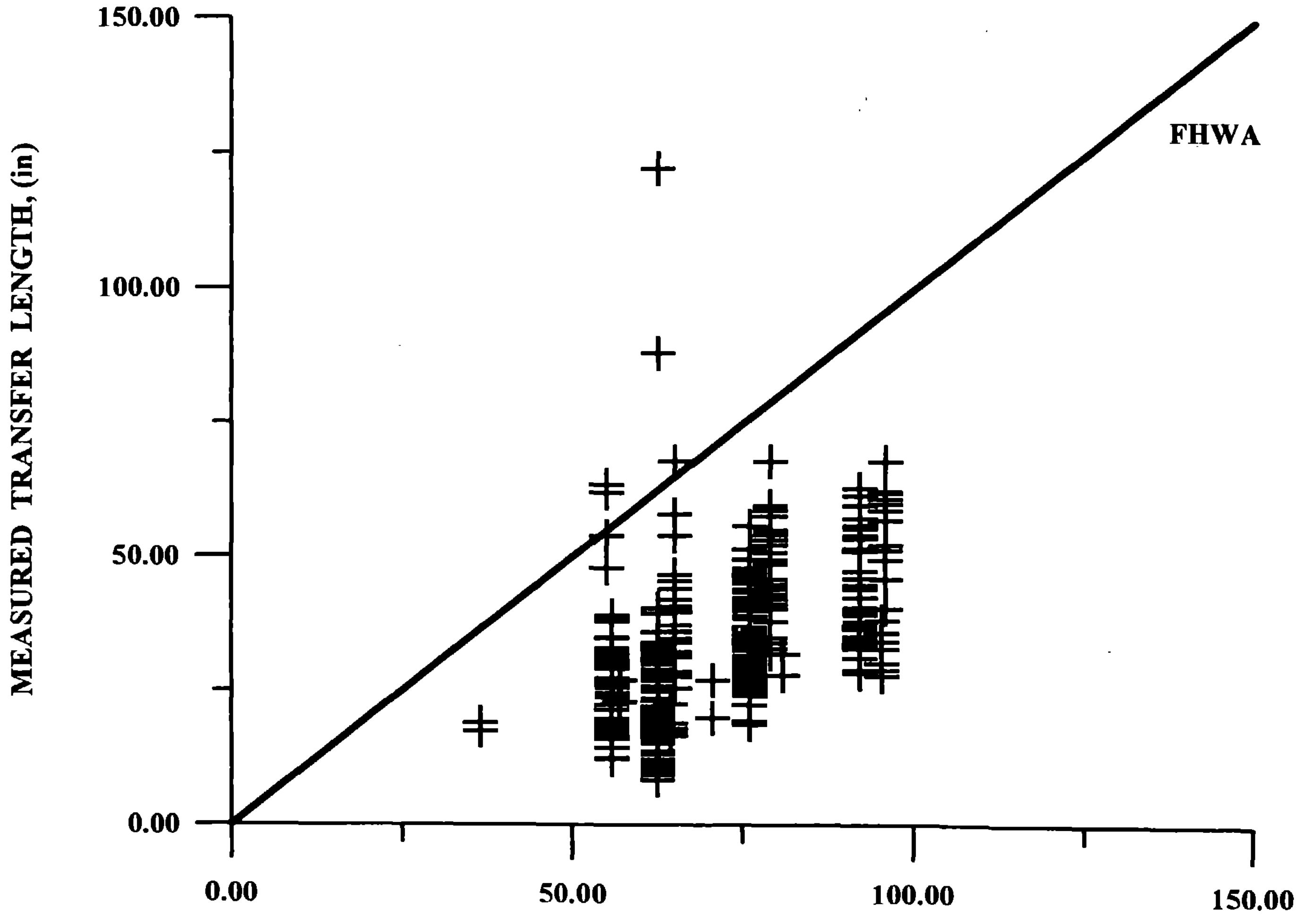


Figure 19. Comparison of measured transfer length values for members having concrete strengths between 55.1 and 68.9 MPa (8000 and 10,000 psi) with the Buckner equation.



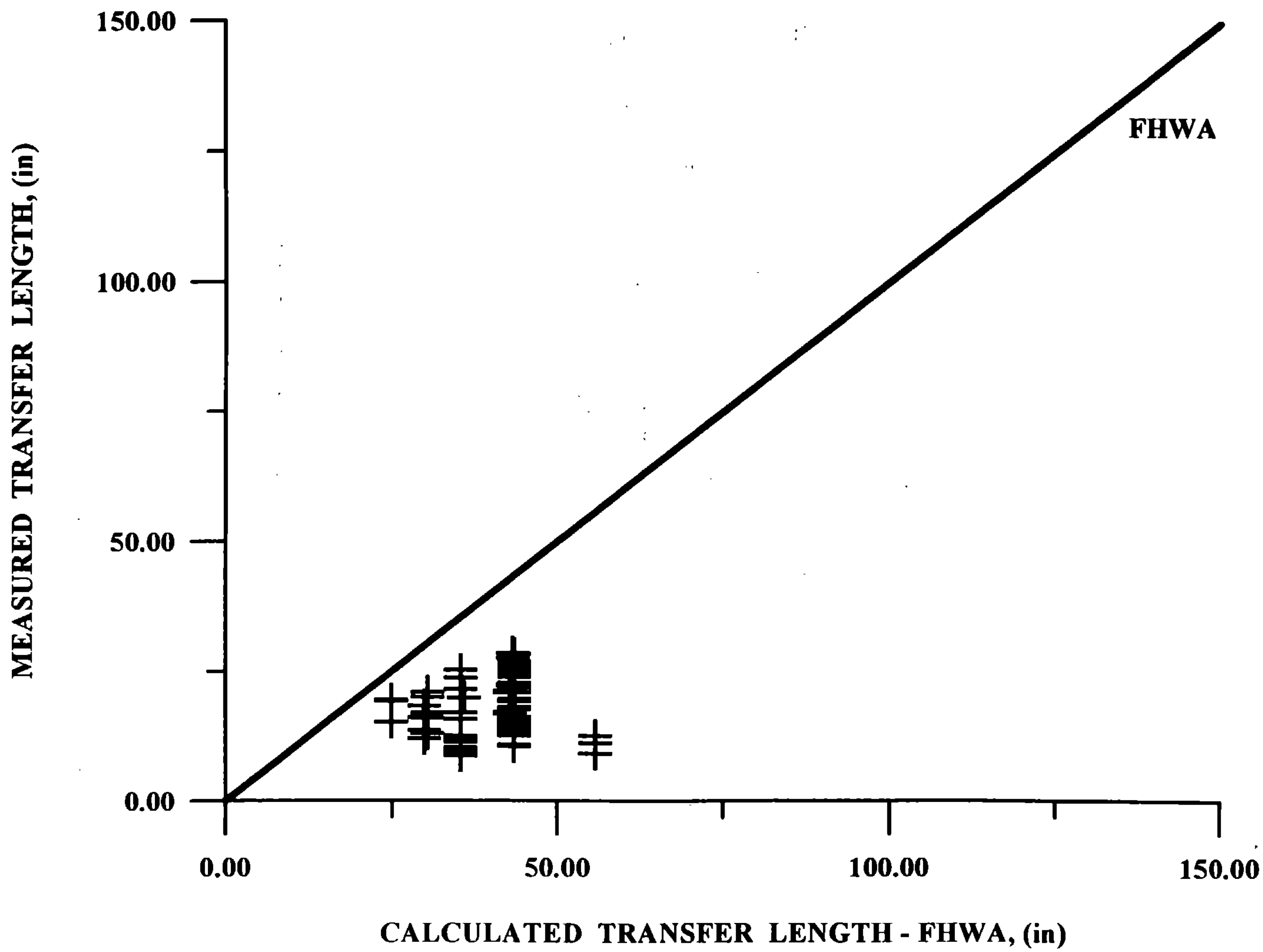
1 in = 25.4 mm

Figure 20. Comparison of measured transfer length values for all members with the FHWA equation.



1 in = 25.4 mm

Figure 21. Comparison of measured transfer length values for members having concrete strengths between 34.4 and 48.2 MPa (5000 and 7000 psi) with the FHWA equation.



1 in = 25.4 mm

Figure 22. Comparison of measured transfer length values for members having concrete strengths between 55.1 and 68.9 MPa (8000 and 10,000 psi) with the FHWA equation.

Similar to the other two equations, the FHWA transfer length equation was conservative for members constructed with high-strength concrete (see figure 22). This was significant because the FHWA equation has the parameter "concrete strength, (f'_c)" in the denominator of the equation. Because the concrete-strength parameter is in the denominator, a designer obtains a shorter transfer length as the concrete strength increases. This shows that the FHWA transfer length equation is not only conservative, but it also benefits the designer who uses high-strength concrete.

These figures indicate that the best choice for a transfer length equation is the FHWA transfer length equation (Equation (14)).

There were relatively few data points for members with a concrete compressive strength, f'_c, over 10,000 psi. Therefore, until more data becomes available, it was determined that **values of f'_c greater than 10,000 psi shall be taken as 10,000 psi for use in Equation (14).**

Figures 23 through 34 compare four development length equations:

- The new FHWA equation—Equation (16).
- The AASHTO equation—Equation (1).
- The current stop-gap measure of 1.6 times the AASHTO equation (as per the 1988 FHWA memorandum).
- The Buckner equation—Equation (3).

Each figure includes all data (data from rectangular specimens as well as full-size beams) for a given strand diameter and concrete strength, such as 12.7-mm- (0.5-in-) diameter strands in members where the concrete compressive strength at 28 days is 34.4 MPa (5000 psi). Along the y-axis, the actual measured development lengths are plotted. Along the x-axis, the quantity (f^*_{su} - f_{se}) is plotted because it is a quantity that is contained within all of the equations.

For each graph, it was assumed that the stress in the strands prior to transfer (f_{pt}) was 1395 MPa (202,500 psi). To generate the line showing the Buckner equation, a value for the constant λ had to be assumed. The value for λ was calculated for each data point using Equation (4), and then the values of λ for all of the data points on a given graph were averaged together to provide the value for λ used to generate the Buckner equation line for a particular graph.

In these figures, the different failure modes are represented by different symbols:

- The slip/shear failures are shown by the square symbols. Slip/shear failures are the worst types of failure—they occur with no warning and should be avoided.
- The flexural failures are shown by the circular symbols. Flexural failures are acceptable (good) failure types because there is warning (excessive deflection and cracking) before the failure.
- The combination failures are shown by the diamond symbols. This is when flexural failure occurs at the same time as slip/shear failure. These combination failures indicated that the embedment length being tested was the exact development length.

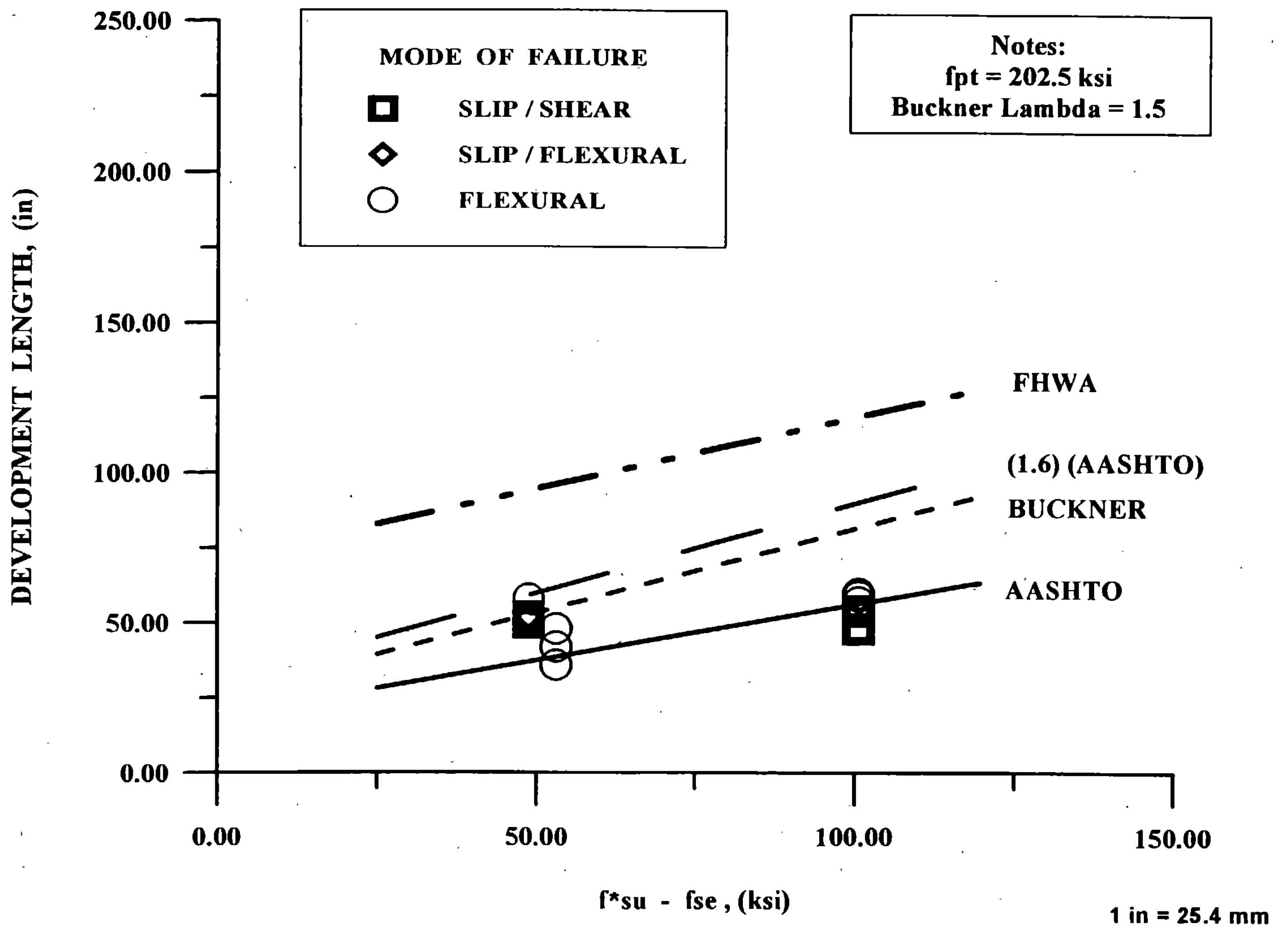


Figure 23. Comparison of development length equations for data with a strand diameter of 9.5 mm (3/8 in) and a concrete strength of 34.4 MPa (5000 psi).

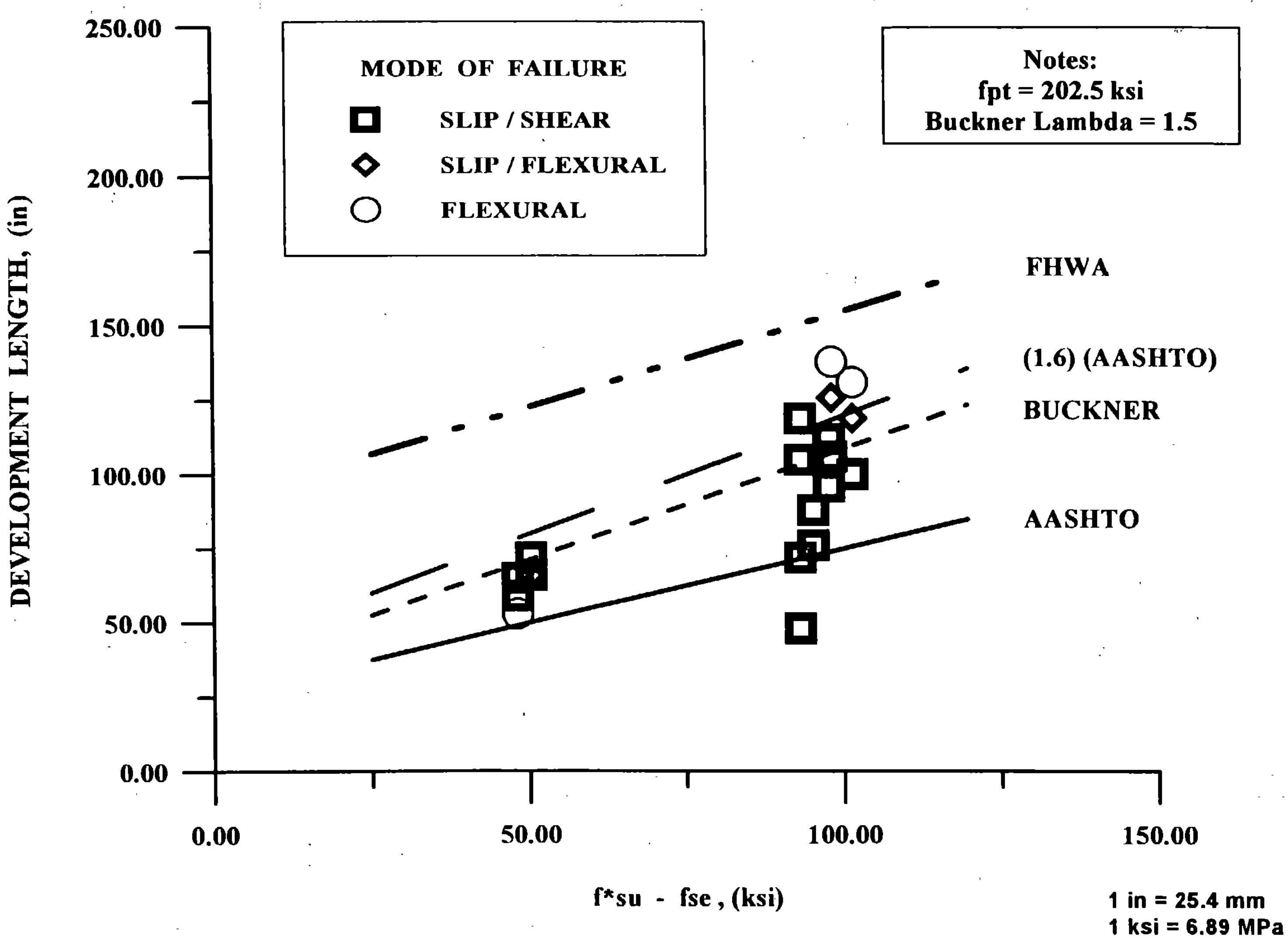
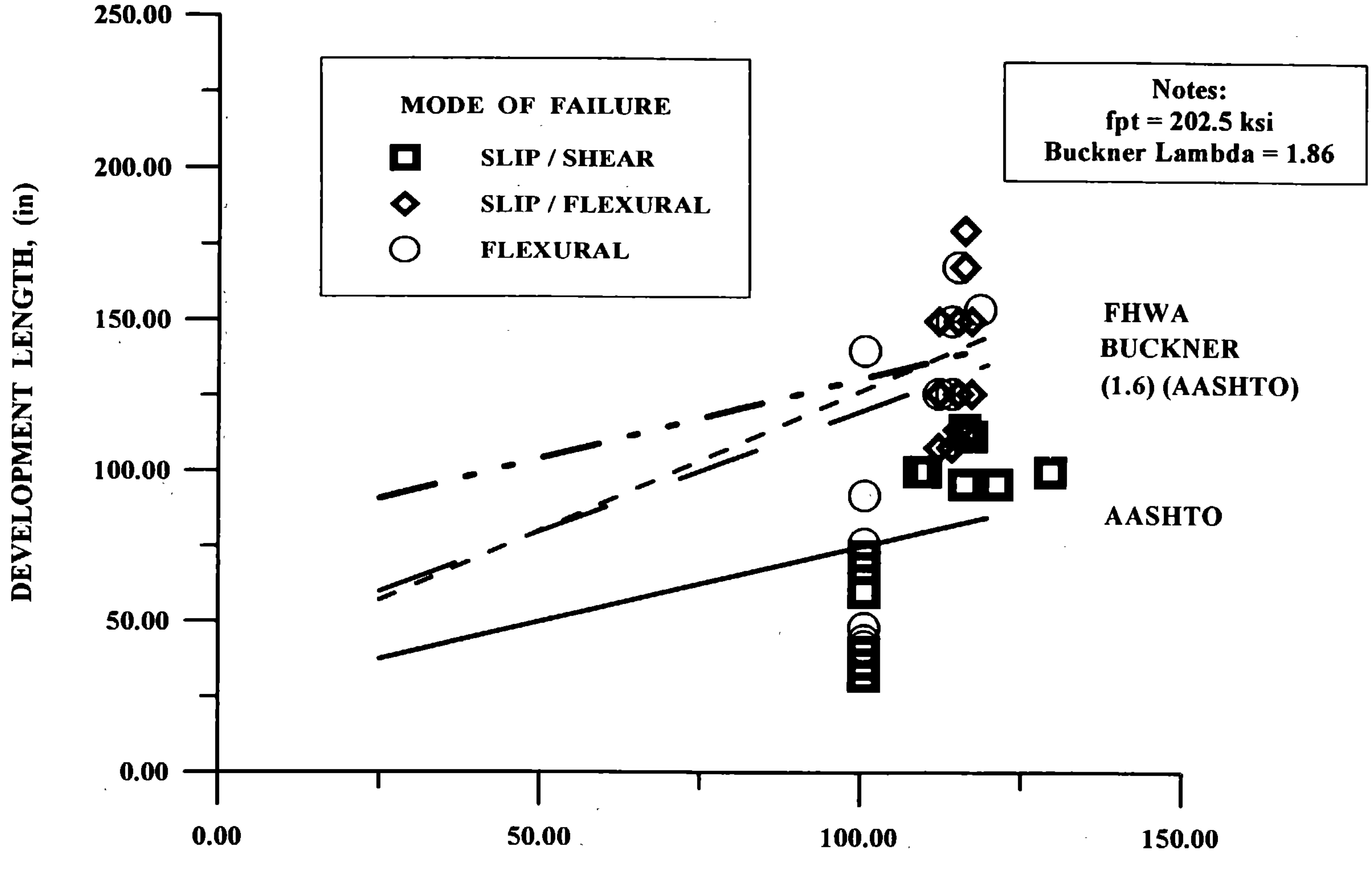


Figure 24. Comparison of development length equations for data with a strand diameter of 12.7 mm (0.5 in) and a concrete strength of 34.4 MPa (5000 psi).



1 in = 25.4 mm
1 ksi = 6.89 MPa

Figure 25. Comparison of development length equations for data with a strand diameter of 12.7 mm (0.5 in) and a concrete strength of 41.3 MPa (6000 psi).

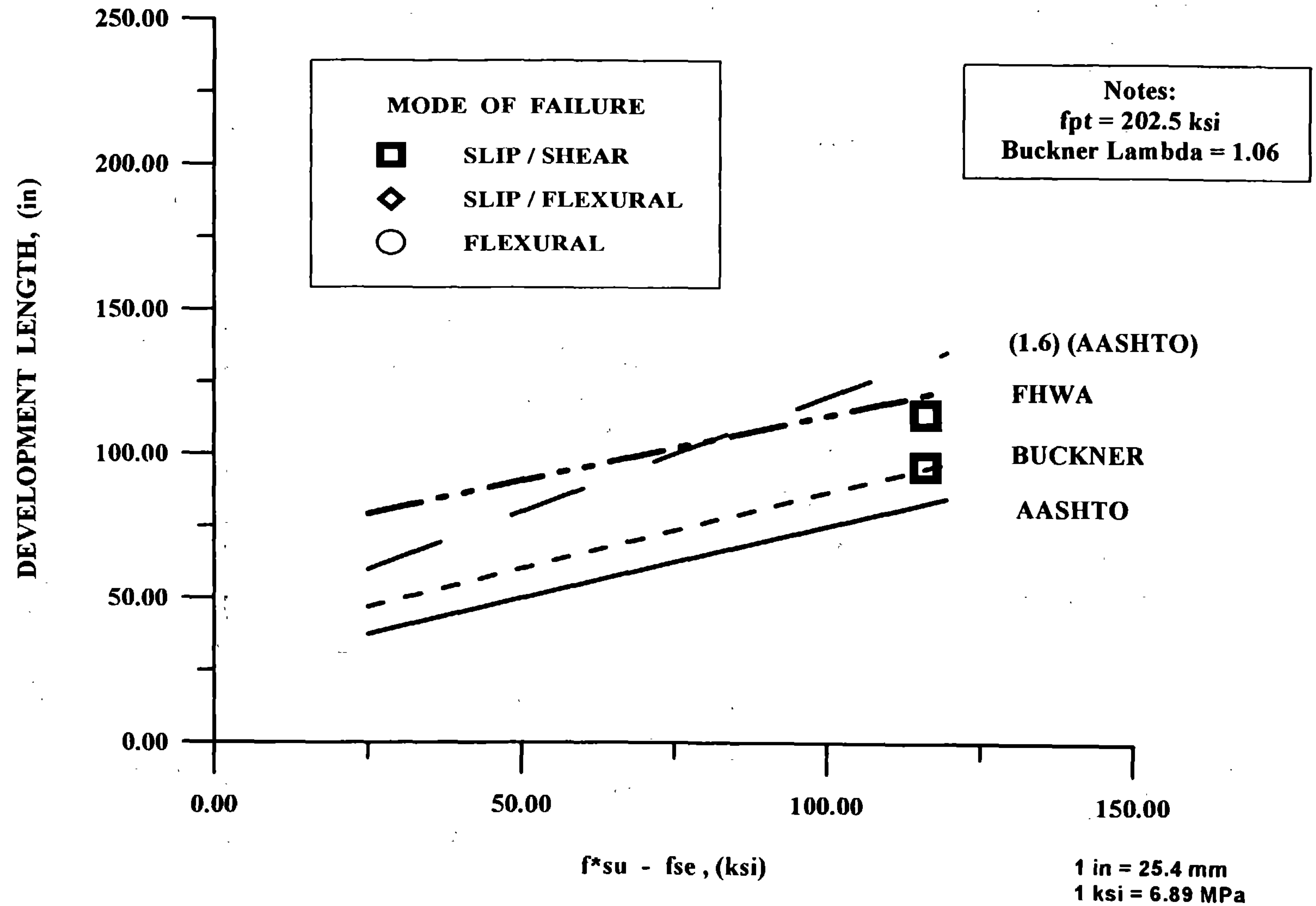


Figure 26. Comparison of development length equations for data with a strand diameter of 12.7 mm (0.5 in) and a concrete strength of 48.2 MPa (7000 psi).

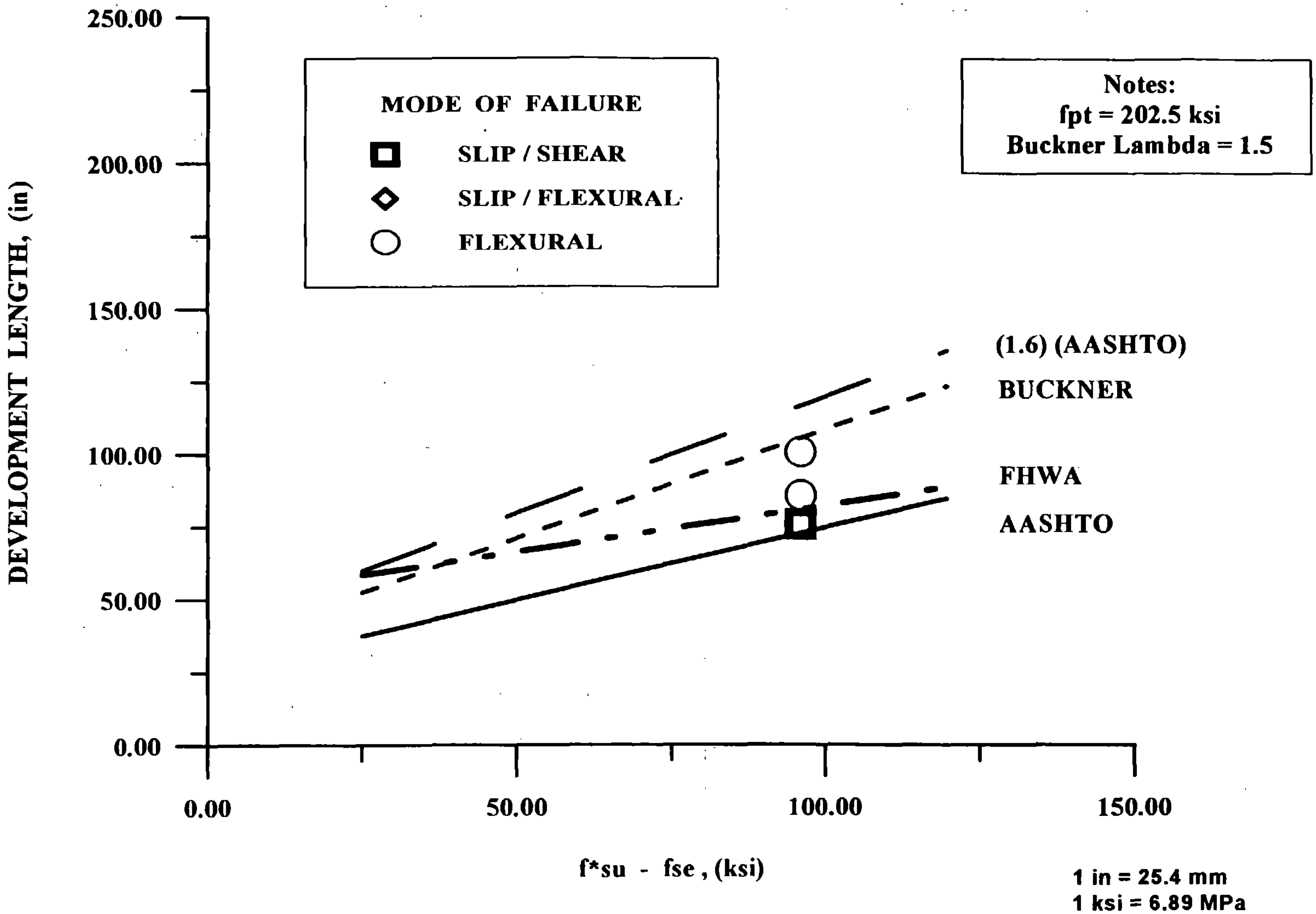
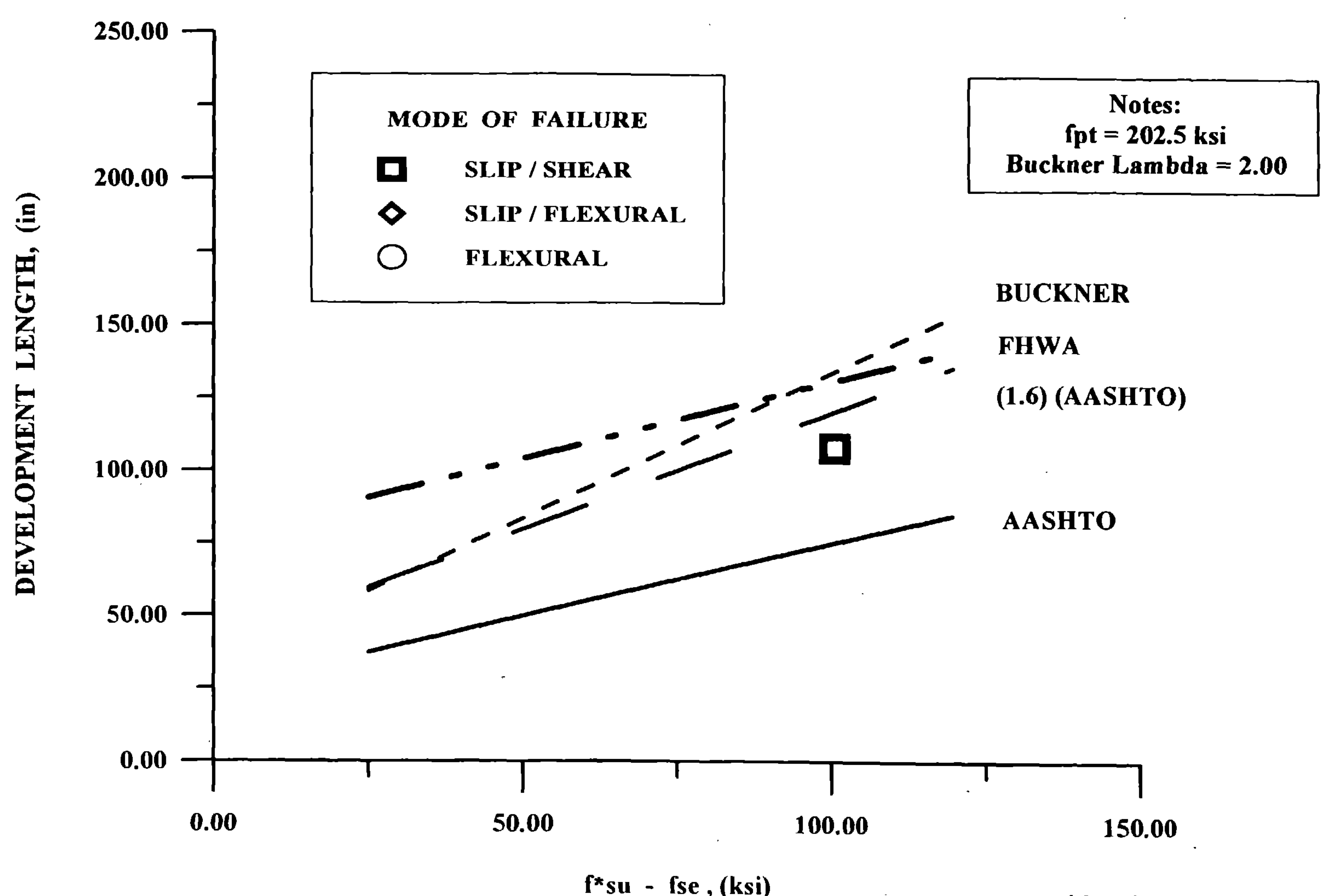


Figure 27. Comparison of development length equations for data with a strand diameter of 12.7 mm (0.5 in) and a concrete strength of 68.9 MPa (10,000 psi).



1 in = 25.4 mm
1 ksi = 6.89 MPa

Figure 28. Comparison of development length equations for data with a strand diameter of 12.7 mm (0.50 in) Special and a concrete strength of 41.3 MPa (6000 psi).

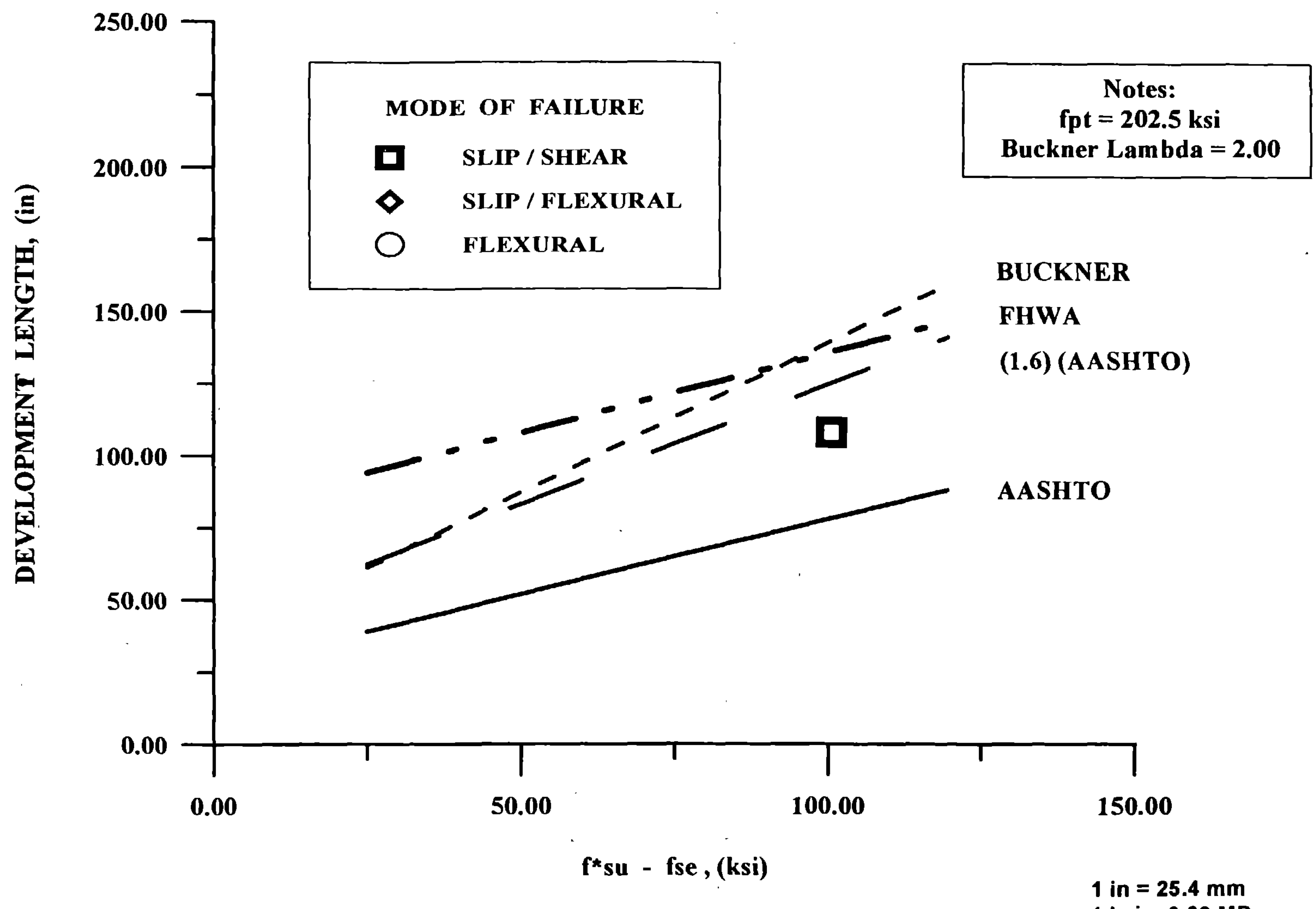
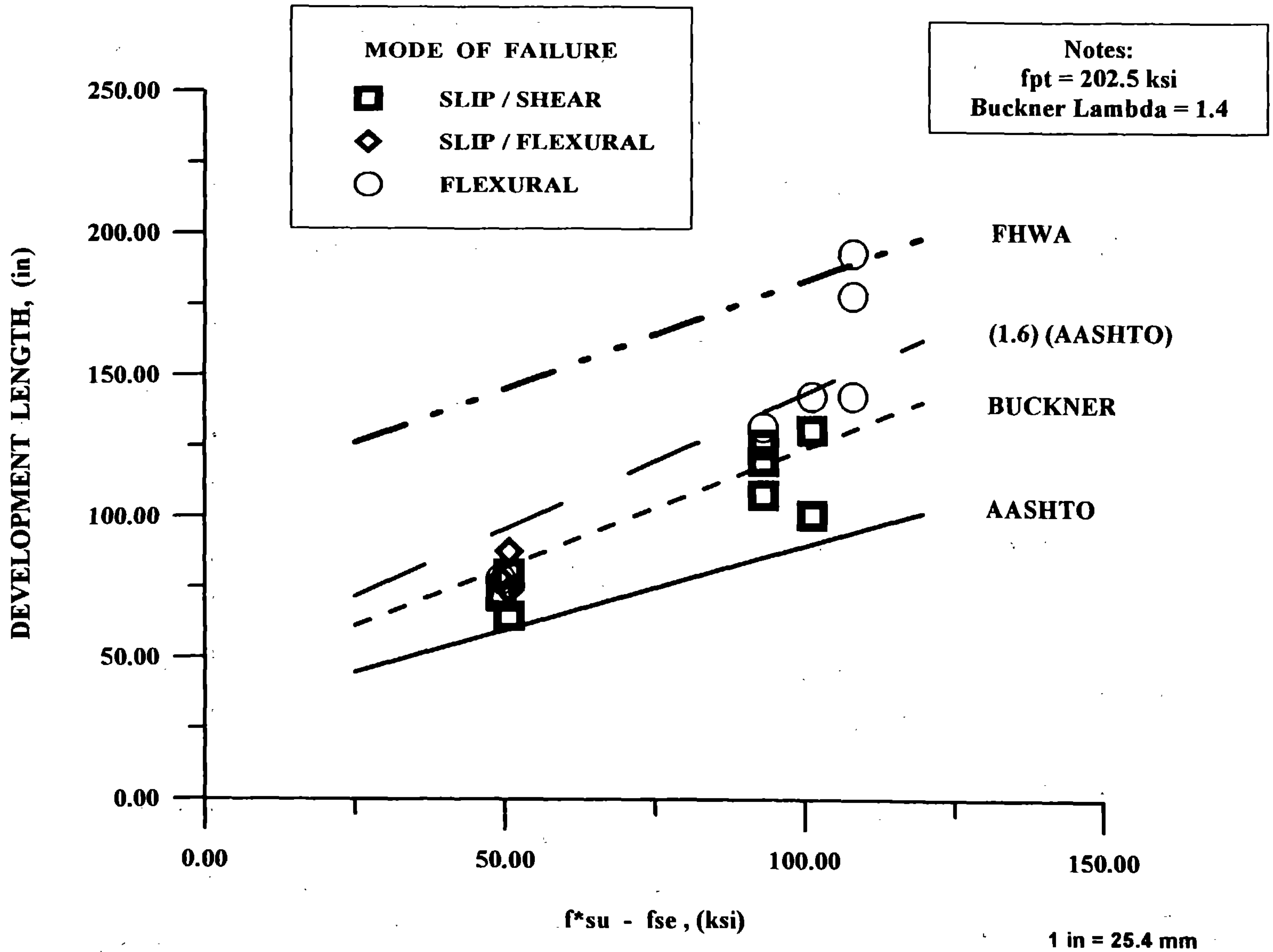
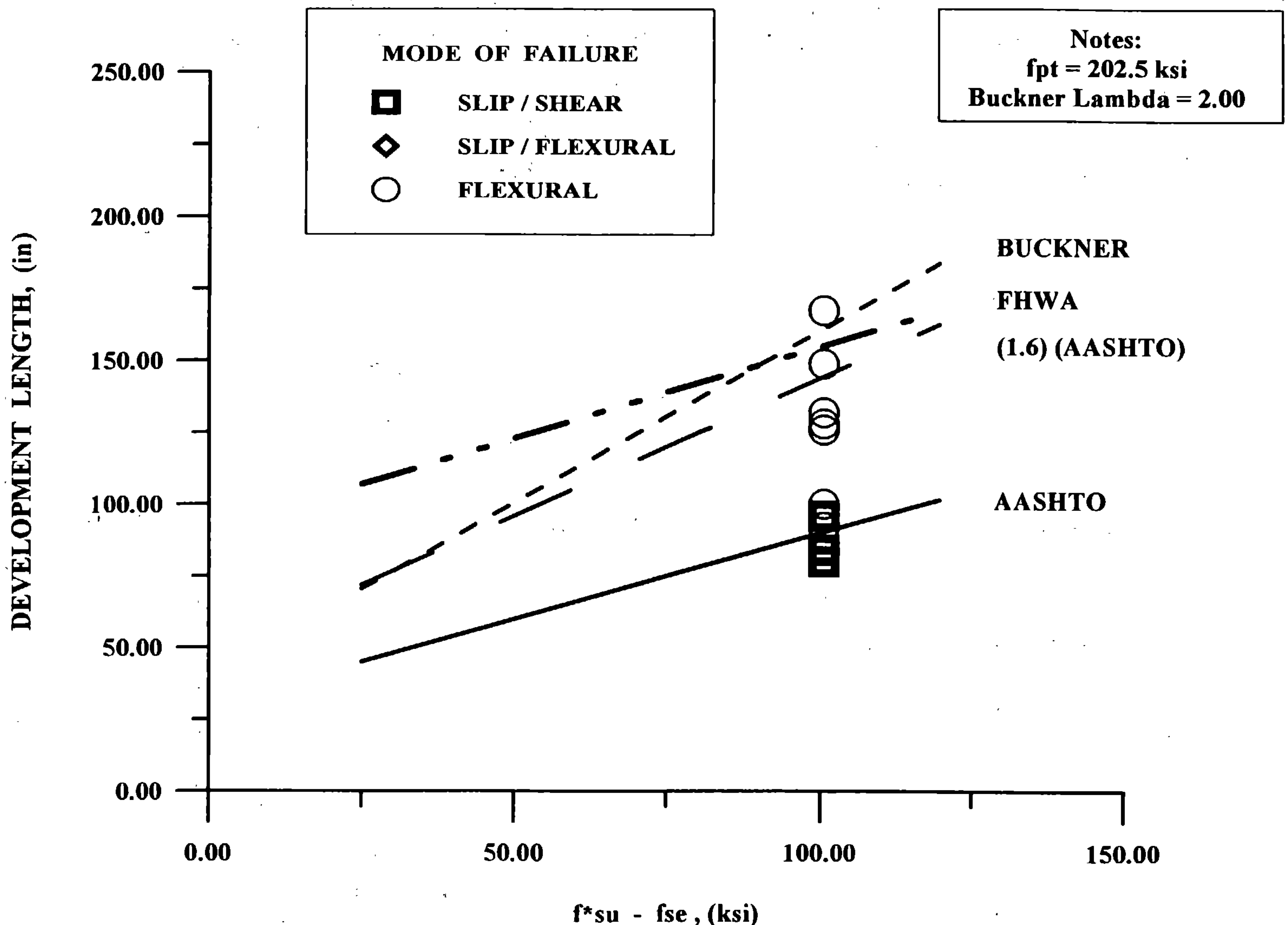


Figure 29. Comparison of development length equations for data with a strand diameter of 12.7 mm (0.52 in) Special and a concrete strength of 41.3 MPa (6000 psi).



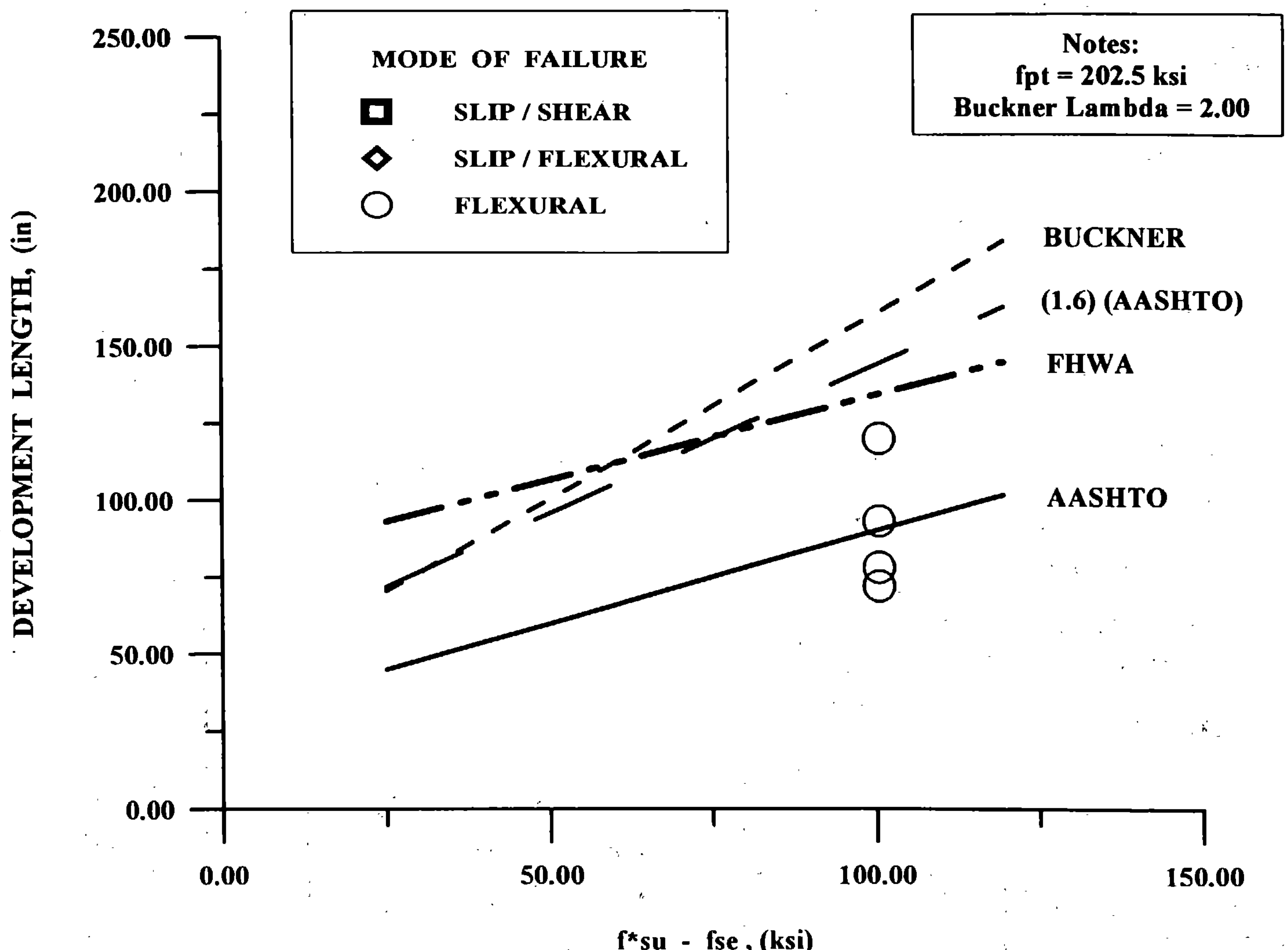
1 in = 25.4 mm
1 ksi = 6.89 MPa

Figure 30. Comparison of development length equations for data with strand diameter of 15.2 mm (0.6 in) and a concrete strength of 34.4 MPa (5000 psi).



1 in = 25.4 mm
1 ksi = 6.89 MPa

Figure 31. Comparison of development length equations for data with a strand diameter of 15.2 mm (0.6 in) and a concrete strength of 41.3 MPa (6000 psi).



1 in = 25.4 mm
1 ksi = 6.89 MPa

Figure 32. Comparison of development length equations for data with a strand diameter of 15.2 mm (0.6 in) and a concrete strength of 48.2 MPa (7000 psi).

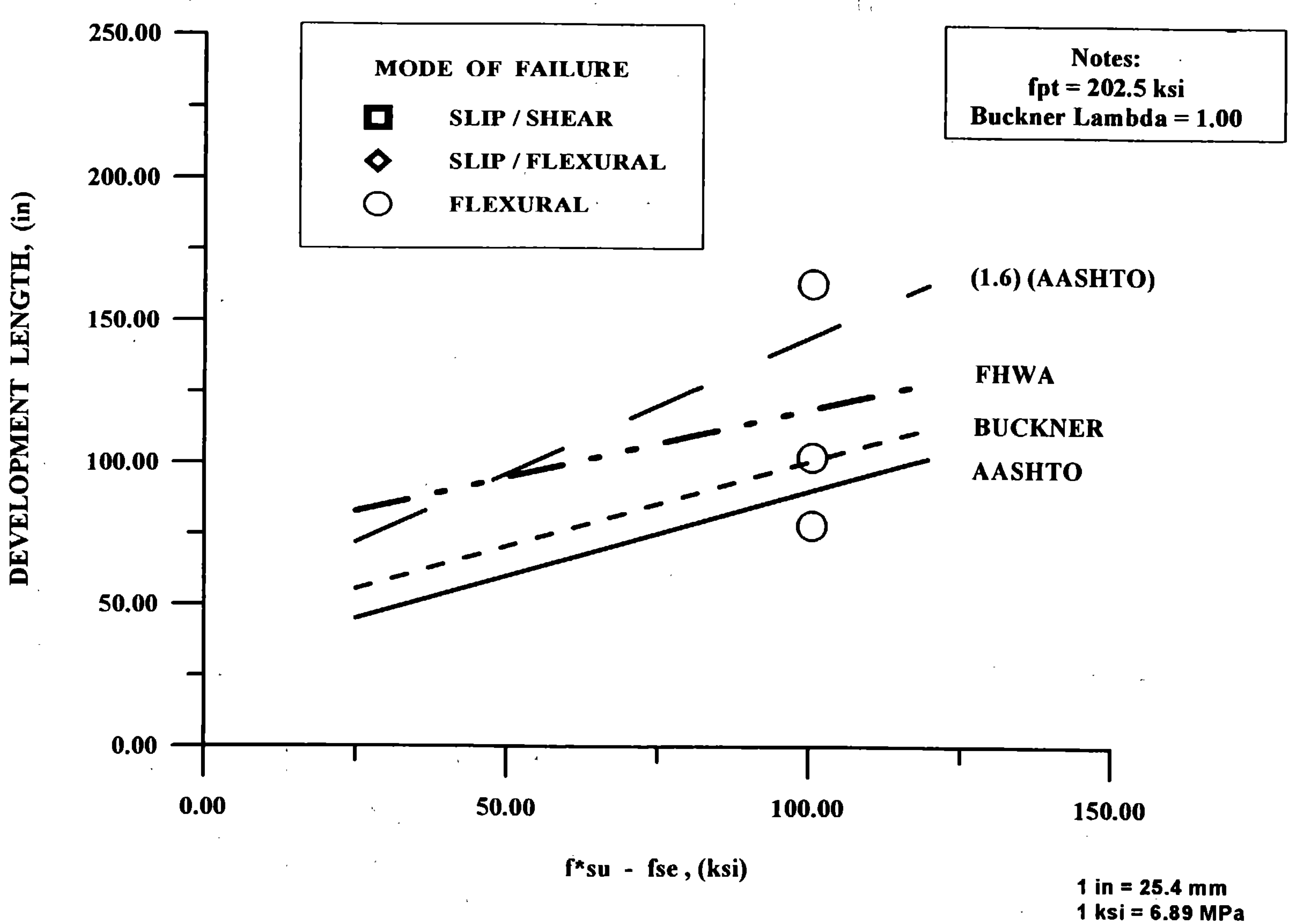
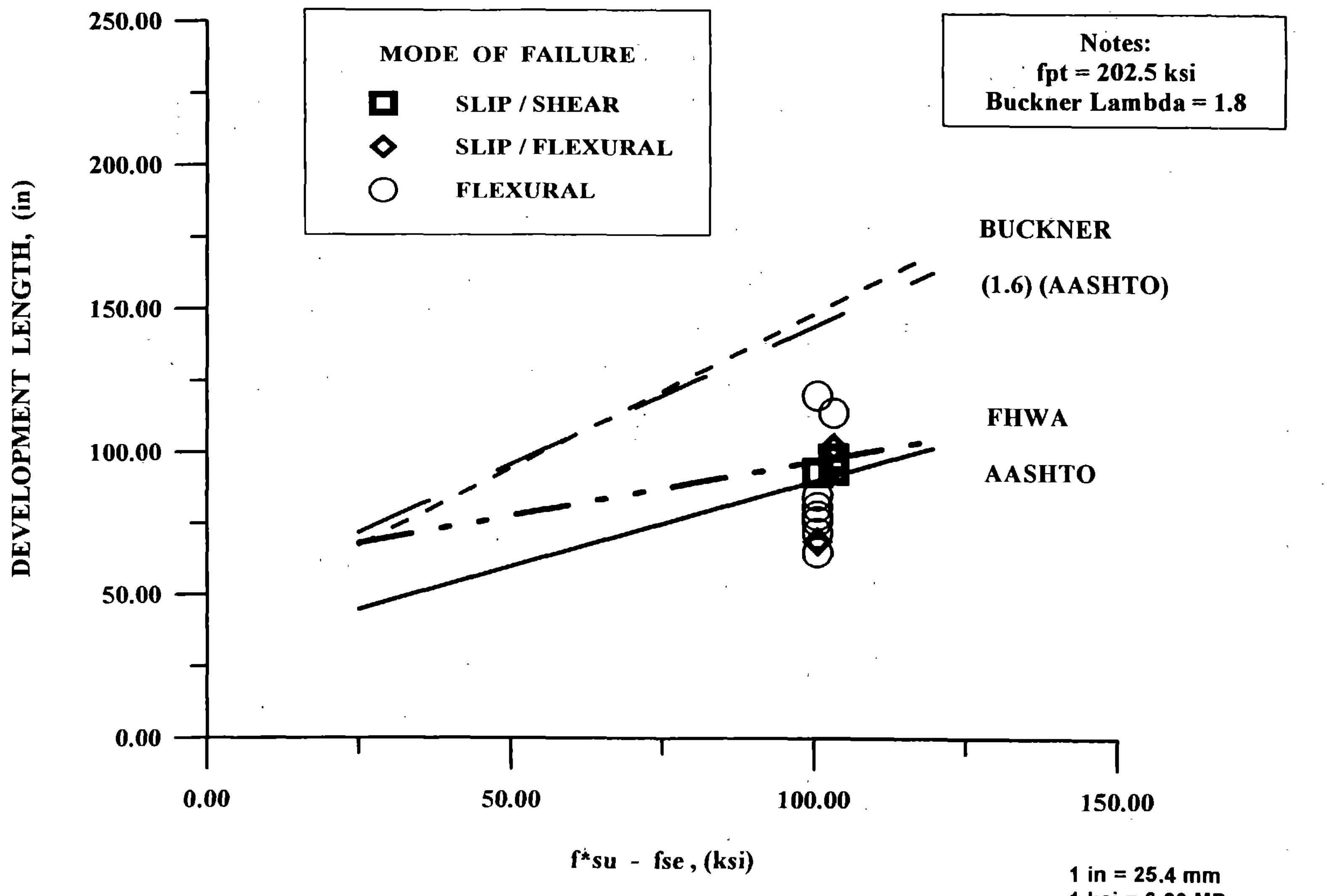


Figure 33. Comparison of development length equations for data with a strand diameter of 15.2 mm (0.6 in) and a concrete strength of 55.1 MPa (8000 psi).



1 in = 25.4 mm
1 ksi = 6.89 MPa

Figure 34. Comparison of development length equations for data with a strand diameter of 15.2 mm (0.6 in) and a concrete strength of 68.9 MPa (10,000 psi).

As seen in these figures, if a slip/shear failure fell above an equation line, the equation was unconservative. It was unconservative because a slip/shear failure indicated that the development length being tested was too short. Therefore, if the development length being tested was too short, yet the equation predicted a value for development length even shorter than the one being tested, then the equation predicted a development length that was too short. Therefore, the development length predicted from the equation was unconservative.

Figures 23 through 26 and 28 through 32 illustrate cases where normal-strength concrete was used (f'_c between 34.4 and 48.2 MPa [5000 and 7000 psi]). It is evident from many of these figures that the AASHTO and Buckner equations were unconservative because slip/shear failures fell above those lines. In one instance, the case of 12.7-mm- (0.5-in-) diameter strands in 34.4-MPa (5000-psi) concrete members (figure 24), the line representing 1.6 times the AASHTO equation was unconservative. However, the new FHWA equation was consistently conservative when using normal-strength concrete (f'_c between 34.4 and 48.2 MPa [5000 and 7000 psi]).

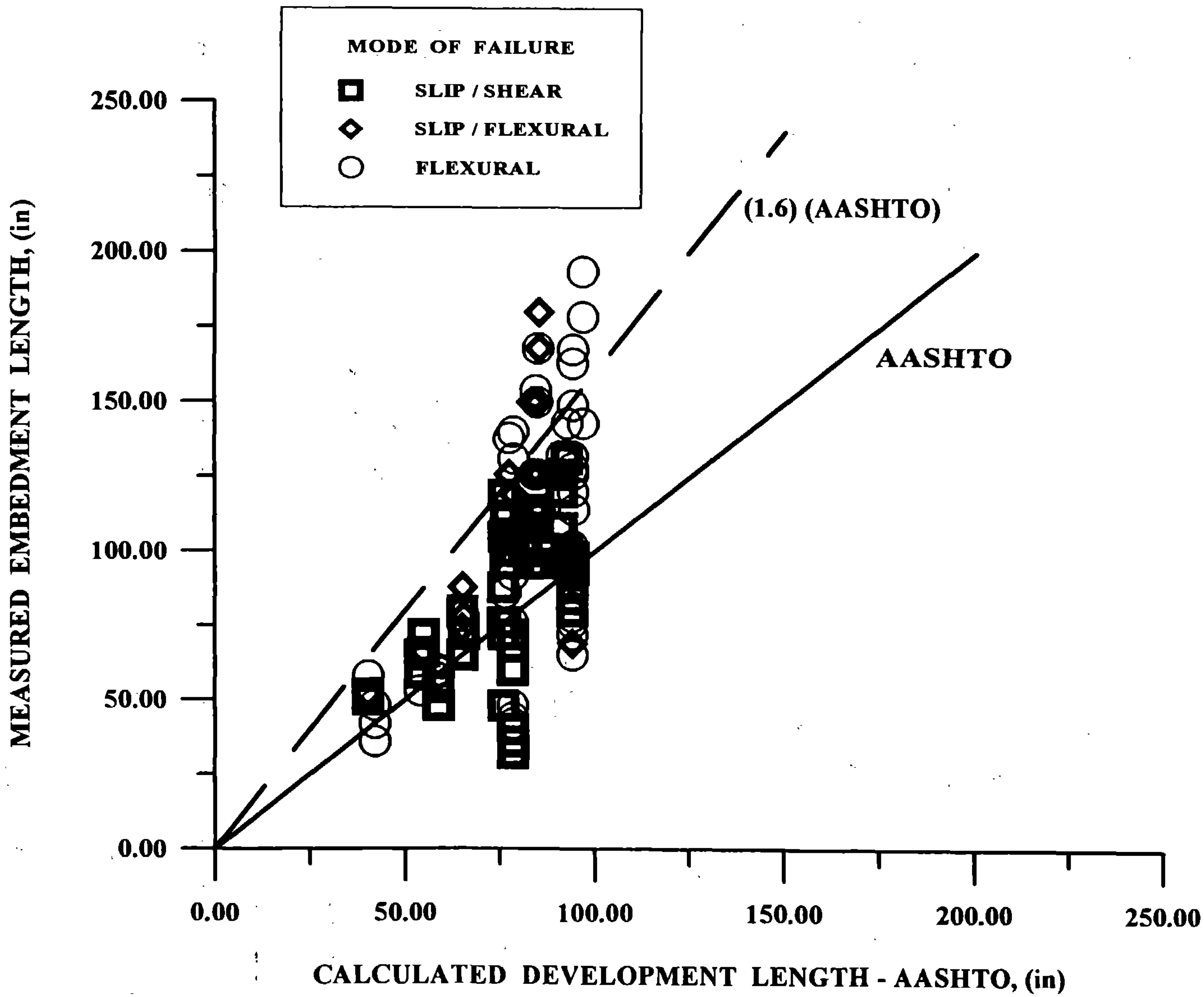
In figure 25, there were some combination failures that fell above all four of the equations. These represented the extreme worst case in terms of long development lengths, but did provide some warning signs due to the flexural failure component of the combination failure.

Figures 27, 33, and 34 illustrate cases in which high-strength concrete was used (f'_c between 55.1 and 68.9 MPa [8000 and 10,000 psi]). Development lengths determined in the research studies were consistently shorter in high-strength concrete. The AASHTO equation was sometimes unconservative (figures 27 and 34) and sometimes conservative (figure 33) for the high-strength concrete members. The Buckner equation was consistently conservative, but it over-predicted development length by a large margin (figures 27 and 34). The FHWA equation was crafted such that designers were given credit for using high-strength concrete and therefore obtained shorter development lengths. Therefore, the FHWA equation is conservative for these cases, but not overly conservative (see figures 27, 33, and 34).

Figures 28 and 29 were included to illustrate the case of using 12.7-mm (0.5-in) Special diameter strand. Figure 28 was drawn using a diameter of 0.50 inches in the equations, while figure 29 was drawn using a diameter of 0.52 inches in the equations. Although there appears to be only one data point on this graph, the one point is actually 15 different development length tests conducted by Florida DOT at the same embedment length.[27-28] Therefore, when plotted, they all appear at the same point. There was virtually no difference between figures 28 and 29, illustrating that none of the development length equations were very sensitive to the slight variation in the diameter of the 12.7-mm (0.5-in) Special diameter strands.

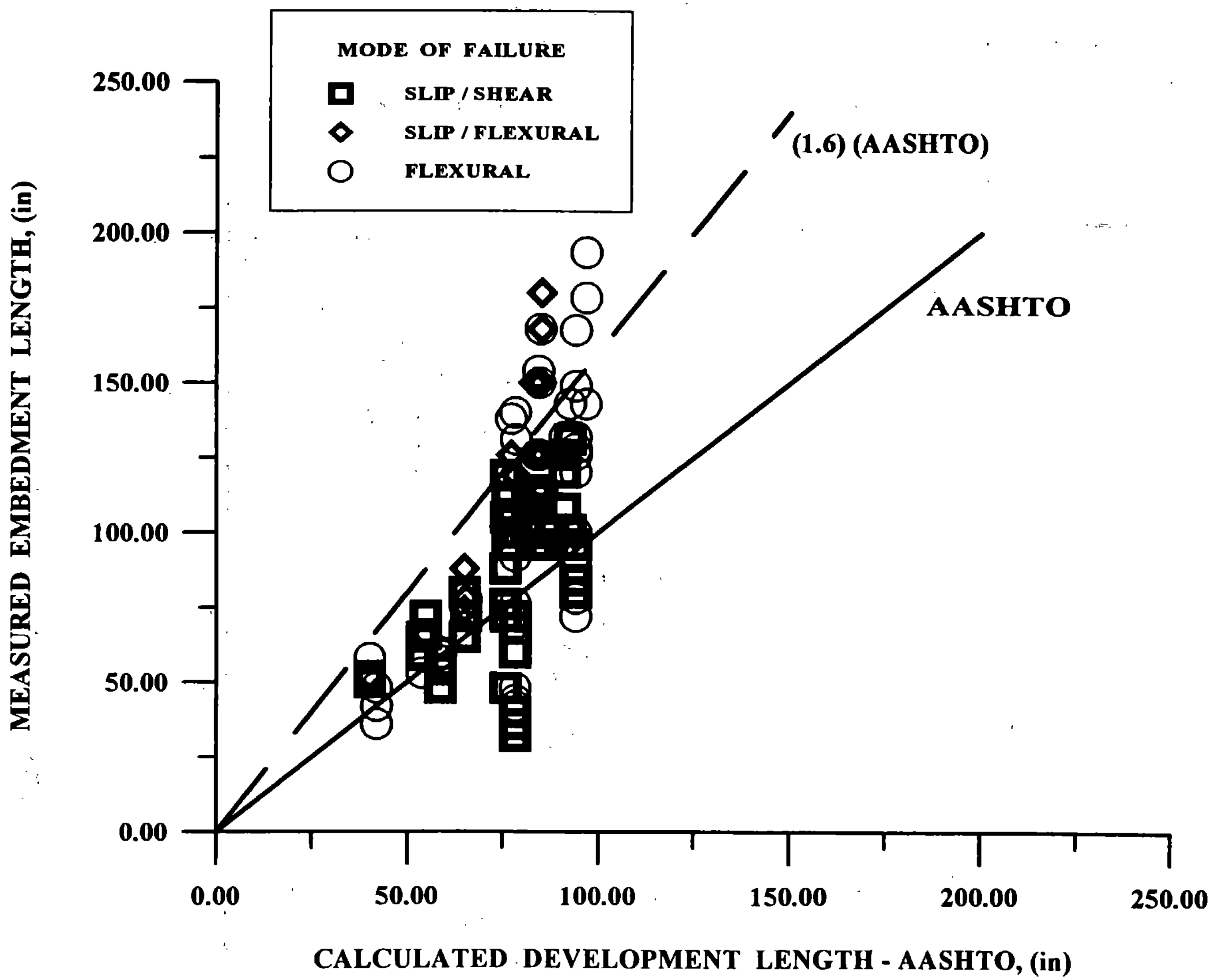
Additional graphs of the development length data are provided in figures 35 through 43. In each of these figures, the different failure modes are again represented by different symbols:

- The slip/shear failures are shown by the square symbols. Slip/shear failures are the worst types of failure—they occur with no warning and should be avoided.



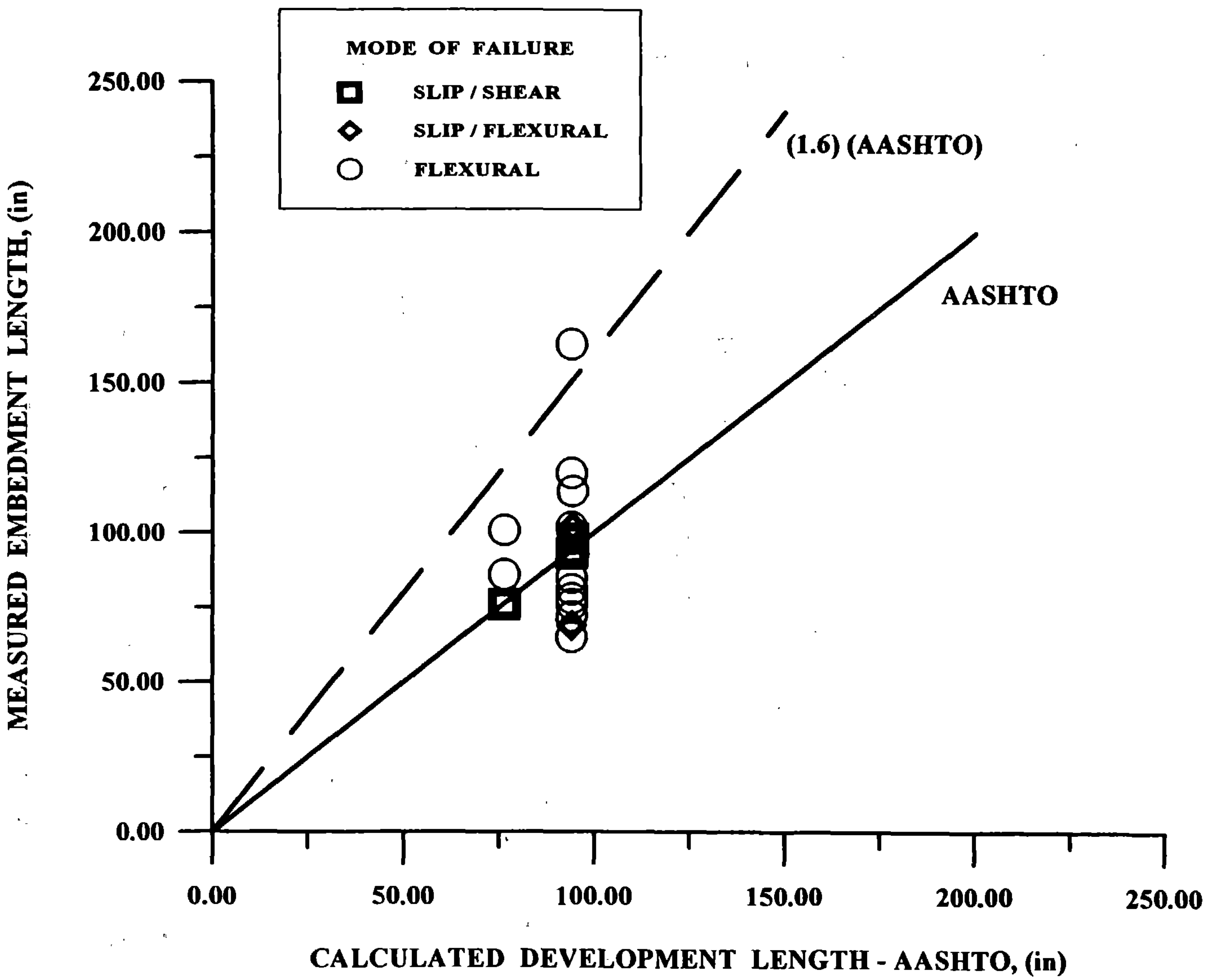
1 in = 25.4 mm

Figure 35. Comparison of measured development length values for all members with the AASHTO and 1.6 AASHTO equations.



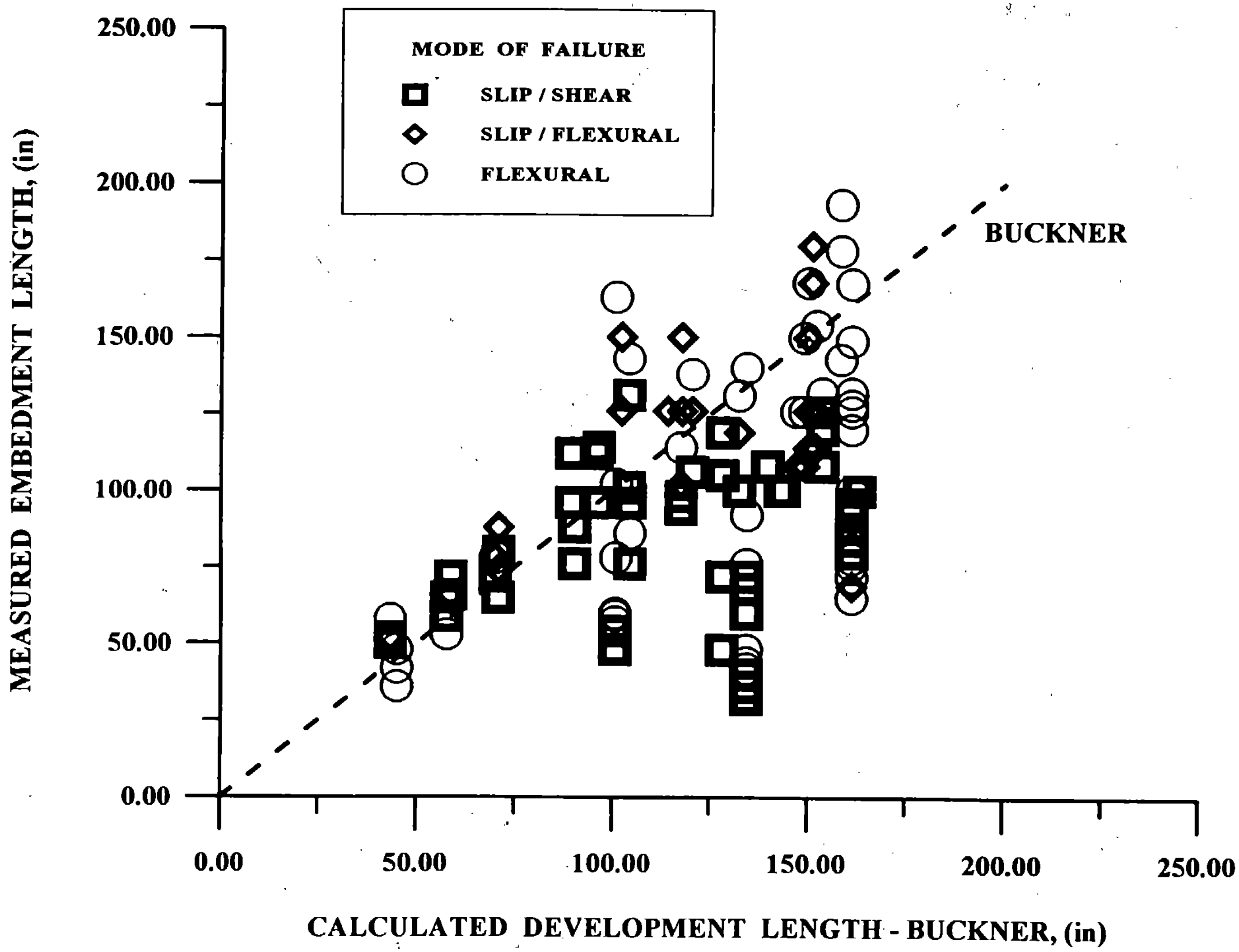
1 in = 25.4 mm

Figure 36. Comparisons of measured development length values for members having concrete strengths between 34.4 and 48.2 MPa (5000 and 7000 psi) with the AASHTO and 1.6 AASHTO equations.



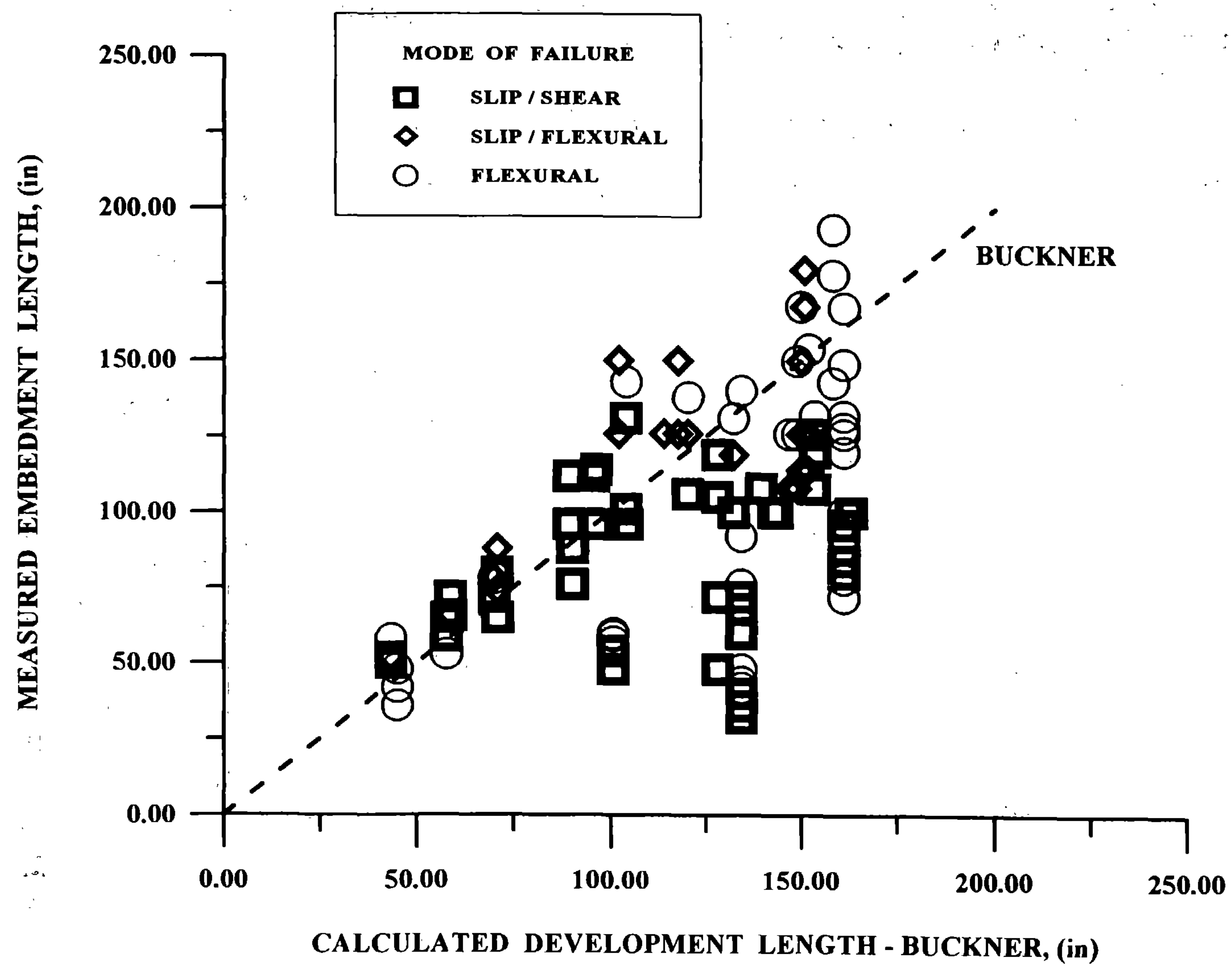
1 in = 25.4 mm

Figure 37. Comparison of measured development length values for members having concrete strengths between 55.1 and 68.9 MPa (8000 and 10,000 psi) with the AASHTO and 1.6 AASHTO equations.



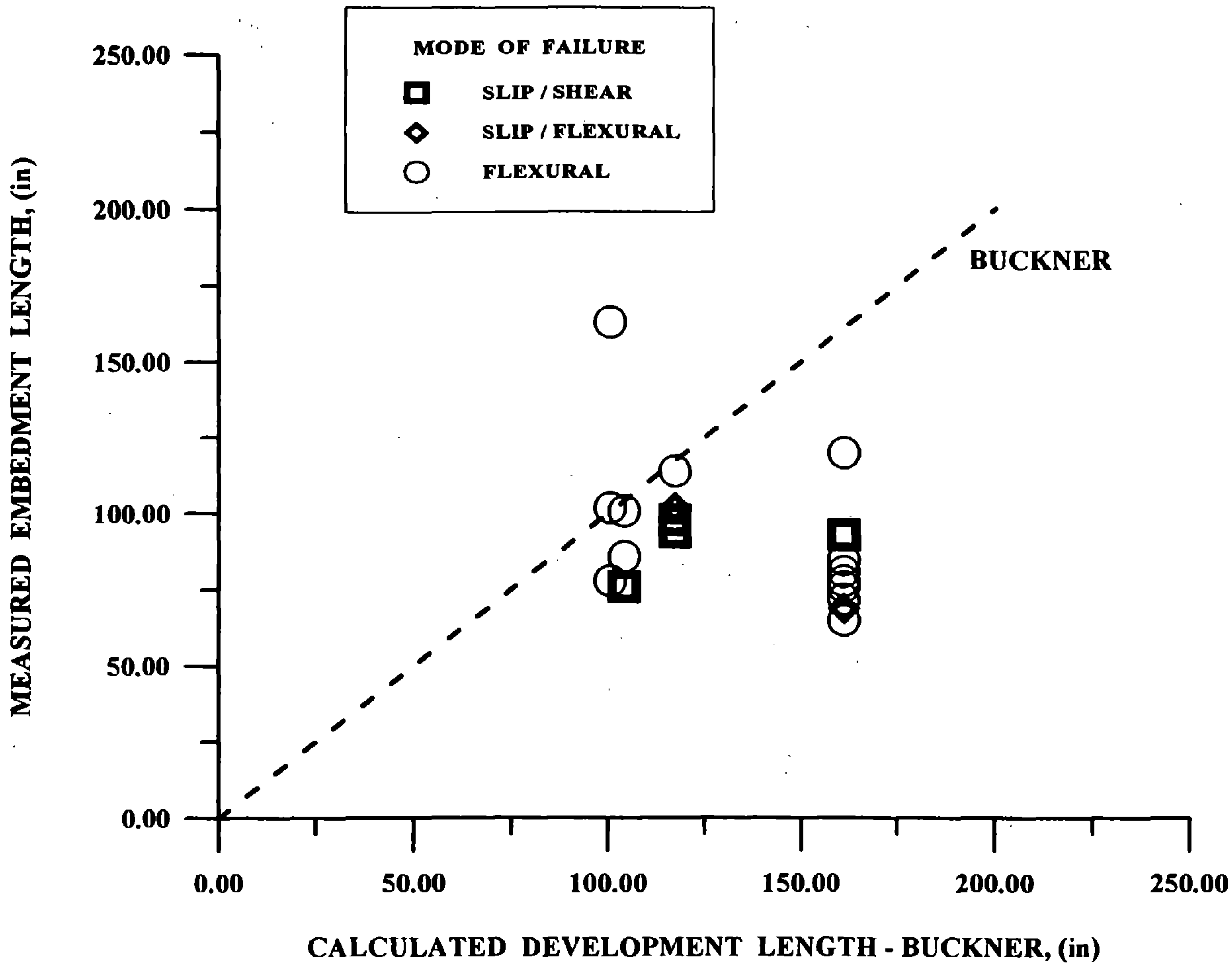
1 in = 25.4 mm

Figure 38. Comparison of measured development length values for all members with the Buckner equation.



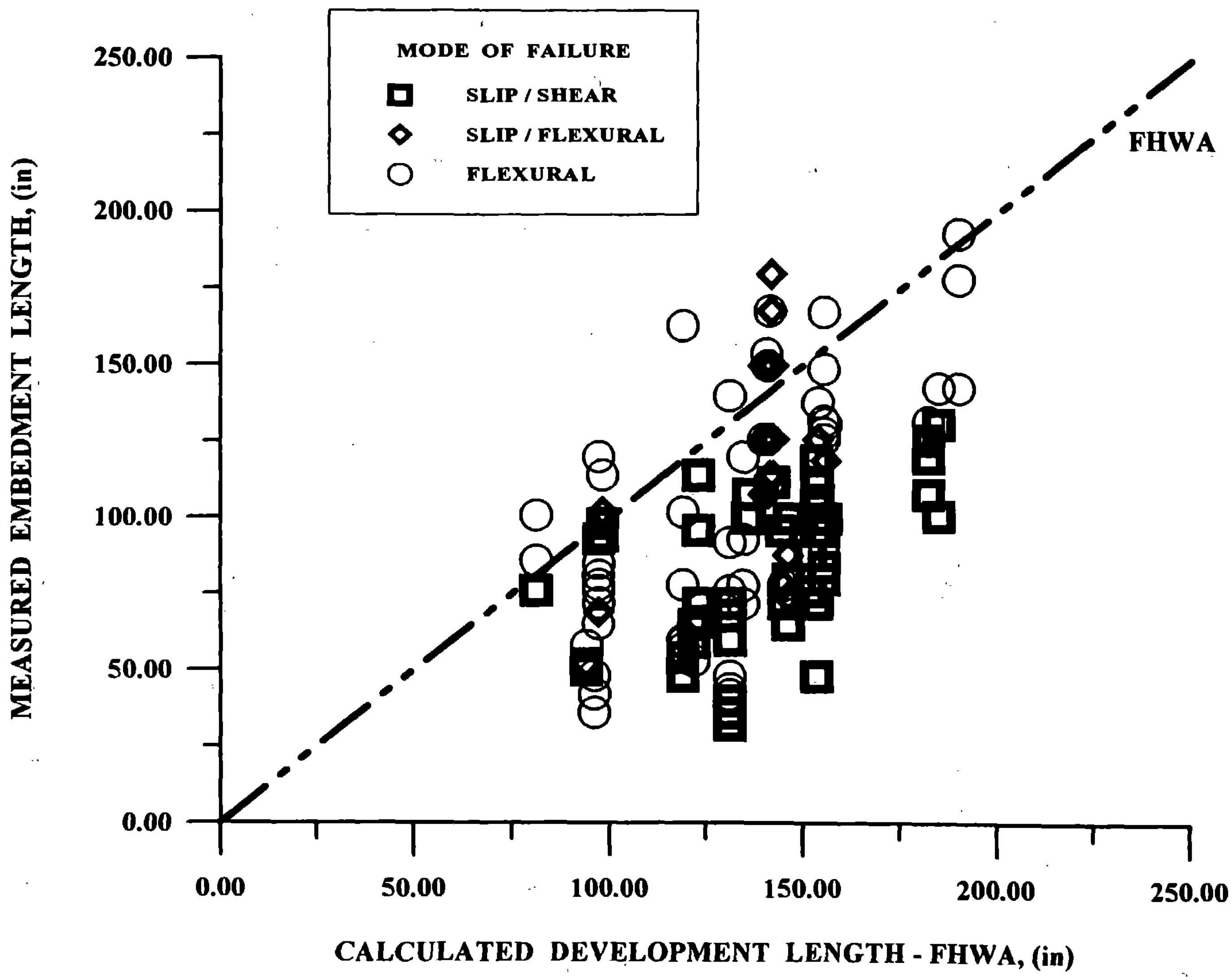
1 in = 25.4 mm

Figure 39. Comparison of measured development length values for members having concrete strengths between 34.4 and 48.2 MPa (5000 and 7000 psi) with the Buckner equation.



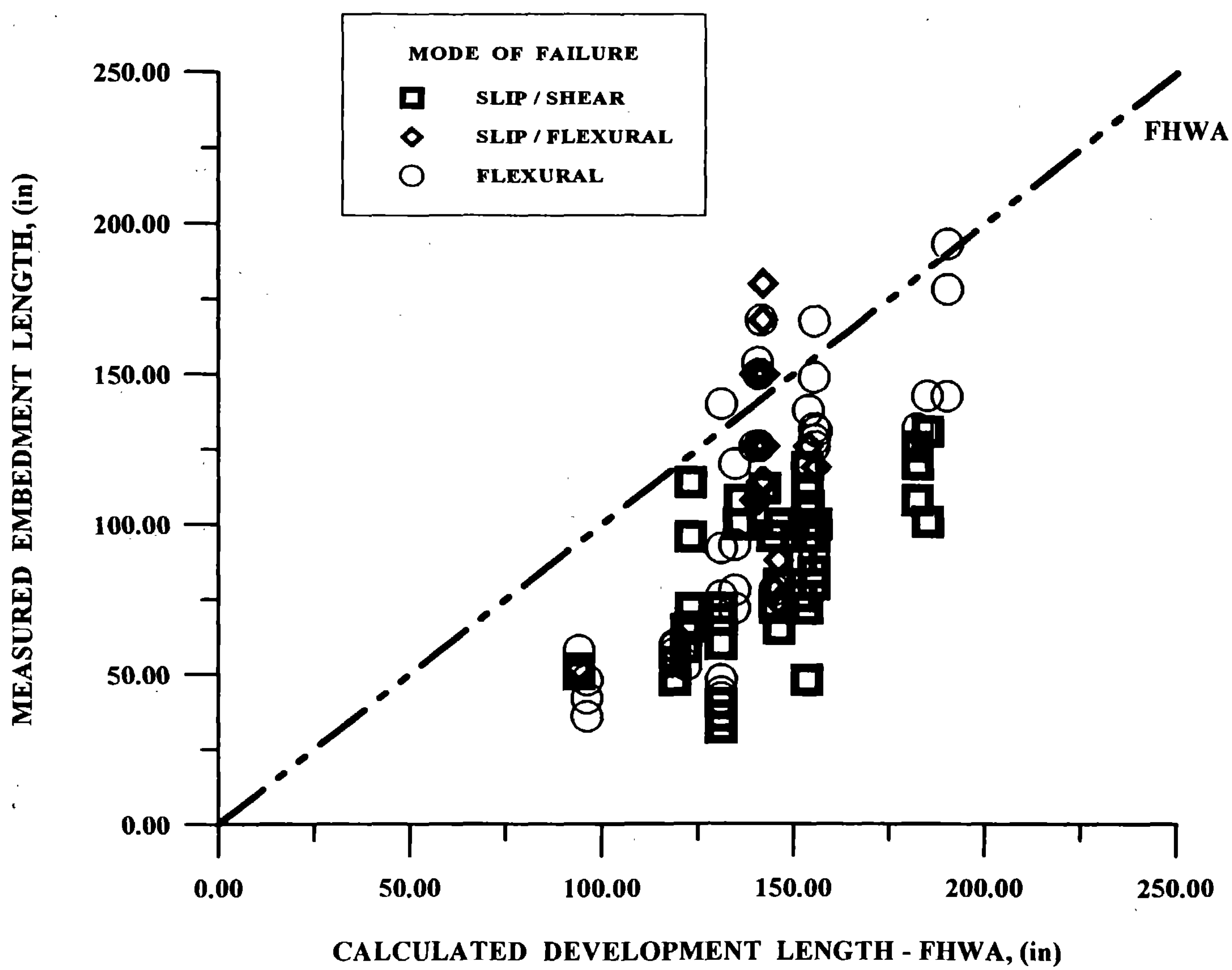
1 in = 25.4 mm

Figure 40. Comparison of measured development length values for members having concrete strengths between 55.1 and 68.9 MPa (8000 and 10,000 psi) with the Buckner equation.



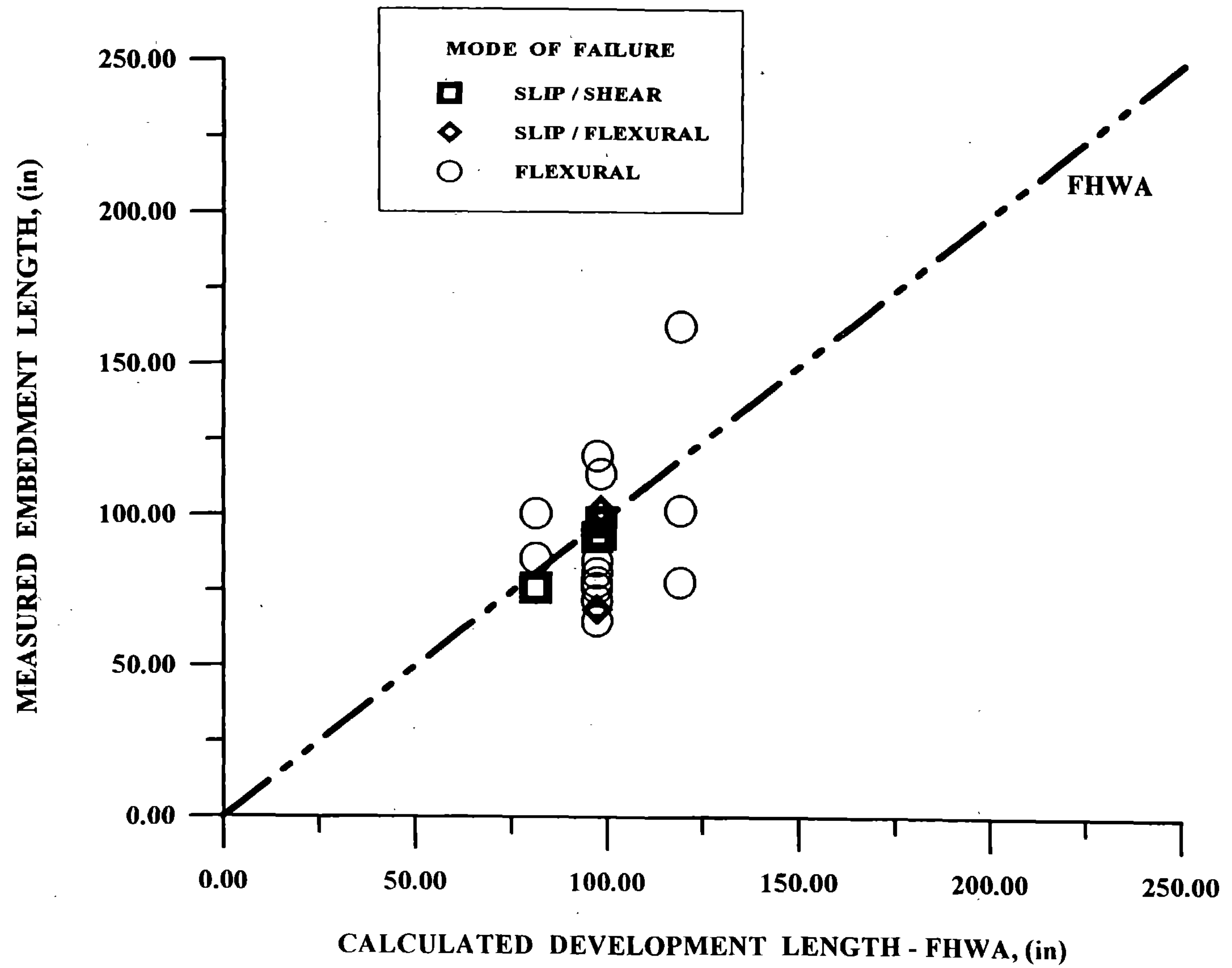
1 in = 25.4 mm

Figure 41. Comparison of measured development length values for all members with the FHWA equation.



1 in = 25.4 mm

Figure 42. Comparison of measured development length values for members having concrete strengths between 34.4 and 48.2 MPa (5000 and 7000 psi) with the FHWA equation.



1 in = 25.4 mm

Figure 43. Comparison of measured development length values for members having concrete strengths between 55.1 and 68.9 MPa (8000 and 10,000 psi) with the FHWA equation.

- The flexural failures are shown by the circular symbols. Flexural failures are acceptable (good) failure types because there is warning (excessive deflection and cracking) before the failure.

- The combination failures are shown by the diamond symbols. This is when flexural failure occurred at the same time as slip/shear failure. These combination failures indicated that the embedment length being tested was the exact development length.

Each figure includes all data (data from rectangular specimens as well as full-size beams). Along the y-axis, the actual measured embedment lengths are plotted. Along the x-axis, the development lengths calculated by a given development length equation are plotted. In the figures, the lines represent the cases in which the measured embedment length equaled the calculated development length using the given equation.

As seen in these figures, if a slip/shear failure fell above an equation line, the equation was unconservative. It was unconservative because a slip/shear failure indicated that the development length being tested was too short. Therefore, if the development length being tested was too short, yet the equation predicted a value for development length even shorter than the one being tested, then the equation predicted a development length that was too short. Therefore, the development length predicted from the equation was unconservative.

Figure 35 represents two equations: the AASHTO equation and the line representing 1.6 times the AASHTO equation line (as prescribed by the 1988 FHWA Memorandum[14]). Many slip/shear failures fell above the AASHTO equation, illustrating that this equation is unconservative.

In chapter 1, the AASHTO equation was presented as a reasonable mean. It is evident from figure 35 that the AASHTO development length equation represents an average equation for the data points and not a conservative equation.

A portion of the development length data is shown again with the AASHTO equation and the line representing 1.6 times the AASHTO equation in figures 36 and 37, with the data grouped by concrete compressive strength. The data with concrete strengths between 34.4 and 48.2 MPa (5000 and 7000 psi) are seen in figure 36, while figure 37 illustrates the data with concrete strengths between 55.1 and 68.9 MPa (8000 and 10,000 psi).

For the normal-strength concrete data shown in figure 36, the AASHTO equation is unconservative, with many slip/shear failures falling above the equation line. As can be seen in this same figure, the line representing the AASHTO equation multiplied by 1.6 is reasonably conservative, with only one slip/shear failure falling on this line.

For the high-strength concrete data shown in figure 37, both equations are conservative. However, the line representing 1.6 times the AASHTO equation is overly conservative. It requires a much longer development length than is needed. It provides no benefit (credit) to the designer for using high-strength concrete because it is simply a multiplier of an equation that provides no benefit (credit) for using high-strength concrete.

In figure 38, all of the development length data are plotted along with the Buckner equation predictions. Many slip/shear failures fell above the line, indicating that predicted development lengths were shorter than actual development lengths. This situation is unconservative.

The data are grouped by concrete strength in figures 39 and 40, with figure 39 graphing normal-strength concrete data and figure 40 plotting high-strength concrete data. These figures show that the Buckner equation is not conservative for members constructed with normal-strength concrete, but is conservative for members constructed with high-strength concrete.

The new FHWA development length equation is plotted along with all of the development length data in figure 41. No slip/shear failures fell above the line, indicating that it is a conservative equation. It is replotted in figures 42 and 43 with the normal-strength concrete data and high-strength concrete data, respectively. No slip/shear failures fell above the line; therefore, the FHWA equation is a conservative equation. Figure 43 shows that the FHWA equation is conservative for high-strength concrete data, without being overly conservative.

As seen in figures 23 through 43, the best choice for a development length equation is the FHWA development length equation (Equation (16)). It is conservative for all data, and provides a benefit to the designer for using high-strength concrete.

Although the current stop-gap measure of multiplying the AASHTO equation by 1.6 is reasonably conservative for the development length data, it is overly conservative and requires a much longer development length than is needed for members constructed with high-strength concrete. Since the AASHTO equation provides no benefit (credit) to the designer for using high-strength concrete, that same equation multiplied by 1.6 does not provide any benefit (credit) to the designer when using high-strength concrete. The new FHWA equation provides a better fit.

There were relatively few data points available for members with a concrete compressive strength, f'_c, over 10,000 psi. Therefore, until more data becomes available, it was determined that **values of f'_c greater than 10,000 psi shall be taken as 10,000 psi for use in Equation (16).**

THEORETICAL ANALYSIS FOR PILES

FHWA researchers wanted Equations (14) through (16) to apply not only to beams, but also to piles, because piles, under certain loading conditions, must resist flexural loads. However, there were limited data available on transfer and development lengths of pretensioned concrete piles. Therefore, it was decided that a theoretical parametric study would be performed using two different pile sections to see whether Equation (16) would be conservative for pretensioned piles.

Two sections were selected for this parametric study, a solid 610-mm- (24-in-) square section and a 760-mm- (30-in-) square section containing a 483-mm (19-in) concentric void (see figure 13).

The structural analysis computer programs that were used with the beam data were written to handle only flexural loads. Dr. Beshah and Dr. Gagarin modified the programs so that an axial load could be applied to the sections (piles), as well as the flexural load, to cover the situation of a ship impacting a pile. It was determined from this theoretical parametric study that the axial load had no detrimental effect on the development length, and that the FHWA development length equation (Equation (16)) was conservative for piles.

The effect of strand position during casting is an issue that was raised by Dr. Buckner is his study for FHWA.[17] The ACI code[6] and the AASHTO Specifications[7] acknowledge and account for the "top bar effect" for reinforcing bars (hereafter known as "rebars") in regular reinforced concrete. The top bar effect is the condition where rebars at the tops of members do not bond as well to the surrounding concrete as rebars at the bottom of members. Rebars that are placed with 305 mm (12 in) or more of concrete cast below (beneath) them are "top bars." As the bleed water (excess water) from the concrete beneath the rebars rises, some of it can be trapped directly beneath each rebar. The bond between the rebar and the concrete below the rebar is, therefore, weakened due to the presence of the bleed water. The ACI code and AASHTO Specifications prescribe that a multiplier of 1.3 must be applied to the rebar development length equation for these "top" rebars, so that additional development length is provided. Dr. Buckner concluded that the same "top bar effect" applies to top strands as well as to top rebars. He cites evidence from a University of Illinois study where the bond strength of strands was reduced by about 25 percent when strands had 254 mm (10 in) of concrete beneath them compared to 50.8 (2 in) of concrete beneath them.[55]

A conclusion from Dr. Buckner's study was that the development length should be multiplied by a factor of 1.3 for any strands (straight or draped) that end in the upper one-third of the member depth and have 305 mm (12 in) or more of concrete cast beneath them.[17,55]

Strand position during casting is an important issue for piles. Piles are cast horizontally and strands are placed around the perimeter of the piles. The top and bottom of the pile is not marked (noted) during casting. Once a pile is cast and removed from the prestressing bed, then its orientation (position) during casting is lost. Therefore, piles installed in a bridge have an anonymous casting position, and it is impossible to tell which strands were "top strands" or "bottom strands" during casting of the pile. Assuming there is 50.8 mm (2 in) of concrete cover for each layer of strands, any pile that has a cross-sectional dimension of 356 mm (14 in) or greater will have strands that have 305 mm (12 in) or more of concrete cast beneath them and fit the "top bar" criteria. Therefore, some of the strands in piles with a cross-sectional dimension of 356 mm (14 in) or greater will be subject to the "top bar effect," and these strands will have worse bond to the concrete. A ship could impact any side of the pile; it could impact the "top", "bottom", or "sides" of the pile (with "top", "bottom", and "sides" referring to the initial casting position). Therefore, a multiplier should be applied to the development length equation for strands in piles to ensure that all sides of a pile could successfully withstand a ship impact.

Recent research by the University of South Carolina indicated that top strands do exhibit greater end slip at detensioning than bottom strands—sometimes two to three times the amount of end slip for top strands as compared to bottom strands.[56]

While there is no ACI nor AASHTO equation linking end slip at detensioning to transfer and development lengths, end slip at detensioning is widely held as an indicator of bond. If there is a long end slip at detensioning, this results in long transfer and development lengths. Therefore, the trends noted by the University of South Carolina researchers are important indicators of longer transfer and development lengths for "top" strands in piles.

Based on all of the above, the FHWA researchers recommend that the FHWA transfer and development length equations (Equations (14) and (16), respectively) be used for piles, and that a 1.3 multiplier be applied to Equations (14) and (16) for any strands (straight or draped) in any member (beam or pile) that has 305 mm (12 in) or more of concrete cast beneath them. However, the 1.3 multiplier should not be applied to strands in beams that do not fit this criterion.

There should be more physical research conducted on pretensioned piles, either by the University of South Carolina and/or by other researchers in the future, to experimentally verify the findings of the theoretical analysis for piles and the top strand/top bar effect.

CHAPTER 5: SUMMARY, FINAL RECOMMENDATIONS, AND CONCLUSIONS

In 1988, FHWA issued a memorandum that outlawed the use of 15.2-mm- (0.6-in-) diameter strands, restricted the spacing of strands, and applied a multiplier to AASHTO's development length equation (Equation (1)).[14] This memorandum initiated considerable research on the subject of bond of pretensioned strands in concrete.

Forty-one research studies have been undertaken since 1988 to clarify the issues in the memorandum. If these studies are added to the bulk of research undertaken prior to the FHWA memorandum, a total of more than 60 studies have been conducted on the topic of transfer and development lengths of pretensioned strands in prestressed concrete.

Construction Technology Laboratories, Inc. (CTL) conducted a study for FHWA on the history of the AASHTO equation.[8] This study documented that the AASHTO transfer and flexural bond length expressions, which make up the development length equation (see Equations (1) and (2)), were actually average expressions, reflecting reasonable means for the data, rather than conservative expressions. Because both components of the AASHTO development length equation reflected reasonable means and not conservative values, the AASHTO equation itself can be expected to be only an average equation and not a conservative equation.

Another important study was conducted for FHWA by Dr. Dale Buckner.[17] In an effort to resolve conflicting recommendations from various researchers, Buckner reviewed the research results available up to 1993 on the topic of transfer and development lengths of pretensioned strands in prestressed concrete. He then proposed a new development length equation for pretensioned strands in concrete (Equation (3)).

Other researchers conducted studies on the same topic after the Buckner equation was proposed, including FHWA researchers. Because of this research, FHWA issued a memorandum in 1996 that lifted two of the provisions from the 1988 FHWA memorandum.[18]

The 1996 FHWA memorandum allowed the use of 15.2-mm- (0.6-in-) diameter strands and allowed reduced strand spacings for 12.7-mm- and 15.2-mm- (0.5-in- and 0.6-in-) diameter strands. However, the multiplier for the AASHTO development length equation was retained until a new development length equation proposed by FHWA was reviewed and commented upon by the AASHTO Bridge Subcommittee on Bridges and Structures.

This new FHWA development length equation emanated from a large research study conducted at FHWA's Structures Laboratory at the Turner-Fairbank Highway Research Center in McLean, VA. Phase I of the study involved 50 rectangular prestressed concrete specimens, while Phase II involved 64 members: 32 AASHTO Type II prestressed concrete I-beams and 32 prestressed concrete sub-deck panels. Half of these members for both phases contained uncoated strands, while the other half contained epoxy-coated strands. Only research results from the members containing uncoated strands were discussed in this report.

Results from the FHWA rectangular specimens (Phase I) indicated that the AASHTO transfer and development length equations were unconservative for members constructed with normal-strength concrete.[3,15-16]

Phase II was then undertaken by FHWA to evaluate the AASHTO and Buckner equations for full-size members. Results from the full-size beams showed that the AASHTO transfer and development length equations were unconservative for members constructed with normal-strength concrete. The AASHTO transfer and development length equations were conservative for full-size beams that were constructed with high-strength concrete.

Results from the FHWA I-beam research also demonstrated that the Buckner transfer length equation was unconservative for beams constructed with normal-strength concrete, but was conservative for members constructed with high-strength concrete.

The Buckner development length equation was inconsistent when compared to the measured values of development length in normal-strength concrete—sometimes the Buckner equation was unconservative (the predicted value was shorter than the measured value) and sometimes it was conservative (the predicted value was longer than the measured value). When compared to measured values for high-strength concrete, the Buckner development length equation values were consistently conservative (the predicted values were longer than the measured values).

It was also evident from the FHWA I-beam research that measured values of transfer and development lengths for a given strand size were shorter for high-strength concrete than for normal-strength concrete.

Because the AASHTO equation was unconservative for normal-strength concrete and the Buckner equation was inconsistent for normal-strength concrete, it was determined that a new development length equation was needed—an equation that could provide conservative predictions of transfer and development lengths for all concrete strengths, yet not be overly conservative for high-strength concretes.

FHWA researchers decided to formulate the new equation based on FHWA's full-size beam research results, and then correlate the equation with other research results to make sure that the equation would be representative of the total applicable data to date.

A list of possible parameters for use in new transfer and development length equations was compiled. Many of the parameters had been measured during FHWA's full-size beam experimentation; however, other parameters had to be determined from structural analyses of that measured data. Two computer programs for structural analyses were developed by Dr. Fassil Beshah and Dr. Nicolas Gagarin, based on the strain-compatibility method of analysis. These programs were used to determine the remaining parameters.

For the new transfer length equation, all of the parameters were investigated initially by inspecting the data and then evaluating the most promising parameters. The promising parameters were evaluated along with the measured 28-day values of transfer length from FHWA's full-size beams using regression analyses. FHWA researchers employed a statistical approach, using the coefficient of determination as an evaluation criterion, to choose a best-fit line. FHWA researchers slightly modified that best-fit line by rounding-off the constants and obtained a new transfer length equation (Equation (7)).

A similar approach was used to determine a new flexural bond length expression. A host of variables were investigated. Researchers initially inspected the data and then numerically evaluated the most promising parameters, along with the "measured" values of the flexural bond length. Because there is no way to directly measure the flexural bond length, the measured transfer length was subtracted from the embedment length being tested to obtain the "measured" flexural bond length. More than 3400 equation types were initially evaluated, including linear, non-linear, log (Base 10 and natural log), and semi-log (Base 10 and natural log) equations, using a computer program called TableCurve 2D. A simple equation filter was also used to ensure that the equation selected was not only an accurate equation, but also an understandable and usable equation.

TableCurve 2D used a DOF-adjusted coefficient of determination as an evaluation criterion between possible equations. FHWA researchers then chose the combination of variables and the equation that had the highest coefficient of determination, and was understandable and usable as the best obtainable flexural bond length equation. That best obtainable flexural bond length equation is given in Equation (8).

The new development length equation (Equation (9)) was the sum of the new transfer length equation (Equation (7)) and the new flexural bond length equation (Equation (8)). The FHWA researchers adopted Equation (9) as the proposed development length equation pending correlation with data from other studies.

FHWA undertook an extensive literature search on the topic of the bond of pretensioned strands in concrete and identified more than 60 studies on this topic. As was previously noted, these studies were on a variety of topics related to the bond of pretensioned strands. For some of the studies, the transfer and development length experimentation was the sole objective of the study; for others, transfer and development length experimentation was an important constituent of a broader study objective. FHWA researchers established data requirements or criteria that a study had to meet for it to be included in the correlation of the new transfer and development length equations. Criteria were established in the following areas:

- Concrete.
- Prestressing strand.
- Structural member types.
- Transfer and/or development length measurements.

All 62 of the bond-related studies were examined in detail to see whether their study focus, instrumentation, and data met the criteria described in the previous section. If their study focus, instrumentation, and data met the criteria described previously, then that study was included in the correlation. If their study focus, instrumentation, and data did not meet the criteria, then the study results were not included in the correlation. Also, if the study focus, instrumentation, and data met the criteria, but some data were not readily available, that data had to be given to FHWA researchers by a certain date to be included in the correlation process. If the data were not received by the cut-off date, then the study results were not included in the correlation.

Sixteen studies, in addition to FHWA's Phase I (rectangular specimens) data, met the criteria, had the data available by the cut-off date, and were used in the correlation. Many fine studies were not included in the correlation. This did not imply that those studies were invalid. It simply meant that those studies had objectives that were beyond the scope of this report, that measurements were made or materials used that were other than those described in the criteria above, or that the study data could not be supplied by the cut-off date.

The data from the following studies were used in the correlation:

- Auburn University.
- University of Colorado at Boulder.
- FHWA Phase I (rectangular specimens).
- Florida Department of Transportation.
- Preston and Janney.
- Louisiana State University and the University of New Orleans.
- McGill University.
- University of Minnesota.
- North Carolina State University.
- University of Oklahoma.
- Purdue University.
- University of South Florida and the University of Illinois at Chicago.
- University of Texas at Austin for Texas DOT.
- University of Texas at Austin—Louetta Road Overpass Project.
- University of Texas at Austin—San Angelo Bridge Project.
- Tulane University and Construction Technology Laboratories, Inc.
- Virginia DOT.

The data from the other studies often required FHWA researchers to conduct additional structural analyses. For the transfer length data, many researchers in other studies determined transfer length using methods such as the slope-intercept method, the 100-percent plateau method, or the 95-percent plateau method. Because the 95-percent plateau method was established as the recommended method for determining transfer length, FHWA researchers had to take the data from the other studies that had not used the 95-percent plateau method and determine the transfer lengths using the 95-percent plateau method.

If the parameters used in the development length equation (Equation (9)) were not provided in the study, then FHWA researchers had to perform a structural analysis for every member in that study to determine the missing parameters. This resulted in the creation of a third computer program for structural analysis (created by Dr. Beshah and Dr. Gagarin) to assist in structural analyses for members that had load and deflection data available, but not data on the external concrete strain of the top fiber, from their development length tests.

Statistical comparisons or correlations were then performed using the data from other studies and the proposed transfer, flexural bond, and development length equations (Equations (7) through (9)).

During this process of correlating Equations (7) through (9) with all of the data from other studies, a few other re-examinations and re-evaluations were conducted. The issue of unit weight as a possible parameter in Equations (7) through (9) was re-examined; the idea of using the area of strand rather than the diameter of strand as a parameter in the equations was re-evaluated; and the possibility of using log, natural log, semi-log, and natural semi-log as equation types was also evaluated. Statistical analyses were performed that determined coefficients of correlation for each of the cases described above. The coefficients of correlation were compared, and it was concluded that Equations (7), (8), and (9) were the best overall choices for the transfer length, flexural bond length, and development length equations, respectively.

Even though it was determined that these equations were the best overall equations, they still only represented the mean of the data (best fit). They were not conservative equations. Therefore, the constants for these equations were adjusted so that the equations were conservative; no longer were they best-fit equations, but they now exceeded the requirements for a 95-percent confidence interval for the data.

The recommended transfer length equation is as follows:

$$L_t = \frac{4 f_{pt} D}{f'_c} - 5 \tag{14}$$

The recommended flexural bond length equation is as follows:

$$L_{fb} = \frac{6.4\,(f^*_{su} - f_{se})(D)}{f'_c} + 15 \tag{15}$$

Equations (14) and (15) were added together to provide the recommended development length equation, which is as follows:

$$L_d = \left[\frac{4 f_{pt} D}{f'_c} - 5\right] + \left[\frac{6.4\,(f^*_{su} - f_{se})(D)}{f'_c} + 15\right] \tag{16}$$

It was noted during the review of these equations that there were relatively few data points for members with a concrete compressive strength, f'_c, over 10,000 psi. Therefore, until more data becomes available, FHWA researchers determined that **values of f'_c greater than 10,000 psi shall be taken as 10,000 psi for use in Equations (14), (15), and (16). Equations (14), (15), and (16) are English-unit equations. The values of the constants in Equations (14), (15), and (16) will change when the equations are converted for metric use.**

It was desired that the above equations (Equations (14) through (16)) apply not only to beams but also to piles, because piles, under certain loading conditions, must resist flexural loads. However, there were limited data available on piles. Therefore, a theoretical parametric study was performed using two different pile sections to see if the recommended development length equation (Equation (16)) was conservative for pretensioned piles.

The theoretical parametric study results concluded that the axial load existing in piles had no detrimental effect on the development length and that the FHWA development length equation (Equation (16)) was conservative for piles.

Another factor to consider when examining the topic of transfer and development lengths of pretensioned strands in prestressed concrete piles is the effect of strand position during the casting of the piles. Because piles are cast horizontally and strands are placed around the perimeter of the pile, evidence shows that strands at the top of pile sections during casting exhibit significantly more end slip at detensioning than do the bottom strands. Because end slip is widely held as an indicator of bond, the longer end slips at detensioning result in longer transfer and development lengths.

AASHTO and ACI have recognized and addressed the phenomenon of worse bond (longer development lengths) for top reinforcing bars (rebars) by their application of a 1.3 multiplier to the general development length equation for rebars. Therefore, **FHWA researchers recommend that the FHWA transfer and development length equations (Equations (14) and (16), respectively) be used for piles, and that a 1.3 multiplier be applied to Equations (14) and (16) for any strands (straight or draped) in any member (beam or pile) that has 305 mm (12 in) or more of concrete cast beneath them.** However, the 1.3 multiplier should not be applied to strands in beams that do not fit this criterion.

FHWA researchers hope that ongoing and/or future physical research on pretensioned piles can experimentally verify the findings of the theoretical analysis for piles, and further define the top strand effect.

The new FHWA transfer and development length equations, Equations (14) and (16), respectively, have been established on the basis of the FHWA full-size beam data and correlated conservatively with research results from other studies with the guidelines outlined above. They are recommended for immediate use in the design of pretensioned beams and piles.

APPENDIX A: NOTATION

Report Symbol	ACI and AASHTO LRFD Notation	AASHTO Standard Specification Notation	Description
a	a	a	Depth of concrete rectangular stress block
A_s*	A_{ps}	A_s*	Area of prestressed reinforcement in the tension zone
b			y-intercept
D	d_b	D	Nominal diameter of prestressing strand
E_c	E_c	E_c	Modulus of elasticity of concrete at 28 days
E_{ci}	E_{ci}	E_{ci}	Modulus of elasticity of concrete at detensioning
E_s	E_{ps}	E_s	Modulus of elasticity of prestressed reinforcement
f'_c	f'_c	f'_c	Concrete compressive strength at 28 days
f'_{ci}	f'_{ci}	f'_{ci}	Concrete compressive strength at detensioning
f_{pt}			Stress in prestressing strand prior to transfer of prestress
f_r	f_r	f_r	Modulus of rupture of concrete
f_{se}	f_{se}	f_{se}	Effective stress in prestressed reinforcement after all losses
f_{si}			Stress in prestressed reinforcement at time of initial prestress (immediately after release in a pretensioned member)—from Buckner report
f^*_{su}	f_{ps}	f^*_{su}	Average stress in prestressed reinforcement at ultimate load
L_d	ℓ_d		Development length
L_e			Embedment length for development length test
L_{fb}			Flexural bond length

Report Symbol	ACI and AASHTO LRFD Notation	AASHTO Standard Specification Notation	Description
L_t			Transfer length
m			Slope of line
w_c	w_c	w_c	Unit weight of concrete
x			Independent variable
y			Dependent variable
ϵ_{ps}			Strain in prestressed reinforcement at nominal strength—from Buckner report
ϵ_{su}			Strain in prestressing strand at ultimate strength of the member
λ			Multiplying factor applied to flexural bond length term in Buckner equation—from Buckner report

APPENDIX B: RESEARCH STUDIES ON THE BOND OF PRETENSIONED STRANDS IN CONCRETE

Authors	Organization	Title	Date
Marshall	University of Leeds	End Anchorage and Bond Stress in Prestressed Concrete	1949
Janney	Portland Cement Association	Nature of Bond in Pre-Tensioned Prestressed Concrete	1954
Base	Cement and Concrete Association	Some Tests on the Effect of Time on Transmission Length in Pre-Tensioned Concrete	1957
Hanson, N. and Kaar	Portland Cement Association	Flexural Bond Tests of Pre-Tensioned Prestressed Beams	1959
Kaar, LaFraugh, and Mass	Portland Cement Association	Influence of Concrete Strength on Strand Transfer Length	1963
Preston and Janney	Colorado Fuel and Iron Corporation, and Wiss, Janney, Elstner and Associates, Inc.	Characteristics of 15% Stronger 7-Wire Strand	1963
Anderson, G.; Rider; and Sozen	University of Illinois at Urbana	Bond Characteristics of Prestressing Strand	1964
Arthur and Ganguli	University of Glasgow	Tests on End-Zone Stresses in Pre-Tensioned Concrete I-Beams	1965
Hanson, J. and Hulsbos	Lehigh University	Overload Behavior of Pretensioned Prestressed Concrete I-Beams With Web Reinforcement	1965
Kaar and Magura	Portland Cement Association	Effect of Strand Blanketing on Performance of Pretensioned Girders	1965
Over and Au	Carnegie Mellon	Prestress Transfer Bond of Pretensioned Strands in Concrete	1965

Authors	Organization	Title	Date
Hanson, N.	Portland Cement Association	Influence of Surface Roughness of Prestressing Strand on Bond Performance	1969
Hanson, N. and Kaar	Portland Cement Association	Bond Fatigue Tests of Beams Simulating Pretensioned Concrete Crossties	1975
Anderson, A. and Anderson, R.	Concrete Technology Corporation	An Assurance Criterion for Flexural Bond in Pretensioned Hollow-Core Units	1976
Martin, L. and Scott	Consulting Engineers Group, Inc.	Development of Prestressing Strand in Pretensioned Members	1976
Zia and Mostafa	North Carolina State University	Development Length of Prestressing Strands	1977
Rabbat; Kaar; Russell, H.; and Bruce	Portland Cement Association and Tulane University	Fatigue Tests of Pretensioned Girders With Blanketed and Draped Strands	1979
Horn and Preston	Computerized Structural Design, and Wiss, Janney, Elstner and Associates, Inc.	Use of Debonded Strands in Pretensioned Bridge Members	1981
Cousins, Johnston, and Zia	North Carolina State University	Bond of Epoxy-Coated Prestressing Strand	1986
Ghosh and Fintel	University of Illinois at Chicago	Development Length of Prestressing Strands, Including Debonded Strands, and Allowable Concrete Stresses in Pretensioned Members	1986
Lin	Construction Technology Laboratories, Inc.	Test of Epoxy-Coated Strand at High Temperatures	1989
Brearley and Johnston	North Carolina State University	Pull-Out Bond Tests of Epoxy-Coated Prestressing Strand	1990
Loov and Weerasekera	University of Calgary	Prestress Transfer Length	1990

Authors	Organization	Title	Date
Deatherage and Burdette	University of Tennessee at Knoxville	Development Length and Lateral Spacing Requirements of Prestressing Strand for Prestressed Concrete Bridge Products	1991
Deatherage and Burdette	University of Tennessee at Knoxville	Development Length and Lateral Spacing Requirements of Epoxy-Coated Prestressing Strand for Prestressed Concrete Bridge Products	1991
Fagundo and Richardson	University of Florida at Gainesville	Bond Slip of Epoxy-Coated Strands at Elevated Temperatures	1991
LeClaire	University of Wisconsin at Milwaukee	The Effect of Temperature on the Bond Strength of Epoxy-Coated Prestressing Strand	1991
Shahawy and Issa	Florida Department of Transportation	Effect of Pile Embedment on the Development Length of Prestressed Strands	1991
Cousins, Badeaux, and Moustafa	Louisiana State University and the University of New Orleans	Proposed Test for Determining Bond Characteristics of Prestressing Strand	1992
Nanni, Utsunomiya, Yonekura, and Tanigaki	Pennsylvania State University	Transmission of Prestressing Force to Concrete by Bonded Fiber-Reinforced Plastic Tendons	1992
den Uijl	Delft University of Technology	"Relation Between Draw-In and Transmission Length in Pretensioned Concrete Members Cut Off By Sawing," and "Bond and Splitting Action of Prestressing Strand"	1992
Balazs	Budapest University of Technology	Transfer Control of Prestressing Strands, and Transfer Length of Prestressing Strand as a Function of Draw-In and Initial Prestress	1992 & 1993
Gopu, Cousins, and Francis	Louisiana State University and Auburn University	Spacing and Cover of Epoxy-Coated Prestressing Strands	1993

Authors	Organization	Title	Date
Shahawy	Florida Department of Transportation	An Investigation of Shear Strength of Prestressed Concrete AASHTO Type II Girders	1993
Mitchell, Cook, Khan, and Tham	McGill University	Influence of High-Strength Concrete on Transfer and Development Length of Pretensioning Strand	1993
Cousins, Stallings, and Simmons	Auburn University	Effect of Strand Spacing on Development of Prestressing Strand	1993
Abdalla, Ramirez, and Lee	Purdue University	Strand Debonding in Pretensioned Beams—Precast Prestressed Concrete Bridge Girders With Debonded Strands	1993
Russell, B. and Burns	University of Texas at Austin	Design Guidelines for Transfer, Development, and Debonding of Large-Diameter Seven-Wire Strands in Pretensioned Concrete Girders	1993
Issa, Sen, and Amer	University of South Florida and University of Illinois at Chicago	Comparative Study of Transfer Length in Fiberglass and Steel Pretensioned Concrete Members	1993
Buckner	Virginia Military Institute	An Analysis of Transfer and Development Lengths for Pretensioned Concrete Structures	1994
Abendroth, Stuart, and Yuan	Iowa State University	Precast Concrete Panel Thickness for Epoxy-Coated Prestressing Strands	1994
Bruce, R.; Martin, B.; Russell, H.; and Roller	Tulane University and Construction Technology Laboratories, Inc.	Feasibility Evaluation of Utilizing High-Strength Concrete in Design and Construction of Highway Bridge Structures	1994
Patel	Pennsylvania Department of Transportation	Development Length of P/S Strands	1994
den Uijl	Delft University of Technology	Transfer Length of Prestressing Strand in HPC	1995

Authors	Organization	Title	Date
Tawfiq	Florida State University	Cracking and Shear Capacity of High-Strength Concrete Bridge Girders	1995
Ahlborn, French, and Leon	University of Minnesota	Applications of High-Strength Concrete to Long-Span Prestressed Bridge Girders	1995
Martin, L. and Korkosz	Consulting Engineers Group, Inc.	Strength of Prestressed Concrete Members at Sections Where Strands Are Not Fully Developed	1995
Russell, B. and Paulsgrove; and Russell, B. and Rose	University of Oklahoma	"Fundamental Mechanisms for the Development of Pretensioned Strands," and "Investigation of Standardized Tests to Measure the Bond Performance of Prestressing Strand"	1996
Petrou and Joiner	University of South Carolina	Continuing Investigation of Strand Slippage in 24-inch Octagonal Prestressed Concrete Piles	1996
Soudki, Green, and Clapp	Queen's University	Transfer Length of Carbon Fiber Rods in Precast Pretensioned Concrete Beams	1997
Ehsani, Saadatmanesh, and Nelson	University of Arizona/FHWA	Transfer and Flexural Bond Performance of Aramid and Carbon FRP Tendons	1997
Logan	Stresscon Corporation	Acceptance Criteria for Bond Quality of Strand for Pretensioned Prestressed Concrete Applications	1997
Lane	FHWA	A New Development Length Equation for Pretensioned Strands in Bridge Beams and Piles	1998
Cooke, Shing, and Frangopol	University of Colorado at Boulder	Colorado Study on Transfer and Development Length of Prestressing Strand in High-Performance Concrete Box Girders	1998

Authors	Organization	Title	Date
Gross, Burns, Carrasquillo, and Ralls	University of Texas at Austin and Texas Department of Transportation	Transfer and Development Length of 15.2-mm-(0.6-in-) Diameter Prestressing Strand in High-Performance Concrete: Results of the Hoblitzell-Buckner Beam Tests [part of the High Performance Concrete Louetta Road Overpass Project]	Ongoing
Shah, Gross, Cordova, Burns, Carrasquillo, Fowler, and Ralls	University of Texas at Austin and Texas Department of Transportation	Bond Behavior of 0.6-inch-Diameter Prestressing Strand at 2-inch Grid Spacing in Fully Bonded High-Strength and Normal-Strength Composite Texas Type C Beams [part of the High Performance Concrete San Angelo Bridge Project]	Ongoing
Burns, Kreger, and Burkett	University of Texas at Austin and Texas Tech University	Development Length of 0.6-inch-Diameter Prestressing Strand at 2-inch Grid Spacing in Standard I-Shaped Pretensioned Concrete Beams [Texas Department of Transportation State Planning and Research Study #1388]	Ongoing
Barr, Fekete, Eberhard, Stanton, Janssen, Khaleghi, and Hsieh	University of Washington and Washington State Department of Transportation	High-Performance Concrete in Washington State SR516 Overcrossing, Eastbound Bridge No. 18/25S [part of the Washington State High Performance Concrete Bridge Project]	Ongoing
Ozyildirim and Gomez	Virginia Transportation Research Council and Virginia Department of Transportation	High-Performance Concrete in a Bridge Structure in Richlands, Virginia [part of the Virginia High Performance Concrete Bridge Project]	Ongoing
Baseheart, Miller, and Shahrooz	University of Cincinnati and Ohio Department of Transportation	"Use of High-Performance Concrete for an Adjacent Box Girder Bridge" [part of the Ohio High Performance Concrete Bridge Project]	Ongoing
Venuti	San Jose State University	Bond of Indented Strand for Railroad Ties	Ongoing

Authors	Organization	Title	Date
Zena and Albrecht	University of Maryland/FHWA	Investigation of Transfer and Development Length of Lightweight Prestressed Concrete Members	Ongoing
Lybas and Fagundo	University of Florida at Gainesville	Effect of Internal and External Constraints on the Bond and Shear Strength of Precast Prestressed Concrete Girders	Ongoing
Ramirez	Purdue University	Investigation of Development Lengths of 0.5-in- and 0.6-in-Diameter Strands for Semi-Lightweight and High-Performance Concrete [Indiana Department of Transportation State Planning and Research Study]	Ongoing
Petrou	University of South Carolina	Continuing Investigation of Strand Slippage in Prestressed Concrete Piles—Phase II [South Carolina Department of Transportation State Planning and Research Study]	Ongoing
Jacob and Russell, B.	University of Oklahoma	Pull-Out Testing at the University of Oklahoma	Ongoing

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